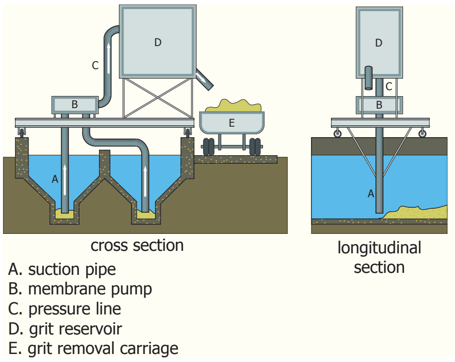
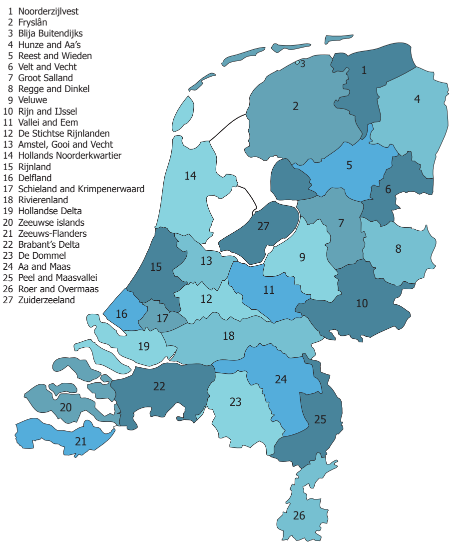
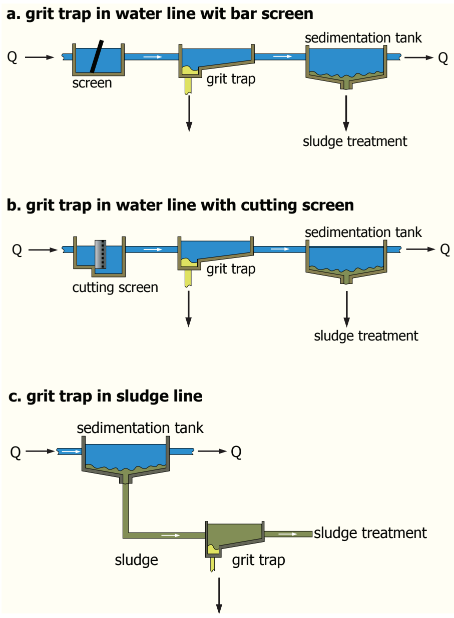
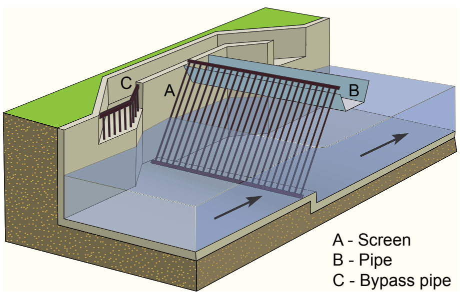
Wastewater Treatment









Preface

The wastewater treatment lectures are designed to provide insight into the processes and techniques that are applied in the wastewater treatment field, where the focus is laid on the basic principles, local conditions, basic circumstances and performance processes. Their mutual relationship is also explained. After this it will be possible for a student to set up a schematic wastewater treatment plant (wwtp) and to design various industrial components. This textbook is thus meant to bring about a general understanding of the basic principles of the treatment of wastewater.

For this the following are necessary:
- knowledge of the composition of wastewater;
- the requirements that are placed on the purified wastewater (the effluent), and the entire wwtp;
- the wwtp design coming from the various treatment concepts;
- the operation and the application of purification techniques.

In this lecture and syllabus we make use of general rules of thumb and maybe somewhat old design methods. Since this course is just an introduction to sewage treatment, the design methods you learn here will give some basic insight in sizing of the different process units. However, nowadays, most designs are based on the sludge kinetics and biological conversion rates. This will be taught in the Masters Course Waste Water Treatment (CIE 4485)

This text book is similar in many ways to a cook book; just as in every cook book the ingredients, techniques and recipes are essential. The ingredients of wastewater treatment are in the wastewater itself, bacterial cultures, and purification chemicals. The techniques are divided into physical, chemical, physical-chemical, and biological (aerobic and anaerobic) treatment techniques. Finally there are various recipes to treat wastewater. This is what differentiates the purification concept (configuration of various purification techniques).

The most important aspect of wastewater treatment is that the effluent (the purified wastewater) that leaves the wwtp and is drained onto the surface water meets discharge requirements. Along with that the purification water authorities (the regional water authority, the district water control board, and the sewage purification board) will pursue an environmentally friendly or sustainable design and operations with minimal effects on society.

