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Internship report: Waste Treatment Technologies

Design of a wastewater treatment plant for Anaerobic Digestion systems

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Foreword and acknowledgements

This internship report has been written as element of the master mechanical engineering at the University of Twente. The assignment is relevant for both existing as well as new build installations of Waste Treatment Technologies, which is a company active in the waste treatment industry. The company profile will be further elaborated in the introduction.

I would like to use this foreword to thank Waste Treatment Technologies for the opportunity to perform this assignment at their company. More specific, I would like to thank Sander ten Hove for the great support, his valuable insights and the guidance. Furthermore a big thanks to Justin Asma for bringing me into the company and to all the WTT employees for the great integration into their company.

Moreover, I would like to thank Kees Venner for his open-minded view on this internship, since it is not really a mechanical engineering topic. Lastly, the Massive Open Online Course (MOOC) of Delft University and EdX has been of great value for this research, especially as basis for all technologies.

For the calculations and dimensioning, an Excel-file has been made named "*Purification Calculation Sheet.xlsx*", to which is refered several times in this report.

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Nomenclature and abbreviations

Symbol	Standard unit	Meaning
A	m^2	Area of a certain tank, could be either frontal or bottom area
BOD_5	mg O_2/L	Biochemical Oxygen Demand (in Dutch: BZV, in German: BSB)
B_D	kg BOD/d	Total BOD supply in the wastewater
B_L	kg BOD/(m^3·d)	BOD-load in a trickling filter
B_X	kg BOD/(kg ds·d)	Sludge loading rate, often referred to as F/M-ratio (food-to-mass)
COD	mg O_2/L	Chemical Oxygen Demand (in Dutch: CZV, in German: CSB)
D	m	Diameter of a certain (circular) tank
DO	mg O_2/L	Dissolved Oxygen
d	m	Depth of a certain tank (sometimes referred to as h)
ds	mg/L	dry solids concentration (in German: ts)
h	m	Height of a certain tank (sometimes referred to as d)
HRT	s, h, d	Hydraulic Retention Time
L	m	Length of a certain tank
N_D	kg N/d	Total nitrogen supply in the wastewater
OD	mg O_2/(L·d)	Oxygen Demand (total O_2 required for the process)
Q	m^3/d	Volume flow rate of the wastewater
R	-	Recycle flow ratio
S	kg BOD/m^3	Food in the wastewater, expressed as the amount of BOD
SRT	h, d	Sludge Retention Time (sometimes referred to as θ)
SS	mg/L	Suspended Solids concentration, removed by sedimentation
SVI	mL/g	Measure for the sludge settleablity
SVLR	m^3/(m^2·h)	Sludge Volume Loading Rate
v	m/s	Velocity of the wastewater
V	m^3	Volume of a certain tank
V_0	m^3/(m^2·h)	Surface loading
V_S	L/(m^2·h)	Sludge volume surface loading rate
W	m	Width of a certain tank
X	kg ds/m^3	Sludge concentration (X_{AT} is aeration tank, X_R is return sludge, X_E is effluent)
Y	kg ds/kg BOD	Biomass Yield (growth) (dependent on B_X)
τ	s, h, d	Time of a certain process

Abbreviation	Meaning
AD	Anaerobic Digestion: breakdown of biodegradable waste by microorganisms, generating biogas
CSTR	Continuous Stirred-Tank Reactor
DAF	Dissolved Air Flotation
FOG	Fat, Oil & Grease
MBR	Membrane Bio Reactor
MSW	Municipal Solid Waste
PAO	Phosphate Accumulating Organisms
PE	Population Equivalent
PNID	Piping and Instrumentation Diagram
RO	Reverse Osmosis
SSO	Source Separated Organics (in Dutch: GFT)
TSS	Total Suspended Solids
UASB	Upflow Anaerobic Sludge Blanket
VSS	Volatile Suspended Solids
WTT	Waste Treatment Technologies: company providing this internship
WWTP	Wastewater Treatment Plant (in dutch: AWZI or RWZI)

Abstract

The goal of this research is to investigate the feasibility of installing a wastewater treatment plant (WWTP) at the anaerobic digestion tunnel-installations of Waste Treatment Technologies (WTT), so that the water can be discharged to the sewers or surface water. The current situation at all existing installations is that redundant and too polluted wastewater is discharged to existing (large) WWTP's using 20 m^3 trucks for a fixed price per cubic meter. Depending in the size of the installation and the annual water discharge, these costs of transportation/discharging can be as high as €400,000.- per year, directly showing the great opportunity for a WWTP.

Usually the water that has to be cleaned exists of two main streams, being leachate from the tunnels and condensate from the piping. The exact volume distribution of the flows is dependent on the installation, but the condensate is way less contaminated compared to the leachate. To fulfil requirements, the leachate could be "diluted" with condensate water, which is especially important to provide sufficient C-source for nitrogen removal.

It turns out that for plants with sufficient annual water discharge, e.g. $\geq 5000\ m^3/a$, the installation of a WWTP could definitely be beneficial. In case the volume is lower, one could consider retaining the current discharge method.

A basic design for an industrial WWTP has been made in this report, together with an explanation of all components. In section 7, a rough costs analysis has been made for a specific example. Ideally a modular design must be made for total discharge volumes between 5000 and 20,000 m^3/a, where standard SKID containers are used for the control and machinery, while scalable tanks are used for the actual treatment of the water. A more detailed design must be made with a company which has more experience in the wastewater treatment business, such as Nijhuis Industries or Colubris CleanTech, so WTT could offer WWTP's for future anaerobic digestion installations.

1 Introduction

In this chapter a little introduction in the business of Waste Treatment Technologies will be given, after which the exact assignment will be further explained.

1.1 What does Waste Treatment Technologies do?

Waste Treatment Technologies (WTT) lies its focus on the organic part of household waste. Treatment technologies vary somewhere between burning waste and using it for landfill, but WTT focuses on everything in between (i.e. composting and anaerobic digestion). This is done by combining both mechanical and biological treatment.

The customers of WTT earn their money in several ways, they receive an entrance fee (€85/t waste) from municipalities to treat the waste. Furthermore, using mechanical treatment, materials such as heavy metals (€200/t), paper (€80/t) and all kinds of plastics (€40-200/t) can be separated and sold. Note that all prices are purely indicative.

The remainder is mostly organic waste, which is treated biologically to further increase revenues. Two processes can be divided, one creating biogas (electricity) and one creating compost (used as fertiliser).

1.1.1 Mechanical treatment

The mechanical part is practically a big sorting machine, removing recyclable materials from the mixed waste input. Valuable materials such as metals, glass, paper and plastic can be removed preceding the biological treatment. This is done using for instance electromagnetic machines, or industrial conveyors, shredders and cyclone separators.

1.1.2 Biological treatment

As mentioned above, biological treatment can be roughly divided into two categories. The first one, creating biogas for electricity generation, is called anaerobic digestion (AD), while the second process uses the remaining material to make compost, used as fertiliser for the land.

Anaerobic digestion:
The incoming waste material is placed inside a large tunnel. The anaerobic digestion process uses anaerobic bacteria to produce biogas (CO_2+CH_4). When the optimal temperature for anaerobic digestion, 38^oC, is reached, the air will be recirculated. In this way, all oxygen contained in the air will be used by the bacteria, creating an anaerobic environment. In order to ensure anaerobic conditions (no oxygen), these tunnels are gas-tight.

These systems are installed with a percolation system, which could be compared to the sprinkler system inside a building. The percolate is of great importance, since biogas generation is activated/stimulated with this water. AD installations have a very large percolation buffer tank. After the biogas production, the remaining material in the tunnels can be used for composting.

Composting:
Composting systems are large tunnels filled with organic material, which is suited with an aeration system. The air comes from the main hall and flows through very tiny pores in the bottom of the tunnel. Aerobic microorganisms (consuming oxygen and waste) convert organic waste into compost. Based on oxygen levels, the temperature can be controlled in the range of around 50^oC.

During the process, the humidity is controlled by the percolation system, similar to the AD process. Although this process does not have such a large reservoir. Water can be sprayed onto the material inside the tunnel through these sprinklers. Just as for AD, this percolate water cannot be reused over and over again due to pollution which is caused during the process. Loads of polluting substances of the organic waste dissolves into the water.

1.1.3 Post-treatment

The content of the readily existing the post-treatment, for example air treatment consisting of acid humidifiers (also called scrubbers) removing the NH_3 from the air and bio-filters removing bad odours from the air, are outside the scope of this project. The actual scope will be the introduction of another type of post-treatment: water purification.

1.2 What is the exact topic of this internship research?

As mentioned in the previous paragraph, the polluted (waste)water generated by the processes of paragraphs 1.1.2 and 1.1.2 cannot be reused forever. Currently, the excess wastewater is collected by large water trucks (20 m^3) for a fairly high price. Especially for AD installations, the amount of excess waste water is relatively high (up to 15,000 m^3/y). Since the disposal price per ton lies around €27.50/t, this results in more than €400,000,- annual costs for disposing wastewater. The environmental impact of trucks is not even considered.

The problem statement of this internship therefore is whether it could be financially beneficial to design and integrate the treatment of wastewater into the system, to dispose the water to the sewers or even surface water. Ideally the design should fit in a modular standard sized container (i.e. 20- or 40-ft containers), since WTT builds most of the installation in SKID-containers.

2 Background information and literature research

The outside dimensions of a 20-ft container are (l·w·h) 5900x2352x2395 mm and the dimensions of a 40-ft container are (l·w·h) 12030x2352x2395 mm. The standard door size (w·h) is 2340x2292 mm for both. More research is required to choose which containers and configuration fits best. This is dependent on the amount of wastewater, the quality of wastewater influent and the effluent demands for discharging.

2.1 Wastewater quality

The wastewater, also referred to as leachate or percolate, has certain influent characteristics and legal effluent demands. Both can significantly deviate. The influent characteristics are mostly determined by the type of installation and the mixed waste input. The effluent demands are dependent on the legislation of the country where the installation is placed.

To give a better understanding, a small introduction into Suspended Solids (SS), Biochemical Oxygen Demand (often called BOD_5, BOD_5^{20} or simply BOD) and Chemical Oxygen Demand (or COD) is given. Suspended Solids are all non-dissolved particles in a certain volume of water, these could be separated by filtration, sedimentation or centrifugation. Both BOD and COD are a measure for the degree of pollution of wastewater. The dirtier the wastewater, the more oxygen is required to treat the pollutants. BOD is the amount of oxygen required for the microorganisms to remove all organic material. COD is the amount of oxygen required for the chemical removal of all (in)organic pollutants. Since the first one is part of the latter, the value of COD will always be higher.

2.1.1 Legal dumping regulations

In table 2.1, the maximum effluent concentrations of what is considered to be the most important pollution measurements can be found for Canada and several countries inside the EU. An important note is that the values have been taken for a Wastewater Treatment Plant (WWTP) of ≤ 10,000 Population Equivalent (PE), since larger WWTP's have more strict effluent demands. The first three pollutants have been explained in the paragraph above, and P and N stands for Phosphorus and Nitrogen. As can be seen from the table, especially Switzerland demands very clean water.