For an in depth knowledge refer to 'Wastewater Engineering, Treatment and Reuse, 4th edition, Metcalf and Eddy, 2003; ISBN 0-07-11225—8. In this syllabus there is continual referral to the chapters in this manual with "ME" This book is also used in the continuation lectures in the Master's Programme.

1 Introduction

1. Introduction

1.1 Background

Water is neccesary for life on earth. In nature it comes in three aggregate forms (snow / ice, liquid, vapour / steam). The majority of particle transport is carried out through water. In water many particles become dissolved. Water also has a great capacity to take on heat.
In our society water fulfils many functions as the foundation for a natural habitat and more specifically for human activities, such as domestic use, industrial and agricultural usages, fishing, transportation and recreation.

A byproduct of human usage is that the water then becomes polluted with contaminants. As a result of this the water quality in the various compartments of the water cycle (ground water, surface water) is influenced. One of the most succinct examples of a disturbed water cycle is shown through the discharge of raw household and industrial wastewater into the surface water. In many cases the natural storage capacity limited to such a degree that a disturbance of the natural functions occur.

1.1.1 Contamination in the Netherlands

In the Netherlands the amount of raw pollution in 1970 amounted to approximately 50•10^6 population equivalents (PE) of which around 30 to 35•10^6 PE was from industrial sources. Of this total pollution it was estimated that approximately 45•10^6 PE was discharged to the surface water. In 2000 the total had decreased to approximately 35•10^6 PE through improvements in industrial sanitation, and due to the construction of purification plants the contamination of the surface water was reduced by approximately 5•10^6PE.

1.1.2 Regulation in the Netherlands

In 1970 the Wet Verontreiniging oppervlaktewater (WVO) (Surface water Pollution Act) was created. The purpose of the WVO is to protect surface water such as ditches, canals, rivers, and lakes paying special attention to the various functions that these waters fulfil in our society, such as drinking water, swimming and recreational functions, water supply for agriculture and horticulture and for industry and ecological functions. From this legislative framework there are many official bodies of state entrusted with the task to purify collected wastewater, sometimes as a province, or as wastewater treatment authority (see figure 1.1). The industries can purify their own wastewater before draining it into the surface water or have it be collected through the sewer system together with the domestic wastewater for treatment by the purification managers.

The (WVO) act makes it possible to tax for the discharge of contaminants which fits with the "polluter pays principle". The goal of the levies is to combat the costs created by the water quality managers for these enactments, which hinder and prevent the contamination of surface water. Assessment basis for the levies is the amount and the quality of the waste materials that are dumped.

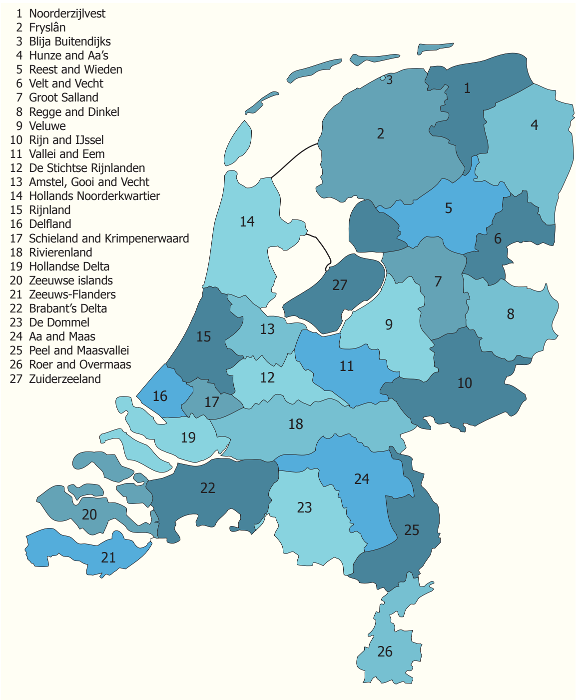


Figure 1.1 - The layout of water quality management in the Netherlands (2009)

Waste materials that are subject to being taxed as a result of discharge into the sewer or into the surface water are

- Oxygen consuming components
  To calculate the levies a unit accounting for the oxygen consuming components is used: the population equivalent (PE); that is the average oxygen consuming components' capacity from the waste produced, within a 24 hour period per person that gets discharged with the wastewater.
- Other materials (e.g. heavy metals)
  The "contamination unit" (v.e.) is employed in the Netherlands to account for other compounds.