Effluent limit	**BOD_5**	**COD**	**SS**	**P-total**	**N-total**
Netherlands [6]	25 mg O_2/L	125 mg O_2/L	30 mg SS/L	2 mg P/L	15 mg N/L
Germany [7]	40 mg O_2/L	120 mg O_2/L	35 mg SS/L	1 mg P/L	12 mg N/L
Belgium [8]	54 mg O_2/L	135 mg O_2/L	90 mg SS/L	2 mg P/L	10 mg N/L
Switzerland [6]	20 mg O_2/L	60 mg O_2/L	20 mg SS/L	0.8 mg P/L	2.5 mg N/L
Canada [6, 9]	25 mg O_2/L	125 mg O_2/L	25 mg SS/L	1 mg P/L	25 mg N/L
Italy [10]	40 mg O_2/L	160 mg O_2/L	80 mg SS/L	10 mg P/L	30 mg N/L

Table 2.1: Sewer/surface water discharge concentration regulations

2.1.2 Leachate quality from (historic) samples

Since the effluent demands are known, the influent characteristics should be determined. Therefore actual data of readily installed systems is used. WTT has measured and documented some leachate data of several projects. The measurements are summarised and averaged in table 2.2 below.

Quantity	**BOD_5**	**COD**	**SS**	**P-total**	**N-total**
Unit	mg O_2/L	mg O_2/L	mg SS/L	mg P/L	mg N/L
Surrey	1570	n.a.	134	n.a.	223
Venneberg	1900	15970	2340	176	180
Wiefels	n.a.	n.a.	28.8	n.a.	330
Blaringhem	1600	9523	n.a.	130	n.a.
Neuss	n.a.	n.a.	n.a.	74	210
Mean	**1690**	**12750**	**834**	**126**	**236**

Table 2.2: Leachate data of several full-scale anaerobic digestion systems

Most of these numbers are averages of many samples, e.g. the numbers for Surrey are the average of 30+ samples, and the same goes for Blaringhem. Although this is a great guideline for designing, of course it should be kept in mind all the time that (large) deviations from these averages occur. For instance, the COD-value for the plant in Giuliano is as high as 45000 mg/L (with a BOD value of 8000 mg/L). Also the nitrogen concentration can be as high as 3000 mg/L, resulting in an enormous increase of aeration costs.

As a rule of thumb, the biological treatability of wastewater can be conducted from the COD/BOD ratio. Roughly, it can be said that a ratio below 2 indicated aerobic treatability, while anything higher than 3 requires sophisticated treatment techniques. From table 2.2 can be concluded that it is safe to say that the wastewater is very polluted.

When combining tables 2.1 and 2.2 we obtain the removal percentages as seen in table 2.3, which are quite impressive. Each of the categories should have a removal percentage of at least 90 %.

Quantity	**BOD_5**	**COD**	**SS**	**P-total**	**N-total**
Influent	1690 mg O_2/L	12750 mg O_2/L	834 mg SS/L	126 mg P/L	236 mg N/L
Effluent	20 mg O_2/L	125 mg O_2/L	30 mg SS/L	2 mg P/L	15 mg N/L
Removal	1670 mg O_2/L	12625 mg O_2/L	804 mg SS/L	124 mg P/L	221 mg N/L
Removal perc.	98.8 %	99.0 %	96.4 %	98.4 %	93.6 %

Table 2.3: Absolute and relative removal requirements (based on Dutch effluent demands)

2.2 WWTP components and design proposal

Some very high-tech solutions for wastewater treatment exist, such as making use of Anammox bacteria or hydrodynamic cavitation [11], but for this process the more classical approach is taken into consideration. A general layout of such a large classical residual wastewater treatment plant can be seen in figure 2.1, this picture as well as some information is extracted from the "Introduction to Treatment of Urban Sewage"-course of DelftX [1].

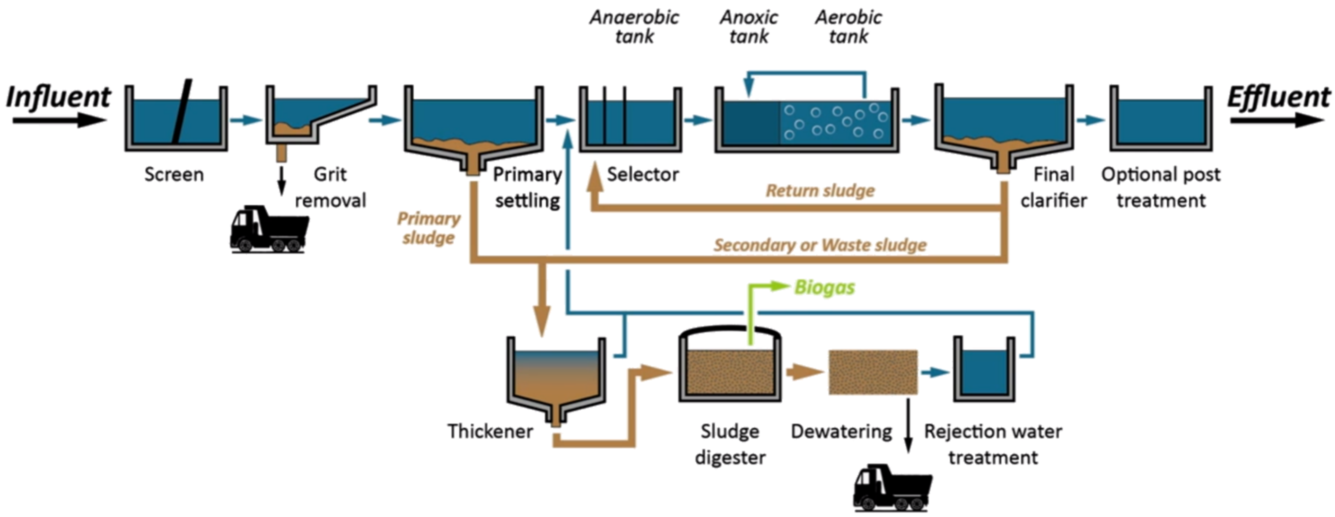

Figure 2.1: Overview of a traditional wastewater treatment plant [1]

The first component, the screen, removes large non-sewage components such as toilet paper, plastic pieces, sanitary towels and various others items between 5 and 100 mm. This is not necessary for the purpose of this project, since such large items will not be present in the wastewater.

The second component, the grit removal tank, is often used to elongate the lifespan of the pumps. Besides, a layer of sand at the bottom of the digestion tank, which could decrease the efficiency, is prevented. Usually this is designed for particles $\geq$ 0.15 mm [12]. Usually the size of this tank is small, due to the short Hydraulic Retention Time (HRT).

An alternative to the grit removal tank is a drum sieve, i.e. the RBS500 from NieuweWeme. This is a self-cleaning device with the same objective as a grit removal tank.

Next up, not shown in figure 2.1, a flocculation device could be installed to enhance the flock-forming process, providing better settleability characteristics of the sludge. This decreases the HRT. Often pipe flocculators (also called coagulators) are used.

Thereafter stands the primary sedimentation tank. In this tank, as many of the settleable undissolved particles are removed. Usually rectangular sedimentation tanks are used in combination with chain scrapers. The sludge will be discharged to the sludge thickener.

An alternative to the primary sedimentation tank is the Dissolved Air Flotation (DAF) device. This device uses aeration at the bottom, making the particles float, so they can be removed using a scraper at the water surface.

So far the removal is completely mechanical. After the primary sedimentation tank the biological part starts, consisting of three chambers and a final clarifier. The latter is comparable to the primary settling tank. The main goal is to reduce BOD and COD, but also the removal of both N and P happens in the biological section. The three chambers all have their own purpose and environment, which will be treated extensively in chapter 4. For now, these three environments are defined and distinguished as follows [13]:

- *Anaerobic*: completely free of the O-atom, i.e. no molecular oxygen (O_2) and bound oxygen (e.g. NO_2, NO_3 from nitrification process).
- *Anoxic*: environment free of molecular oxygen (O_2), with presence of bound oxygen (NO_2, NO_3).
- *Aerobic*: both free oxygen (O_2) and bound oxygen (NO_2, NO_3) are present. In fact, often an aeration machine is installed in such tanks, to ensure sufficient O_2.

The anaerobic tank is also called the selector, and is a small contact tank (HRT of 20-60 min), where the return sludge is mixed with the influent. In this tank, due to the anaerobic conditions, filamentous bacteria species cannot grow. However it stimulates the growth of the non-filamentous Phosphate Accumulating Organisms (PAO) [12], which will be explained broadly in chapter 6. An often used anaerobic device is the Upflow Anaerobic Sludge Blanket (UASB). Preceding to the anaerobic tank is a heat exchanger, to reach the optimal temperature for mesophillic microorganisms, around 38^oC.

The anoxic tank stimulates the denitrification process, which is part of the nitrogen removal process. Denitrification transforms NO_2/NO_3 into NO, N_2O and N_2 gases using an organic C-source, e.g. methanol.

The aerobic tank has some kind of aeration system, e.g. Continuous Stirred-Tank Reactor (CSTR) or fine bubble aeration strips on the bottom, supplying oxygen (air in fact, due to financial concerns). It has several functions, amongst which the removal of COD and BOD is the main concern. Besides, in this tank the P-uptake of PAO organisms takes place. The PO_4^{3-} leaves the water and can be discharged through the sludge/PAO's [5]. Lastly the nitrification process takes place, which is part of the nitrogen removal process. Nitrification transforms NH_4^+ into NO_2 and NO_3. Later these are transported through the internal N-cycle and will be denitrified to N_2, N_2O, and NO in the anoxic tank.

The sludge formed in these three tanks is allowed to sediment in the final clarifier, which has a very similar design as the primary sedimentation tank. An alternative is a Membrane Bio Reactor (MBR) to hold off the sludge, this has its own (dis)advantages compared to a final clarifier, which is explained in section 4.3

An alternative to these three tanks is the principle of a trickling filter, which is a huge tank filled with packing material (lava rocks or porous plastic). Bacteria can grow on this material, which has a very high specific area. Therefore the wastewater flowing along gets treated and leaves cleaner. More in depth explanation will be given in chapter 4.2. In this process, only N-removal takes places (and only if the surface load is sufficiently low), so a solution must be found for P-removal. An advantage of a trickling filter compared to the aeration tanks, is the amount of sludge generated is way less. The biggest disadvantage is the startup time of approximately 2 months, depending on the season.

If designed correctly, the effluent should now be sufficiently clean to discharge to either the sewer or surface water. If not, a optional post treatment is possible. Furthermore, part of the sludge is reused as indicated with "return sludge" and is called "activated sludge". The reason will be explained in section 4.1 as well. The excess waste sludge will be combined with the sludge from the primary settling tank and can be treated by the readily existing AD and composting systems from WTT. The bottom part of figure 2.1 will therefore be outside the scope of this research.

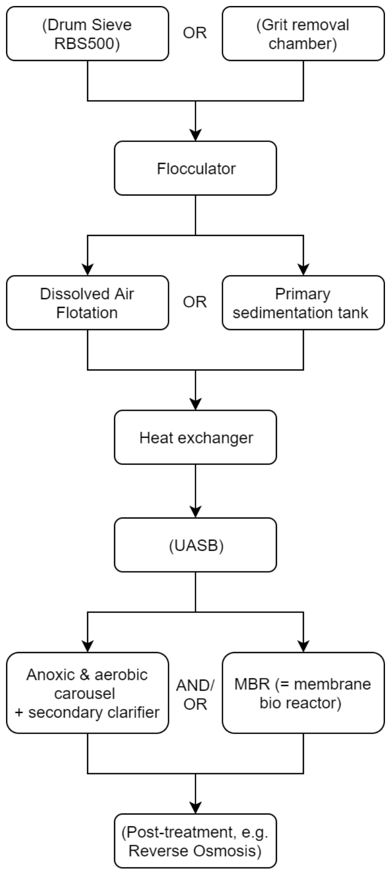

Figure 2.2: Suggested WWTP layout, components between brackets are optional

A design proposal for the situation described in section 1.2 can be seen in figure 2.2. All the components between brackets are optional, and also the alternatives mentioned in this paragraph are shown in the figure. In the coming sections all individual components will be further elaborated.

3 Design of the mechanical treatment

The first thing to consider while designing is the annual wastewater volume and the maximum instantaneous flow. WTT has data available of their readily installed full scale installations. Therefore the the annual discharge volumes are known. A few things can be seen from the annual discharge data: when comparing composting installations with AD installations, the latter overproduces loads of water, while for composting installations this is nearly zero (or even a water requirement). Therefore only AD installations will be considered for the wastewater treatment container. Typical AD installations, such as the readily existing Vennerberg, can produce up to 2000 m^3 of leachate per year, which is the equivalent of about 4 m^3 per day for a continuous flow over the year. Due to the buffer capacity of AD installations, a very decent continuous flow can be achieved throughout the year. Although there might be disturbances, varying influent characteristics and very contaminated leachate might need to be diluted, increasing the flow. Besides, when using a safety factor, also larger installations might be suitable for this design in the future. This results in a safety factor of 1.5, meaning a daily flow rate of 8 m^3 as design characteristic (equivalent to 0.33 m^3/hour).

The working principle of a wastewater treatment plant has already be explained in section 2.2, but not each component is necessary. Therefore only the components considered necessary, as seen in figure 2.2, are described below. The dimensions can be seen below, although the actual dimensioning is found in appendix B.

3.1 Grit removal chamber

There are several grit chamber designs, but the shape that makes the most sense for this design is a rectangle. Therefore the rectangular horizontal flow type of grit chamber is considered for this design, as can be seen in figure 3.1. The general explanation of a grit chamber has already been told in section 2.2.

An instantaneous peak flow factor of 4 is considered for the design, which is an often used value. In an usual WWTP, this is done to cope with a so called wwf (wet (rain) weather flow) scenario. In this design, it has been done to be able to work in batches (in stead of continuous flow).

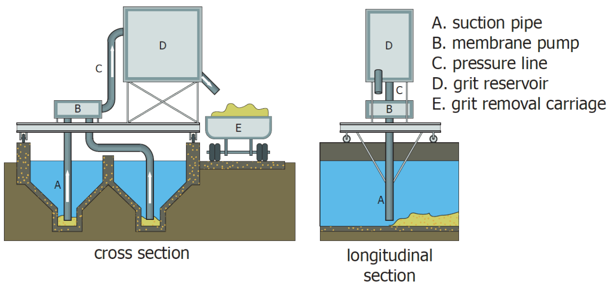

Figure 3.1: Rectangular horizontal grit chamber [1]

The derivation of the example dimensioning for the values mentioned in the previous paragraph can be found in appendix B.1, but the most important results are the HRT, volume and dimensions. Furthermore it is important to check that the surface loading stays below 40 $m^3/(m^2 \cdot h)$.

3.2 Flocculator

In order to enhance the flock-forming process, a pipe flocculator (often also referred to as coagulator) could be added before the primary sedimentation tank (or DAF unit). Both flocculating and coagulating induce a neutral charge of particles, allowing them to form flocks and thus have a higher sedimentation speed, being advantageous for the process. Typical and often used flocculants are Alum and Ferrix. The difference between coagulation and flocculation is that the latter uses high molecular weight polymeric materials as an addition [14]. The result of such a machine is a decreased sedimentation volume, at the costs of adding chemicals to the wastewater.

3.3 Drum Sieve: RBS500

As an alternative to the grit removal chamber, the RBS500 could be used. This is a self-cleaning automatic Rotating Drum Separator. Basically the working principle of the RBS500 is quite simple, as it is an accelerated natural process of depositing particles. Usually the settling process occurs with respect to Stokes' law, but this machine improves it. The gravitational force is increased by replacing it with centrifugal force. Furthermore, the number of obstacles which the wastewater encounters is increased by replacing the grid by a brush. This machine also removes about 50% of the COD contained in the wastewater [15]. The choice for this specific machine is due to the close cooperation between WTT and NieuweWeme.

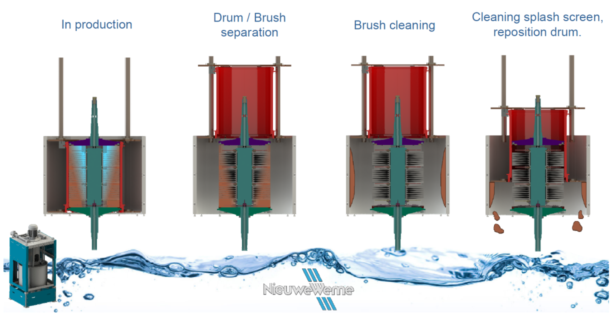

Figure 3.2: Automatic Rotating Drum Separator

3.4 Primary sedimentation tank

The primary sedimentation tank of preference will be a shallow, long and small rectangular tank equipped with a chain scraper, as can be seen in figure 3.3. This is very often used as primary sedimentation tank. The actual dimensioning can be found in appendix B.2.

The HRT is a very important design parameter. If this is too short, say ≤ 1.5 hours, the wastewater does not have sufficient time to settle and form sludge. On the other hand, it the HRT becomes ≥ 12 hours, bad odours start to develop and an expensive air treatment system is required. Therefore it should be between those boundaries.

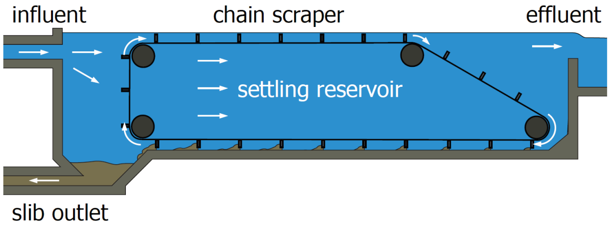

Figure 3.3: Rectangular primary sedimentation tank with chain scraper [1]

When designing the primary sedimentation tank in more detail, the left part of the bottom in figure 3.3 must be lower, to enhance the sludge removal process. The slope must be around 1:12. Based on the average HRT of 2 hours, in combination with the advised length:width and length:depth ratios, all dimensions can be found. Since the volume is designed using peak flow, it is important to check that the HRT using average flow does not exceed 12 hours for odour reasons. Furthermore the surface load must be checked to be below 2.5 $m^3/(m^2 \cdot h)$.

3.5 Dissolved Air Flotation

A Dissolved Air Flotation (DAF) device is an alternative to the primary sedimentation tank. As can be seen in figure 3.4, the Dissolved Air flotator is kind of an upside-down sedimentation tank. The DAF is a more modern alternative, as it is not limited by the sedimentation of particles, so the tank volume can be smaller. Due to the injection of compressed air (or other gases such as hydrogen, CO_2, nitrogen or ozone), pollutants such as suspended solids and fat, oil & grease (FOG) float to the surface, to be removed as sludge. The MicroGas Bubble Generator has very low OpEx costs and a high efficiency. It is able to cope with any flow larger than 1 m^3/h.

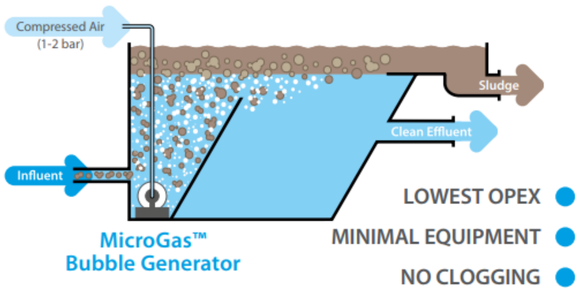

Figure 3.4: Dissolved Air Flotation working principle

3.6 Final clarifier

The function of a final clarifier is relatively similar to the primary sedimentation tank. Although the latter is usually rectangular, while a final clarifier is often circular. Still, it has the same 1:12 bottom slope to enhance sludge removal and its main goal is to remove sludge as thinly formed flocks from the wastewater as well.

A major problem in a final clarifier is the occurrence of bulking sludge. It is generated by massive growth of filamentous microorganisms that reduce the settleability of the sludge. Furthermore tank radius must not exceed five times the water depth. Due to the but due to the long and narrow design which should fit nicely in the container, this will not be a problem.

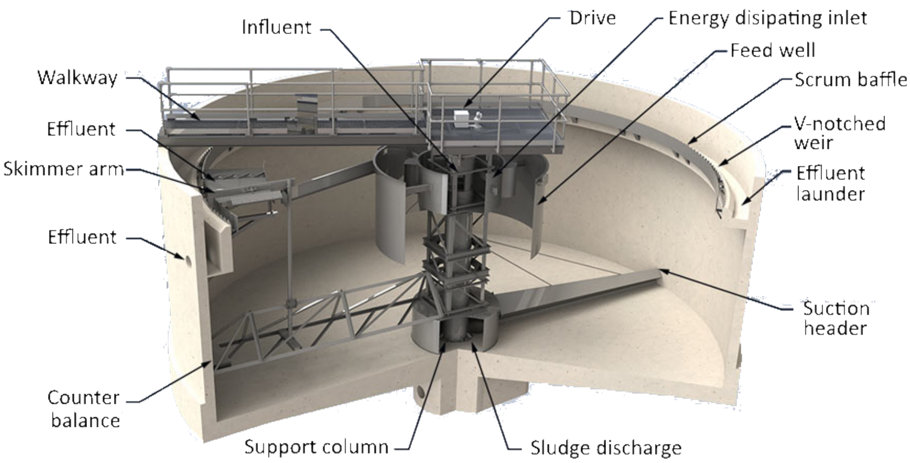

Figure 3.5: Working principle and overview of a circular final clarifier [2]

Using the Sludge Volume Loading Rate (SVLR) based on loads of experience with both round and rectangular tanks, together with the Sludge Volume Index (SVI) and sludge concentration in the aeration tank (X_{AT}), the surface loading could be calculated. Taking the recycle flow into consideration gives the bottom area. Lastly the volume (and indirect height) is determined by the HRT. These calculations can be seen in appendix B.6. Often the circular design is used for the final clarifier, the working principle can be seen in figure 3.5.

4 Design of the biological treatment

After the mechanical treatment, the wastewater enters the biological part of the wastewater treatment plant. Two main designs are considered: the more simple design consist of a trickling filter with optional post-treatment, while the more modern and more complex systems have a combination of three chambers with different environmental conditions. Both will be explained in the following sections and appendix B.3. Additionally a Membrane Bioreactor can be placed (or even replace).

4.1 Anaerobic, anoxic and aerobic tanks

The proposed design for the three tanks can be seen in the figure below. After the primary sedimentation, which has already been explained in section 3.4, an Upflow Anaerobic Sludge Blanket (UASB) will be used, see figure 4.6. This is an often used anaerobic system with a short HRT of less than 1 hour. In this tank water enters from the bottom and, as the name suggests, leaves at the top. If flows through a blanket of microorganisms where especially organic material (COD) is converted. Nearly no nitrogen/phosphorus is removed. COD is converted into methane by the anaerobic microorganisms and is caught at the top. The optimal operating temperature for these mesophilic organisms is 38°C [16], so a heating system is required to keep the influent at the correct temperature. Since there are loads of residual heat available in the plant, a heat exchanger would be the most logical solution.

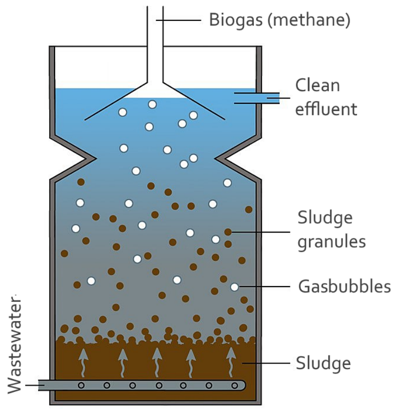

Figure 4.6: UASB overview [3]

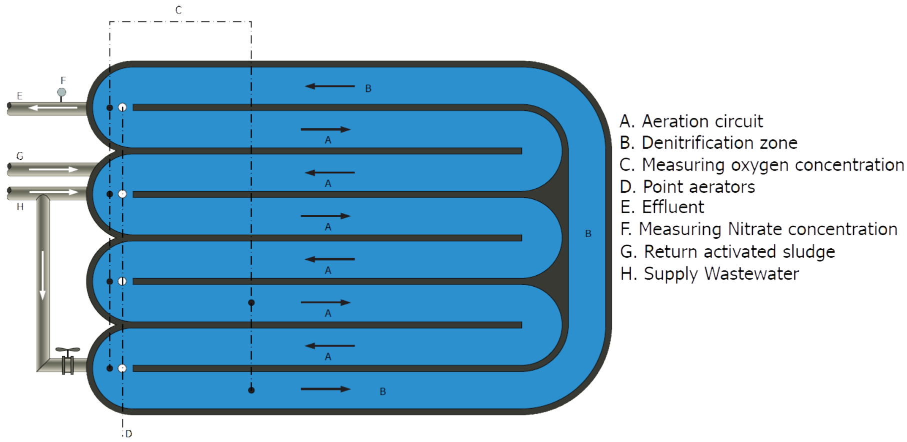

Figure 4.7: Anoxic and aerobic configuration: combined in a carousel [1]

The anoxic an aerobic part are combined into the configuration as seen in figure 4.7. Such a alternating configuration has a positive impact on the COD and nutrient removal, as well as sludge settleability [17]. The water is recycled a certain amount of times, for instance five times, before leaving through the effluent pipe. First of all, this recycling flow is important to ensure the return of activated sludge. This continues the process started in the anaerobic UASB tank, where PAO cells can grow and PO_4^{3-} is released. Now the cells will uptake phosphorus, so it could be removed together with the sludge. This recycle flow is an important parameter in COD removal. The second reason is the internal N-cycle, since the NO_2^- & NO_3^- cells generated by nitrification in the aerobic part of the carousel are transferred to the anoxic tank, where they can be denitrified to exhaust gases. Further explanation is given in the nitrogen-removal chapter 5. It is very important that NO_2^- and NO_3^- do not come in contact with the (anaerobic) UASB tank.

In the aerated part of the carousel, microscopical turbulence is necessary to enable the dissolved oxygen to be transferred to sludge flocks. There are several ways of aerating a tank, amongst which fine bubble aeration is by far the most often used and most efficient solution. Fine bubble aeration uses tiny holes on the bottom of the tank. Since the aeration efficiency is dependent upon the tank depth, this must be taken into consideration while designing.

4.2 Trickling filters in combination with RBS500

The working principle of a trickling filter can be seen in figure 4.9, as it works based on the water trickling down packing material with gravity. Both rectangular as well as circular design are possible. As indicated in the introduction, in high-loaded trickling filters, only BOD removal takes place. This is caused by the BOD loading, feeding the fast growing heterotrophic bacteria. When applying a sufficiently low BOD load, also in the bottom there is still sufficient oxygen and nitrogen-removing bacteria can grow on the packing material. Another solution must be found for the phosphorus-removal: this will probably be done by the RBS500 from Nieuwe Weme, which removes 80-90% of the phosphorus contained in cells.

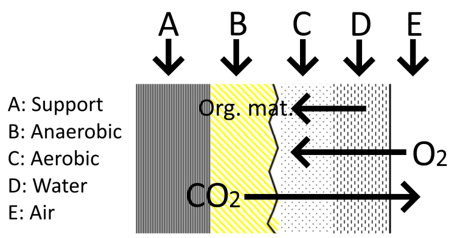

Figure 4.8: Schematic biofilm (aerobic+anaerobic) on the supportive packing material in a trickling filter

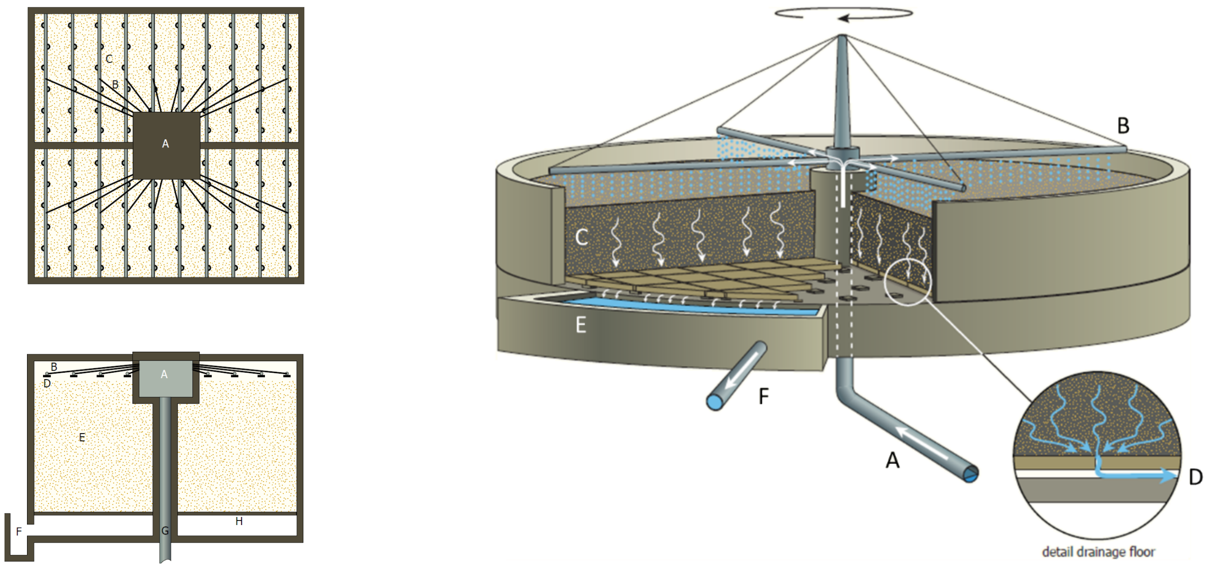

Figure 4.9: Two types of trickling filters: rectangular (left) and circular (right) [1]

An indication of how such packing material particle looks like with the (both anaerobic and aerobic) biofilm on it can be found in figure 4.8. More specific calculations are present in appendix B.5.

Typically the packing material of trickling filters is lava rock. This has a great specific surface area (m^2/m^3) and is relatively cheap. This is a great material, but it limits the height of trickling filters to ≈ 2.5 meters due to the surface load of the tanks, resulting in an enormous area. Recently another material is developed: artificial porous plastic. It has comparable material characteristics, but the specific weight is drastically lower. Therefore the trickling filters can be up to 10 meters high. Although in this case, due to the container-restriction, the trickling filter cannot be higher than 2 m.

The airflow through the filter bed is driven by ΔT of the sewage and outside temperature. If the outside temperature is higher, a downward flux occurs. If the sewage is hotter, vice versa.

Since the influent wastewater demands a very high BOD-removal as well as nitrogen oxidation, the choice has been made to have a high-loaded trickling filter, which does not contribute to the nitrogen removal. Furthermore a low-loaded trickling filter is placed in series to ensure nitrogen-removal, although it remains questionable whether full nitrification could occur [18, 19]. The total setup can be found in figure 4.10. There are two types of recirculation: the effluent is added to the outlet of the primary sedimentation, in order to further decrease BOD in the trickling filter. Furthermore the humic sludge is added to the inlet of the primary sedimentation tank, so sludge can sediment and be discharged.

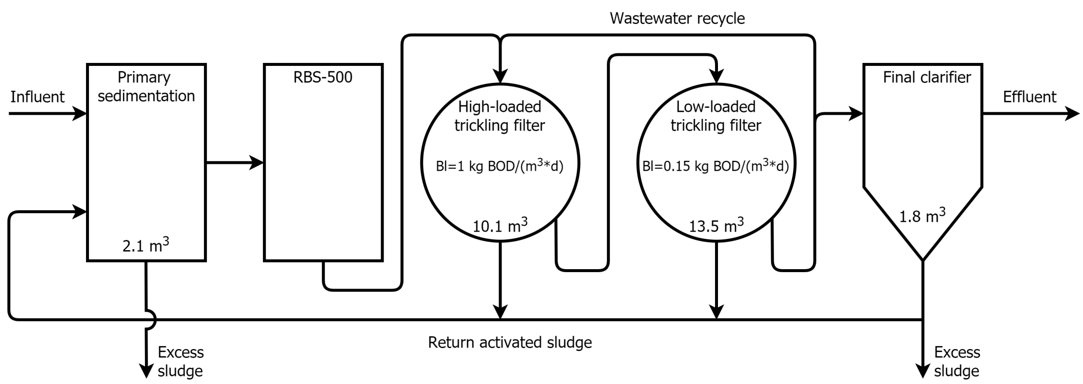

Figure 4.10: Configuration of the second solution for biological treatment: two trickling filters

* The shown volumes are based on the average influent characteristics of table 2.2 and 1500 m^3/a

4.3 Membrane Bioreactor

A Membrane Bioreactor (MBR) is often used in WWTPs. It could operate as standalone biological reactor, or in series with other biological components, such as the three tanks of section 4.1. As the name suggests, A MBR is a combination of (an)aerobic biological treatment and membrane filtration. These membranes use ultrafiltration and have pore sizes as small as 0.01 μm, their working principle can be found in figure 4.12, while a possible setup is shown in figure 4.11.

The advantage of an MBR over a final clarifier is that no disinfection is required after, this already happened in the membrane. If a final clarifier is used, this means another disinfection step such as ozonization, chlorination or an active carbon filter is required to kill bacteria and pathogens.

If the MBR operates as standalone biological reactor, the high-strength anaerobic MBR might be interesting, as it operates as its best at COD values larger than 7500 mg/L and is able to cope with high concentrations of dissolved solids. Removal percentages of 98% and beyond are easily reached. Due to the high specific surface area, the footprint of such devices is minimal. Since it operates in an anaerobic environment, also biogas is generated.

In case the MBR is placed after for instance the three tanks of section 4.1, the effluent is already much cleaner (e.g. lower COD values). Therefore the Crossflow MBR might be more suitable, as it performs optimally at COD $\leq$ 7000 mg/L, while still being able to handle flows up to 100 m^3/d.

Optionally a Reverse Osmosis (RO) device could be added using the clean effluent of the MBR. These devices use nanofiltration in stead of ultrafiltration, meaning that the pore sizes are even smaller at 0.0001 μm. The effluent of a RO device is essentially pure water, since all organics, viruses, bacteria, ions and some minerals are removed.

In order to clean the membranes, it is required to backwash the system once every period of time. For large plants, this must be done every hour or so. Since the backwash costs less than a minute, the availability for these fully-automated and self-cleaning MBR's is high.

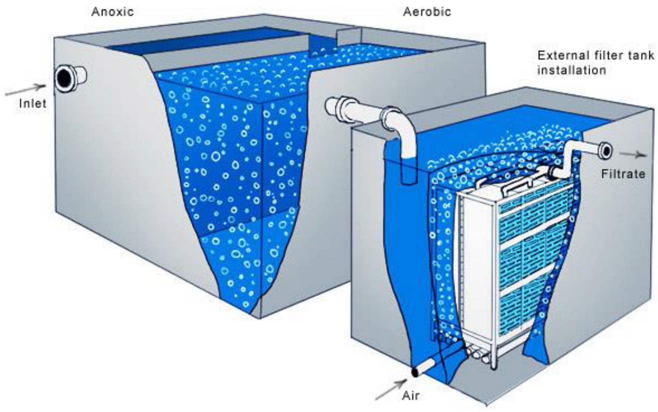

Figure 4.11: Possible setup of a Membrane Bioreactor

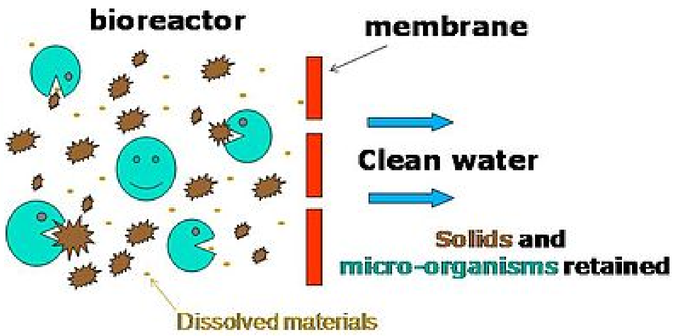

Figure 4.12: Working principle of a Membrane Bioreactor

5 Nitrogen removal: nitrification and denitrification

Nitrogen removal can occur in both option 1 (three tanks) and 2 (trickling filter) from sections 4.1 and 4.2, since this is only dependent on oxygen supply, C-source supply and sludge age.

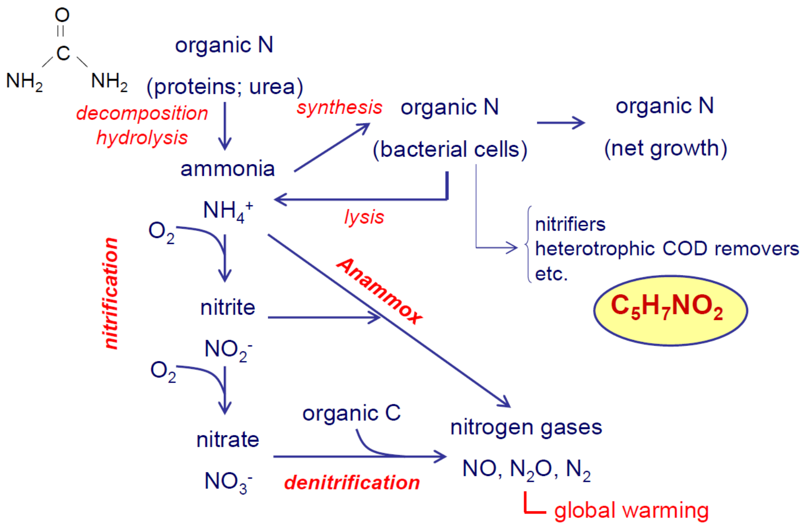

Figure 5.1: Schematic overview of (ammonium-)nitrogen removal [4]

The traditional removal of nitrogen occurs in two phases, as can be seen in figure 5.1. The first step is nitrification, where ammonium (NH_4^+) gets converted into nitrite (NO_2^-) and nitrate (NO_3^-) while consuming oxygen, this happens in an aerobic environment. The seconds step is denitrification, where the products of nitrification (NO_2^- and NO_3^-) are converted into nitrogen gases such as NO, N_2O and N_2 using an (in)organic C-source, e.g. methanol or BOD. The processes therefore are limited on oxygen and C-source supply. A more energy efficient "shortcut" in the natural nitrogen process is called Anammox, and does nitrification and denitrification in one step. Although it is being used more and more nowadays, it is still in development phase. All of the above is clearly indicated in figure 5.1.

5.1 Nitrification

Nitrification occurs in 2 steps: the first step corresponds to the top equation of equation 5.1 and is performed by Nitrosomonas bacteria, which converts ammonium into nitrite. The second step is performed by the Nitrobacter bacteria, which converts nitrite into nitrate, as seen in the middle equation. When these two equations are add up, the overall equation can be seen in the bottom equation.

$$\begin{aligned} 2NH_4^+ + 3O_2 &\rightarrow 2NO_2^- + 2H_2O + 4H^+ \\ 2NO_2^- + O_2 &\rightarrow 2NO_3^- \\ 2NH_4^+ + 4O_2 &\rightarrow 2NO_3^- + 2H_2O + 4H^+ \end{aligned} \tag{5.1}$$

Now, using the molar ratio's of ammonium (NH_4) and oxygen (O_2), together with their molar weights, the total oxygen demand (g) per gram of ammonium-nitrogen (denoted as NH_4-N) can be calculated and turns out to be 4.57 g O_2/g NH_4. However, in reality this value lies around 4.33, because part of the nitrogen is incorporated into the bacteria cells.

5.2 Denitrification

For denitrification a similar calculation could be done. In stead of molecular oxygen, O_2, now a C-source is required. This could be BOD from the wastewater (organic matter in wastewater is generally denoted as $C_{10}H_{19}O_3N$ [20]), as seen in the top equation of equation 5.2. The minimum amount of BOD compared to nitrogen must be at least around ratio 4:1, but preferably 6:1, to ensure sufficient denitrification. Although depending on exact influent characteristics, often this is insufficient, and an additional non-organic C-source such as methanol or acetate must be added. These reactions are given in the middle and bottom equation of equation 5.2.

$$\begin{aligned} C_{10}H_{10}O_3^-N + 10NO_3 &\rightarrow 5N_2 + 10CO_2 + 3H_2O + NH_3 + 10OH^- && \text{(with BOD)} \\ 5CH_3OH + 6NO_3^- &\rightarrow 3N_2 + 05CO_2 + 7H_2O + 6OH^- && \text{(with methanol)} \\ 3CH_3COOH + 8NO_3^- &\rightarrow 4N_2 + 10CO_2 + 6H_2O + 8OH^- && \text{(with acetate)} \end{aligned} \tag{5.2}$$

Depending on the BOD load, the remainder of NO_2^- and NO_3^- can be calculated, and using stoichiometry the amount of additional methanol or acetate required could be calculated. Every step of reduction can be seen in figure 5.2 below.

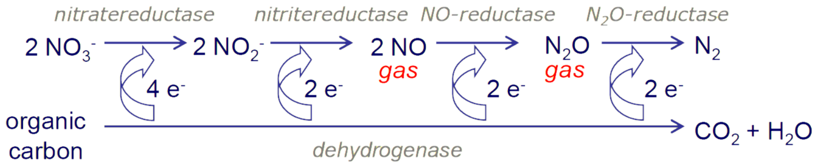

Figure 5.2: General overview of the process of denitrification

Lastly, the acidification of the wastewater must be considered. As can be seen from equation 5.1, one mole of NH_4^+-N creates two moles H^+. However, as can be derived from equation 5.2, one mole of NH_4^+-N also created one mole of OH^-, which restores one H^+. Still, some acidification takes place due to the H^+. This must be solved by either adding a alkaline component or making a buffer. In the discussion, a bit more in-depth explanation is given.

5.3 Oxygen demand

Passive molecular oxygen (O_2) diffusion occurs according to Fick's law, however this is way too slow for this wastewater treatment process. Therefore, an active oxygen supply in the form of fine bubble aeration has been chosen, as explained before. Usually O_2 supply is the main energy consumption of WWTP's, therefore the corresponding oxygen and energy demand has been calculated. The total oxygen demand can be derived from equation 5.3

$$\mathrm{OD} = \mathrm{O_e} + \mathrm{O_s} + \mathrm{O_n} + \mathrm{O_o} + \mathrm{O_z} \tag{5.3}$$

This formula shows five contributions to the oxygen demand, although the last two can be neglected. The oxygen demand of a WWTP can be split into endogenous respiration (oxidation of the own cellular mass of microorganisms), substrate respiration (oxidation of the cellular mass of e.g. BOD) and (de)nitrification as is explained in this section. A more extensive explanation and calculation can be found in appendix B.7, as well as the costs estimation, which will be further discussed in chapter 7.

6 Phosphorus removal: biological and/or chemical

Phosphorus is becoming a scarcer material lately. The enormous mines in China are getting smaller and therefore recycling it is important. There are several ways to remove phosphorus from wastewater, which can be subdivided into biological and chemical. The biological removal capacity is limited by the COD supply and PAO bacteria involved. Chemical removal is not limited, but produces large amounts of additional sludge. Besides, the costs for chemicals are significant. Since the effluent demands are quite strict, the preferred treatment is to do as much as possible with biological treatment, and supplement it to reach effluent demands with chemical treatment.

6.1 Biological removal using PAO's

Biological treatment is based on PAO-bacteria cells, which take up phosphate under aerobic conditions, while releasing phosphate under anaerobic conditions. These cells are able to store up to 300 mg P per gram VSS, compared to 20 mg P for regular cells. The COD which is present in the wastewater converts into acetate, which is used by the PAO cell to release phosphate under anaerobic conditions. Under aerobic conditions, exactly the opposite happens, and phosphate is accumulated in the cell, as can be seen in figure 6.1. The PAO cells are naturally available in the wastewater, since it just left the anaerobic fermentation tank. The phosphate will leave in the cells with the sludge. This method is limited by the amount of COD. A schematic representation of the phosphorus concentration in the treatment plant can be seen in figure 6.2.

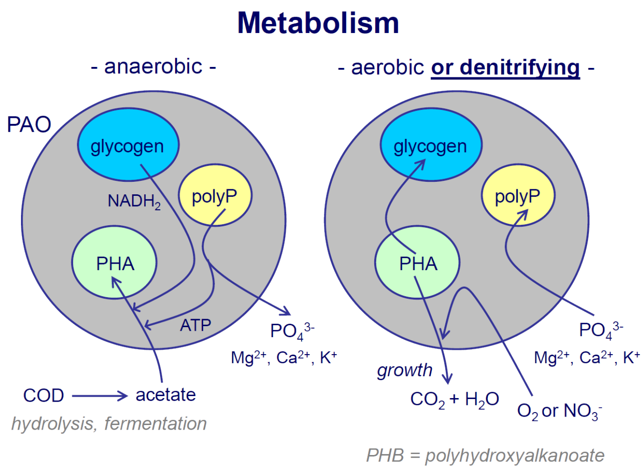

Figure 6.1: Working principle of biological phosphorus removal using PAO's [5]

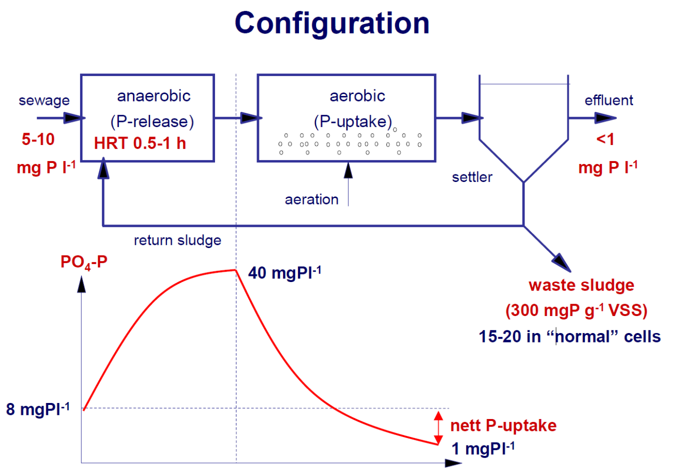

Figure 6.2: Schematic flowchart of biological phosphorus removal [5]

There are some environmental requirements for PAO bacteria to work, amongst which the following are the most critical for an industrial wastewater plant. Wastewater characteristics must have the following molar based ratios: $Mg/P \geq 0.70$, $K/P \geq 0.50$ and $Ca/P \geq 0.25$. Furthermore in the aerobic (uptake) phase the DO concentration must be ≥ 1.0 g/m^3.

6.2 Chemical removal using additives

Aside from the biological PAO removal, there are several other chemical ways to remove phosphorus, since in such polluted wastewater, biological removal of phosphorus is (often) insufficient. One way is to add lime ($CA(OH)_2$), Aluminium (Al^{3+}) or Iron (Fe^{3+}), creating precipitate which can be removed. The downsides are the high costs involved with the chemicals and the enormous sludge generation. Another method used regularly is the addition of Magnesium [21]. The result now is still sludge, although now called Struvite [22]. This Struvite is a high quality fertiliser, which can be sold in stead of being discharged. Struvite is made of magnesium, phosphorus (phosphate), nitrogen (ammonium) and water, and reacts according to equation 6.1. The precipitate can easily be discharged. Small sized Struvite reactors already have been developed [22], even as small as 0.2 m^3/d. A possible layout can be seen in figure 6.3.

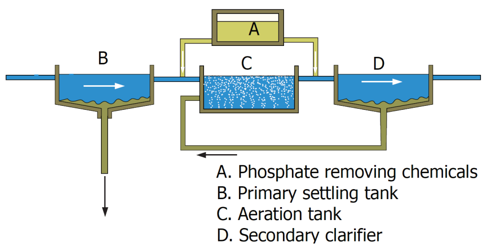

Figure 6.3: Schematic flowchart/layout of chemical phosphorus removal [1]

$$Mg^{2+} + NH_4^+ + PO_4^{3-} + 6H_2O \rightarrow Mg(NH_4)PO_4 \cdot 6H_2O \qquad (6.1)$$

7 Costs analysis

The costs for a leachate purifying container can be divided into Capital Expenditures (CapEx) and Operating Expenses (OpEx). The first mentioned stands for the purchase of the container with all machinery and components belonging to it, but also the salary for installation and transportation. The latter are day-to-day expenses to keep the container operating; such as heating, aerating and the costs for chemicals. As guideline for the costs, the values of table 7.1 have been used. These costs change drastically for different inflow characteristics.

Quantity	Annual flow	BOD	COD	N-total	P-total
Value	2190 m3/a	1690 mg O_2/L	12750 mg O_2/L	236 mg N/L	126 mg P/L

Table 7.1: Design inflow characteristics for both capital and operational expenditures

* In reality all parameters turn out to be much higher for the big installations

7.1 Capital Expenditure (CAPEX)

The capital investment costs can be subdivided into several systems: the control system, the electrical system, the water system (including all sensors) and the tanks (including installation and piping). A summarised indication of the capital costs for all categories mentioned above are given below in table 7.2. These are only the group costs. A way more extensive table is given in appendix C on page F, where the groups have been divided into several parts. Note that for this small amount of water, it fits in a 40ft. container. For larger annual flows, big containers will be required, increasing the capital expenditures exponentially. Therefore, large installations rather have SKID containers containing the machinery and separate delivery of reaction tanks.

Category	Tanks and piping	Water	Electrical	Control	**Total**
Costs	€50,000	€30,000	€8,000	€28,000	**€116,000**
Percentage	43	26	7	24	**100**

Table 7.2: Summarised indication of Capital Expenditures per category

7.2 Operational Expenditure (OPEX)

Operating costs can be divided in a few factors, amongst which the following are the most essential.

- Salary of the plant operators
- Price of the excess sludge disposal
- WTT services; for performance guarantees
- Aerating the aeration chamber, so the aerobic part of the carousel
- Heating, pumping and other energy costs of the wastewater cycle through the WWTP
- Chemicals required during the process

The salary is heavily dependent on the amount of time which is necessary on for instance maintenance, on which big assumptions have to be done. As average salary €2500 has been taken, with an occupation of half a day (4 hours) per week, this comes down to €3000 for salaries in one year operating.

Another major operational cost is the disposal of excess sludge, which is heavily dependent on the annual volume flow. For the values described in table 7.1, while assuming 0.25 weight-percent of excess sludge, being too contained to return to the AD tunnels. With a dry solids content of the sludge of 20% and a sludge disposal price of €30 per tonne, this comes down to €820 per operating year.

The costs for the WTT services are very hard to determine, since they are heavily dependent on the specific contract details. Therefore this will be left out. Besides, these are optional costs for the customer.

Although depending on the exact composition and other factors, aeration is commonly the biggest contributor to the operational expenditures. The oxygen demand is dependent on several factors, amongst which nitrogen removal, endogenous respiration, and substrate respiration are the most important. A more extensive calculation can be found in appendix B.7, but the result is an oxygen demand of 8286 kg O_2/year. With a fine bubble aeration efficiency of approximately 4 kg O_2/kWh and the average non-household energy price, this comes down to only €270 per year, which can be explained by the low nitrogen concentration.

Heating, pumping and other electricity costs besides aeration of the carousel must be estimated using the components given in the CapEx section and their corresponding energy consumption. The heat exchanger is expected to require 500 W, while all the pumps for both water and air displacement are assumed to require somewhere around 2 kW.

This means an electricity demand of 60 kWh/d, with corresponding costs of approximately €2900 per year using the same average non-household electricity price as before.

Chemicals required for the functioning of the process, for instance to supplement the biological phosphorus removal, pH regulation and as external C-source, required for denitrification in case insufficient organic-C is present. This is heavily dependent on the inflow characteristics, but the attached Excel-file in combination with leachate tests could help with a more accurate estimation. For now it is approached to be around €1000 per year.

Summarised, this counts up to the following operational expenditure distribution:

Category	Salary	Sludge disposal	Aeration	Heating/pumping	Chemicals	**Total**
Costs	€3000	€820	€270*	€2900	€1000	**€7990**
Percentage	38	10	3*	36	13	**100**

Table 7.3: Summarised indication of Operational Expenditures per category

* this increases very rapidly with (more common) higher nitrogen concentrations

8 Discussion

When taking a further look into the second biological option, as seen in section 4.2, there are serious concerns about the (very) high nitrogen and phosphorus concentrations in the influent. Trickling filters have no phosphorus removal capacity and nearly no (de)nitrification capacity, even the low-loaded trickling filters [18]. Besides, the phosphorus removed by the rotating drum separator (RBS-500), is only the in-cell phosphorus [15]. This means another solution must be found for dissolved phosphorus (e.g. PO_4^{3-}). Lastly, the volumes occupied by the second solution is larger, which might cause problems fitting it in a container design. Overall, alternating tanks are a way better solution.

In case of any pH-disturbances, this can be solved by a chemical dosing system. Such a systems exists of two chemical reagent storages for either acidification or becoming more bases. Furthermore a dosing pump and control valve are required. Acidification could be solved by the addition of NaOH, while a more bases solution requires for instance H_2SO_4 [23].

The annual volume flow of around 2000 m^3/a which has been used for the example calculation in this report, turns out to be on the low side, since this is coming from small installations. In reality the volume flow turns out to be more in the range of 15,000 m^3/a, which causes the installation to be much larger than a single container-design.

Another point worth noticing is the fact that the total volume flow can be divided into two flows: the leachate and the condensate. The latter is way less polluted than the first one, which means the average BOD/COD values will probably drop, resulting in smaller tanks. In addition, the stronger wastewater could be used to provide sufficient C-source for denitrification, so no external C-source such as methanol is required.

The anaerobic stage in the WWTP might not be necessary for the sake of PAO-forming, since the leachate is already in anaerobic conditions, meaning that PAO's are readily present in the wastewater. Although still lots of removal take place in this stage. Further testing is required to make sure the exact removal requirements.

9 Conclusion and recommendations

The aim of this internship research was to investigate feasibility of a wastewater treatment plant for the anaerobic digestion tunnels of WTT. Although exact completion of an integrated WWTP is very dependent on specific influent characteristics, legislation, wages for installation, discharge costs and other details, it seems to be worth the effort to install such a treatment installation for plants with sufficient leachate surplus (i.e. $\geq$ 5000 m^3/a). In case less water has to be discharged, it is probably be more favourable to discharge the water to the nearest WWTP.

Further research is required to make the final design, for instance to check the actual removal rates, possibly in cooperation with companies already in the WWTP market, such as Nijhuis Industries or Colubris CleanTech.

In the current situation, everything is based on the strong wastewater characteristics, probably resulting in an overkill design. To improve the accuracy of the model, the inflow volume should be split into two flows. This directly means further testing is required for precise measurements.

A design for the case of 2000 m^3/a mentioned throughout this report has been made and consists of several stages. A nice way to map the flows, the piping and instrumentation (e.g. sensors, pumps, valves, etc.) is using a so called PNID, which can be seen in appendix D. The attached Excel sheet "*Purification Calculation Sheet.xlsx*" can be used in order to calculate dimensions of all components, oxygen and chemical requirements, operational expenditures and other information, all dependent on influent characteristics.

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Appendices

A List of most important assumptions

Assumptions are essential for nearly any research, therefore the assumptions done for this report are listed per category below. Note that parameter values are not noted if they are already explained in the corresponding paragraph.

General assumptions are the following:

- The water surplus of AD installations is much more than the water discharge of composting installations, and therefore only installations where AD tunnels are installed will be taken into consideration for this report.
- The Sludge Loading Rate, also called F-M-ratio (Food-to-Mass-ratio), considered is now 0.3 kg BOD/(kg ds·d). To improve effluent characteristics, this number could be decreased at the cost of a larger installation.

Assumptions with regards to *influent water characteristics* are the following:

- All sample data is acquired under similar conditions with the same testing methods.
- The average value of all data has been taken as average to use for the design. There might be critical changes in design if the values differentiate too heavily.
- The current peak factor used for the design is 4. That might be too high, since large deviations do not occur due to for instance rainfall, so 2 will be sufficient as well.
- The Sludge Volume Index (SVI) of the created sludge is assumed to be 140 mL/g, which corresponds to quite bad settleablity. Measurements should be performed to guarantee this number, which could differ in reality.

Assumptions with regards to *effluent water demands* are the following:

- Since there are rules for WWTP's of less and more than 10,000 PE, this design applies for smaller than 10,000, although it is not completely clear whether other laws apply to industrial WWTP's.
- The regulations for the amount of chlorine, salts and acids (pH) in the wastewater have not been taken into account.

Assumptions with regards to *dimensioning* are the following:

- The sludge concentration of the effluent is equal to $X_E = 0$. This is only for the mass balance, to calculate the sludge concentration in the return sludge X_R.
- The sludge concentration in the carousel/aeration tank is assumed to be 4 g/L, which lies in the middle of the recommended interval of 3-5 g/L.
- 30% of the carousel volume is an anoxic environment, while the remaining 70% is aerated. This corresponds to existing installations.
- The maximum (activated) return sludge concentration is based on an empirical relation with the Sludge Volume Index mentioned earlier.

Assumptions with regards to *nitrogen and phosphorus removal* are the following:

- The sludge age must be at least 20 days for sufficient (de)nitrification possibilities.
- Currently it has not been considered that there might be insufficient organic C-source (in the form of COD) to denitrify all ammonia/nitrogen present in the wastewater. If it turns out there is insufficient organic C-source, another external source such as methanol must be added.
- BOD exists of a soluble and non-soluble (or dissolved) part. For the easy of calculation, it has been assumed that 75% of the BOD is non-soluble and 25% is soluble.
- While calculating the PAO biomass produced, all cell debris is neglected.

Assumptions with regards to the *oxygen demand* are the following:

- The oxygen demand for both the discharge of dissolved oxygen as well as due to quickly oxidizable components is assumed to be zero, which is a reasonable assumption for normal conditions.
- Both the factors for endogenous and substrate respiration have been based on previous experience, their values are 0.1 kg O_2/(kg ds·d) and 0.9 kg O_2/kg BOD respectively.
- The efficiency of fine bubble aeration have been assumed from experience of suppliers, which means they can supply approximately 4 kg O_2 per kWh.
- The average energy price for non-households in Europe has been taken for the calculations.

B Extensive dimensioning

Important notice: the results of these calculations, as can be seen in the tables at the bottom of each section, have been calculated using the "incorrect" volume flow. The actual volume flow turns out to be more in the range of 15,000 m^3/a, rather than the 2000 m^3/a used.

B.1 Grit chamber

For the hydraulic retention time τ, only the theoretical depth d and the minimal vertical velocity for the smallest particles to settle $v_{min,settling}$.

$$\tau_{grit} = d_{grit}/v_{min,settling} \tag{B.1}$$

The volume and length of the grit chamber are both dependent on the retention time τ_{grit}, as well as horizontal inflow velocity v_{hor} and the instantaneous peak flow Q_{max}.

$$V_{grit} = \frac{Q_{max}}{3600} \cdot \tau_{grit} \tag{B.2}$$

$$L_{grit} = v_{hor} \cdot \tau_{grit} \tag{B.3}$$

The frontal area easily follows by dividing the volume by the length. Furthermore, the width can be calculated by dividing this frontal area by the theoretical depth, mentioned earlier.

$$A_{grit} = \frac{V_{grit}}{L_{grit}} \quad \text{and} \quad W_{grit} = \frac{A_{grit}}{d_{grit}} \tag{B.4}$$

Now all dimensions are known (still an optional freeboard could be added to the depth), so the surface loading $V_{0,grit}$ and length-width ratio can be calculated. In order to be on the safe side for the surface loading (and to be able to cope with even higher instantaneous loads), the width will be multiplied with a safety factor of 2.

$$V_{0,grit} = \frac{Q_{max}}{2 \cdot W_{grit} \cdot L_{grit}} \tag{B.5}$$

The final input and output characteristics can be seen in the tables below.

What?	d_{grit}	$v_{min,settling}$	$v_{hor,max}$	v_{hor}	Q_{max}	$V_{0,grit,max}$
Input	0.05 m	0.012 m/s	0.30 m/s	0.10 m/s	1 m^3/h	40 $m^3/(m^2 \cdot h)$
How?	assumed	experience	experience	assumed	assumed	experience

What?	τ_{grit}	V_{grit}	A_{grit}	L_{grit}	W_{grit}	$V_{0,grit}$	LW-ratio
Output	8 s	5.0 L	498 cm^2	83 cm	6 cm	21.6 $m^3/(m^2 \cdot h)$	13.8

B.2 Rectangular primary sedimentation tank

The volume of the primary sedimentation tank V_{pri} can be based on the maximum instataneous peak flow Q_{max} and the average hydraulic retention time of the primary sedimentation tank τ_{pri}.

$$V_{pri} = Q_{max} \cdot \tau_{pri,peakflow} \tag{B.6}$$

Guidelines based on experience concerning length:width:depth ratio's are considered for the design of the rectangular primary sedimentation tank. Width:length must be around 1:5-1:6 and depth:length must be around 1:20. Therefore, if depth is considered unity, the length is 20*depth and the width is approximately 3.5*depth. Since the volume is known, the depth can be calculated, as it is known that:

$$V_{pri} = L_{pri} \cdot W_{pri} \cdot d_{pri} = 20 \cdot d_{pri} \cdot 3.5 \cdot d_{pri} \cdot d_{pri} = 70 \cdot d_{pri}^3 \Rightarrow d_{pri} = \sqrt[3]{\frac{V_{pri}}{70}} \tag{B.7}$$

As the depth and ratio's are known, all dimensions can be calculated. An optional freeboard could be added to the depth. Furthermore the design of a small slope (about 1:12) must be designed for a good sludge removal. The surface load $V_{0,pri}$ can be calculated using the bottom area, according to the formula

$$V_{0,pri} = \frac{Q_{max}}{A_{bottom}} = \frac{Q_{max}}{W_{pri} \cdot L_{pri}} \tag{B.8}$$

Since the design is made to cope with the peak flow, a check must be done in order to be sure that the hydraulic retention time is between the limits of $1 \leq \text{HRT} \leq 12$ hours. The lower limit is based on the settling of particles, the upper limit is based on the prevention of bad odours.

$$\tau_{pri,avg} = \frac{W \cdot L \cdot d}{Q_{avg}} \tag{B.9}$$

The final input and output characteristics can be seen in the tables below.

What?	$\tau_{pri,peakflow}$	Q_{max}	$V_{0,pri,max}$
Input	2 hours	1 m^3/h	2.5 $m^3/(m^2{\cdot}h)$
How?	assumed	assumed	experience

What?	$\tau_{pri,avg}$	V_{pri}	L_{pri}	W_{pri}	d_{pri}	$V_{0,pri}$	Bottom slope
Output	8 hours	2.4 m^3	6 m	1 m	0.4 m	0.15 $m^3/(m^2{\cdot}h)$	1:12

B.3 Biological treatment: general calculations

Before even thinking about the design, first the influent characteristics are required to make a design. In section 2.1.2, the average of 30+ samples have been taken, which resulted in the influent characteristics of table 2.2. The most important values are duplicated into table B.1 below.

Quantity	**BOD_5**	**COD**	**SS**	**P-total**	**N-total**
Value	**1690** mg O_2/L	**15899** mg O_2/L	**834** mg SS/L	**119** mg P/L	**241** mg N/L

Table B.1: Most important influent characteristics

Using the average BOD concentration per L of wastewater, combined with the total volume flow per day Q_{avg}, the total BOD supply per day, B_D (kg BOD/d) can be calculated. Exactly the same can be done for the nitrogen supply Nd.

$$B_D = \frac{BOD_5 \cdot Q_{avg}}{1000} \quad \text{and} \quad N_D = \frac{\text{N-total} \cdot Q_{avg}}{1000} \tag{B.10}$$

B.4 Three alternating tanks: anaerobic, anoxic and aerobic

When designing the three alternating tanks, consisting of an anaerobic tank, an anoxic tank and an aerobic tank, several rules of thumb and guidelines must be considered. A few important ones are described below.

According to the theory, the maximum return sludge concentration $X_{R,max}$ is defined as 1000/SVI. Although empirical research shows that a more realistic value is 1200. In formula form this looks as:

$$X_{R,max} = 1200/\text{SVI} \tag{B.11}$$

In which SVI stands for Sludge Volume Index, which is an indication for the settleability of the sludge. It is defined as the sedimentation volume occupied by one gram of activated sludge, with the unit mL/g. For this design, a SVI of 140 mL/g is assumed, which is considered to be (very) poorly settling sludge. Now using equation B.11, the maximum return sludge concentration $X_{R,max}$ can be calculated.

The storage of total activated sludge must be below 30% in order to have an efficient process. If this becomes more, modifications must be made.

The sludge concentration X_{AT} in the aeration tank almost always lies somewhere between 2 g/L and 5 g/L. The most often chosen value is 4 g/L. Introducing the recycle flow ratio R, the return sludge volume flow Q_R can be calculated. Using the maximum concentration of equation B.11 and the aeration tank concentration X_{AT}, the minimum return sludge recycle flow ratio can be calculated as well:

$$Q_R = R \cdot Q_{avg} \quad \text{with} \quad R_{min} = \frac{X_{AT}}{X_{R,max} - X_{AT}} \tag{B.12}$$

To calculate the resulting return sludge concentration X_R, the effluent concentration X_E must be assumed to be zero, which is a reasonable assumption. It can be calculated using equation B.13, which is based on the mass balance around the second clarifier.

$$X_R = \frac{(Q_{avg} + Q_R) \cdot X_{AT} - Q_E \cdot X_E}{Q_R} \tag{B.13}$$

Now an important design parameter is introduced, the sludge loading rate (often referred to as F/M-ratio, or Food-to-Mass ratio), from now on simply Bx. Its unit is kg BOD/(kg ds· d). It is so important since it determines the effluent characteristics, as well as the HRT and whether nitrification takes place. It is also a bit temperature dependent.

Using the Food-to-Mass ratio Bx, the total volume of the aeration tank V_{AT} can be calculated, according to the following:

$$V_{AT} = \frac{B_\mathrm{D}}{B_\mathrm{X} \cdot X_{AT}} \tag{B.14}$$

Now every ingredient is there to calculate the Sludge Age θ (also called Sludge Retention Time or SRT) and HRT of the wastewater.

$$\mathrm{SRT} = \frac{V_{AT} \cdot X_{AT}}{Q_R \cdot X_R} \quad \text{and} \quad \mathrm{HRT}_{AT} = \frac{V_{AT}}{24 \cdot Q_{avg}} \tag{B.15}$$

Sludge is created in the process. The amount of sludge growth can be calculated and is dependent on the sludge loading rate to a large extent. In general dissolved contaminates generate more sludge growth than undissolved contaminates. The sludge created by chemical phosphorus removal is significant and will be calculated separately. Empirical values for the sludge growth Y (kg ds/kg BOD) depending ion the sludge loading rate Bx are present. In the case that Bx=0.30 kg BOD/(kg ds·d) is chosen, the sludge growth rate Y is 0.87 kg ds/kg BOD. Since the BOD supplied $\mathrm{B_D}$ is 10.14 kg BOD/d, this means 8.8 kg of biomass grows per day, which must be removed.

What?	SVI	R	Q_{avg}	$\mathrm{B_D}$
Input	140 mL/g	1.50	0.25 m^3/h	10.14 kg BOD/d
How?	assumed	chosen	calculated	calculated
What?	Q_E	X_E	X_{AT}	Bx
Input	0.25 m^3/h	0 g/L	4 g/L	0.30 kg BOD/(kg ds·d)
How?	= inflow	assumed	assumed	chosen

What?	R_{min}	Q_R	$X_{R,max}$	X_R	V_{AT}
Output	0.875	0.375 m^3/h	8.57 kg ds/m^3	6.67 kg ds/m^3	8.45 m^3
What?	SRT	HRT	V_0	Vs	
Output	13.5 d	1.41 d	0.71 m^3/(m^2·h)	400 L/(M^2·h)	

B.5 A high- and low-loaded trickling filter

Another possibility might be the trickling filter, which is an older and simpler manner to treat wastewater. The biggest disadvantage is that it is not able to remove phosphorus. This could be solved by RBS500 from the company Nieuwe Weme, a well known company to WTT. Although, further testing should be done on the actual phosphorus removal. A minimum height h of 1.5 m is considered, in order to ensure sufficient cleaning. Since the container is approximately 2.3 meters in height, a tank height h_{trick} of 2 meter is chosen. The way of calculating does not differ for low- or high-loaded filters. The starting point is calculating the volume V_{trick} by the BOD-supply $\mathrm{B_D}$ and acceptable BOD-load $\mathrm{B_L}$ for the type of filter.

$$V_{trick} = \mathrm{B_D}/\mathrm{B_L} \tag{B.16}$$

Using the given height h_{trick} and the volume V_{trick}, the required area A_{trick} can be calculated. An important factor to take into consideration is the maximum hydraulic load $V_{0,trick}$, which is dependent on the type of filter. In combination with the maximum volume flow rate Q_{max}, this results in the minimum area A_{min}. In practice, the real A will always be larger due to the limited height.

$$A_{trick} = \frac{V_{trick}}{h_{trick}} \quad \text{and} \quad A_{min} = \frac{Q_{max}}{V_{0,trick}} \tag{B.17}$$

This area can be filled by either a circular or rectangular trickling filter, it does not make a big difference. Due to the limited space, rectangular filters are more efficient. All input and output parameters (for both the low-loaded and high-loaded filter) are given in the upcoming tables.

What?	$\mathrm{B_D}$	h_{trick}	$\mathrm{B}_{L,low}$	$\mathrm{B}_{L,high}$	$V_{0,trick,low}$	$V_{0,trick,high}$	Q_{max}
Input	10.14 kg/d	2 m	0.15 kg/(m^3·d)	1.0 kg/(m^3·d)	0.20 m^3/(m^2·h)	1.80 m^3/(m^2·h)	1 m^3
How?	calculated	limited	experience	experience	experience	experience	assumed

What?	V_{low}	V_{high}	$A_{min,low}$	A_{low}	$A_{min,high}$	A_{high}
Output	16.9 m^3	10.14 m^3	5.00 m^2	8.45 m^2	0.56 m^2	5.07 m^2

B.6 Final clarifier

The Sludge Volume Loading Rate (SVLR) is introduced in order to determine the dimensions of the final clarifier. This is an empirical based value which turns out to lie between 0.3 and 0.4 $m^3/(m^2 \cdot h)$ from previous experience with both round and rectangular tanks. An important parameter to keep in mind is the sludge volume surface loading rate Vs, which should not exceed 400 $L/(m^2 \cdot h)$. It can be calculated using the surface loading V_0 as follows:

$$Vs = V_0 \cdot X_{AT} \cdot \text{SVI} \quad \left(\text{with SVLR} = V_0 \cdot X_{AT} - \frac{\text{SVI}}{1000} \quad \Rightarrow \quad V_0 = \frac{1000 \cdot \text{SVLR}}{\text{SVI} \cdot X_{AT}} \right) \tag{B.18}$$

Since the design must fit in a 2.5m high container, the height of the secondary clarifier will be h=2 m. The surface area and diameter of the tank then follow from the surface loading and then amount of wastewater (including recycle flow) flowing through.

$$A_{final} = \frac{(1+R) \cdot Q_{avg}}{V_0} \quad \text{and} \quad D_{final} = \sqrt{4 \cdot \frac{A_{final}}{\pi}} \quad (\text{ if } h = 2) \tag{B.19}$$

Since the height has been determined to be 2 meter and the area is known, the volume can be determined by multiplying them. Furthermore the hydraulic retention time can be calculated according to the same formula as in equation B.15.

What?	SVI	SVLR	R	Q_{avg}	X_{AT}
Input	140 mL/g	0.4 $m^3/(m^2 \cdot h)$	1.50	0.25 m^3/h	4 g/L
How?	assumed	chosen	calculated	assumed	= inflow

What?	V_0	Vs	A_{final}	D_{final}	V	HRT
Output	0.71 $m^3/(m^2 \cdot h)$	400 $L/(M^2 \cdot h)$	3.50 m^2	2 m	7 m^3	28 h

B.7 Oxygen demand

The total oxygen demand can be calculated from equation B.20. All parts will be further explained below.

$$\text{OD} = \text{O}_\text{e} + \text{O}_\text{s} + \text{O}_\text{n} + \text{O}_\text{o} + \text{O}_\text{z} \tag{B.20}$$

$$\text{where} \quad \text{O}_\text{e} = b \cdot V_{AT} \cdot X_{AT} \quad \text{and} \quad \text{O}_\text{s} = c \cdot Bd \quad \text{and} \quad \text{O}_\text{n} = d \cdot Nd \tag{B.21}$$

Where O_e is the oxygen required for endogenous respiration, O_s is the oxygen required for substrate respiration, O_n is the oxygen required for both nitrification and denitrification, O_z is the oxygen required for the discharge of dissolved oxygen, which can be considered to be zero in this case. And lastly O_o is the oxygen required for quickly oxydizable components, which is also zero for this situation.

Endogenous respiration is the degradation of other bacteria for the benefit of energy production for the cells movement and other vital functions. Its oxygen demand can be calculated as in equation B.21, with the specific endogenous respiration factor b. b is dependant upon the sludge loading and the temperature: for design purposes with high sludge loadings, $b = 0.10$ kg O_2/(kg ds·d) is used.

If substrate is added to the wastewater, oxygen demand increases very rapidly. The substrate respiration factor c is a big unknown but is estimated from experience to be $c = 0.9$ kg O_2/kg BOD removed for very polluted wastewater.

Nitrification and denitrification require oxygen as well, stoichiometrically 1 kg of ammonia-nitrogen (NH_4-N) requires 4.57 kg O_2, but in reality this comes down to 4.33 kg O_2 due to nitrogen involved in bacteria cells (and it will be even further decreased if Struvite is made to remove phosphorus). Denitrification requires 2.86 kg O_2 for 1 kg of nitrate-nitrogen (NO_3-N), and therefore the factor d becomes 4.33+2.86 kg O_2/kg N.

What?	b	V_{AT}	X_{AT}	c	Bd	d	Nd
Input	0.10	8.45 m^3	4 g/L	0.90	10.14 kg BOD/d	7.19	1.446 kg N/d
How?	experience	calculated	chosen	experience	calculated	calculated	calculated

What?	OD	O_e	O_s	O_n	O_o	O_z
Output	22.9 kg O_2/d	3.38 kg O_2/d	9.13 kg O_2/d	10.40 kg O_2/d	0 kg O_2/d	0 kg O_2/d

As can be seen, the daily oxygen demand is 22.9 kg. The kWh price for a non-household is €0.13. The aeration efficiency for mechanical aerators lies around 1-2 kg O_2/kWh, and diffusive aeration around 2-8 kg O_2/kWh, depending on the system [24]. Since fine bubble aeration is chosen, a conservative average of 4 kg O_2/kWh is used. Combining these facts, this comes down to an annual cost for oxygen supply of €274 according to equation B.22.

$$C = 365[\text{d}] \cdot 22.9[\text{kg O}_2/\text{d}] \cdot \frac{1}{4}[\text{kWh/kg O}_2] \cdot 0.13[€/\text{kWh}] \tag{B.22}$$

C Capital Expenditures

System	Description	Qty	Price/pce	Total	Total group
Tanks	**Total**				**€51.200**
Tanks	Wall grills	2	€150	€300	
Tanks	Ductwork Alu tunnel suction (d=300mm)	20	€200	€4.000	
Tanks	Axial fan to air treatment (3.000 m3/h, 1.5 kW)	1	€2.500	€2.500	
Tanks	Pneumatic valve (d=300mm)	5	€1.100	€5.500	
Tanks	Compensator DN150 (15/t)	4	€150	€600	
Tanks	Pneumatic valves air supply (1 per AD tunnel)	4	€450	€1.800	
Tanks	Mounting works skid system on site (incl. elec.)	200	€75	€15.000	
Tanks	Frequency drive (1.5kW)	4	€500	€2.000	
Tanks	Cabling suction fan	1	€500	€500	
Tanks	Manual knife gate valves (DN63)	5	€300	€1.500	
Tanks	Pneumatic piping valves technical corridor	1	€1.000	€1.000	
Tanks	Fine bubble aeration tank-floor	1	€1.500	€1.500	
Tanks	Container 40ft. (incl. doors, vents, etc.)	1	€15.000	€15.000	
Water	**Total**				**€28.700**
Water	Mixers fermentation tanks	1	€1.000	€1.000	
Water	Level sensors fermentation tanks	4	€200	€800	
Water	Temperature sensor 1000m3 tank	4	€200	€800	
Water	Heating system 1000m3 fermentation tanks, 2 Kw	1	€500	€500	
Water	Water piping connections percolate tank	1	€4.000	€4.000	
Water	pH regulation system	1	€1.500	€1.500	
Water	Pumps water lock AD+CO, 1,5kW	2	€3.000	€6.000	
Water	Cabling sed. pit and water lock AD	2	€2.000	€4.000	
Water	Knife gate Valves at pumps water lock AD	2	€450	€900	
Water	PN knife gate valves at WWTP	2	€350	€700	
Water	Heating pumps heat exchangers AD Water	4	€1.250	€5.000	
Water	Flow meter process water waterskid AD	2	€1.750	€3.500	
Control	**Total**				**€28.500**
Control	PLC Siemens	1	€15.000	€15.000	
Control	Loopchecks and cold and warm start on location	1	€2.000	€2.000	
Control	Software PLC AD and Composting	1	€5.000	€5.000	
Control	Visualisation computer by SCADA	1	€6.500	€6.500	
Electrical	**Total**				**€8.000**
Electrical	Composting system by MCC	1	€8.000	€8.000	
Total					**€116.400**

D Preliminary WWTP PNID

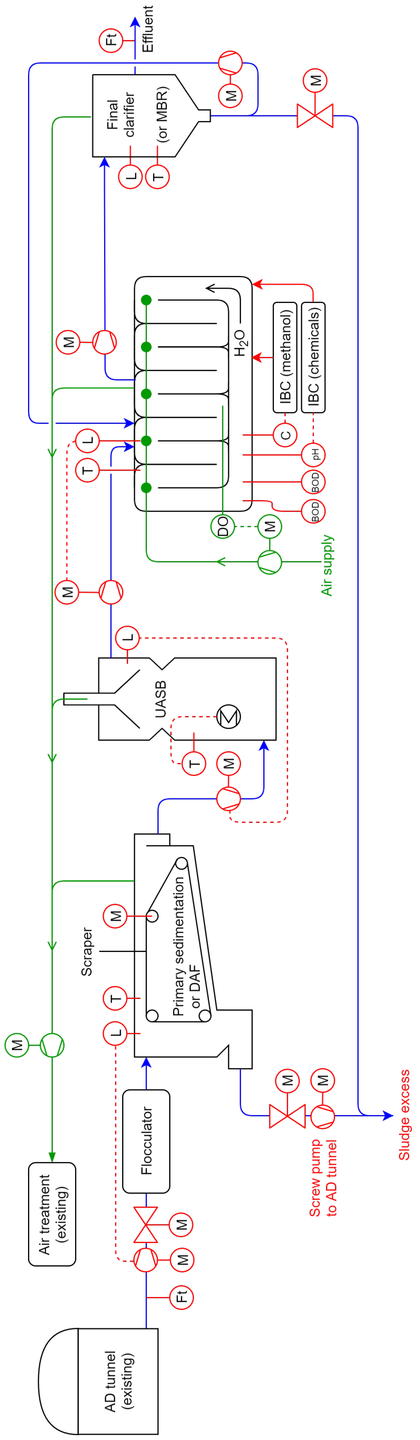

Figure D.1: Preliminary Piping and Instrumentation Diagram