Aside from oxygen consuming components since the 1980s much attention is being increasingly paid to fertilizing compounds, such as phosphates and nitrogen. What is more, in 1988 and 1990 thanks to the Rhine and North Sea campaigns international agreements were made that have led to a substantial reduction of phosphate and nitrogen emissions.

In recent years the issuing of rules has established a more European character. In this way the European Union directives set requirements in respect to city wastewater concerning contaminating particles in the effluent. There are three main directives which impact the emission limit values which wastewater treatment plants in member states must achieve prior to discharge of their effluent to surface water. These directives are the Urban Waste Water Directive, the Integrated Pollution Prevention and Control Directive/Industrial Emissions Directive, and the Water Framework Directive.

The Urban Waste Water Directive 91/271/EEC (UWWD) was adopted in 1991 to protect the environment, namely surface waters, from the adverse effects of the discharge of urban wastewater (combined domestic and industrial wastewater). The UWWD thus addresses the collection , treatment and discharge of domestic (or municipal) wastewater, industrial wastewater discharged to municipal sewers, and some direct industrial wastewater discharges with characteristics similar to municipal waste water (e.g. the food industry). Specific standards set by the UWWD depend on the population equivalent (p.e.) of the waste stream and whether or not the surface water being discharged to is considered a "sensitive area". The UWWD explicitly requires:

- collection and treatment of all waste water streams >2000 p.e.,
- secondary treatment of all discharges from waste streams > 2000 p.e.,
- advanced treatment for streams >10 000 p.e. discharging to sensitive areas,
- pre-authorisation of all discharges of urban wastewater, from the food-processing industry and industrial discharges into urban wastewater collection systems
- performance monitoring of treatment plants and receiving waters status
- sewage sludge disposal and re-use controls
- treated waste water re-use whenever it is possible and appropriate.

Integrated Pollution Prevention and Control Act (IPPC) was originally adopted in 1996 as Directive 96/61/ECC, and it along with 4 minor amendments was codified as Directive 2008/2/EC in 2008 and has subsequently been recast, along with other related directives, as a part of the new comprehensive Directive on Industrial Emissions 2010/75/EU (IED), which will repeal the existing directives in 2014. The IPPC directive provides a framework for regulating industrial discharges whose activities are potentially polluting, including the waste treatment industry. The IED was compiled to further reduce emissions from industrial installations, simplify implementation and streamline the permitting, reporting and monitoring requirements of the Directives it recasts. The IED is essentially a more comprehensive version of the IPPC, requiring even more industrial facilities to obtain permits for emissions discharge. The concepts of integrated pollution control and best available technology (BATs) based Emission Limit Values

(ELVs) in permits set forth in the IPPC are still central to the IED approach.
"Different approaches to controlling emissions into air, water or soil separately may encourage the shifting of pollution from one environmental medium to another rather than protecting the environment as a whole. It is, therefore, appropriate to provide for an integrated approach to prevention and control of emissions into air, water and soil, to waste management, to energy efficiency and to accident prevention."

Additionally the IED allows for flexibility when cost of compliance is significantly higher than the environmental benefit, has mandatory requirements for environmental inspections and ensures transparency and public participation in the process.

The Water Framework Directive 200/60/EC was adopted in 2000 as a means of restructuring European water policy and water management in a coherent way. In order to take a comprehensive look at the water cycle and how the resource should be protected, the framework was developed with input from a variety of stakeholders including local and regional authorities, water users and non-governmental organisations (NGOs). Specific goals of the WFD include:

- Protection of all waters; surface waters and groundwater
- achieving "good status" for all waters by a set deadline
- water management based on river basins
- "combined approach" of emission limit values and quality standards
- getting the prices right
- getting the citizen involved more closely
- streamlining legislation

The WFD addresses these goals by taking a river basin management approach as the most desirable model for water management and protection since it is a natural geographical and hydrological unit rather than an arbitrary political one.

The most important aspects of the WFD with respects to WWTP discharge limitation are those dedicated to surface water quality protection. The WFD requires ecological based protection for all waters by directing that they meet the standards off good biological and chemical status. Good status requires that the surface water's biological community be as close as possible to that which would be expected with no anthropogenic interference and that all chemical pollutant quality standards are achieved.

1.1.3 Investment in the Netherlands

The provisions for the treatment of wastewater form the largest environmental health activity in the Netherlands in terms of the costs. In the period between 1960 and 1990 the government invested nearly 4,000 million Euros in treatment works (largely in wastewater treatment plants and approximately 25% in transportation pipelines and pumping stations). Industry invested accordingly.

In the last 20 years a further 4,000 million Euros was invested and also in the future the annual investment level (for expansion and replacement) will certainly cost roughly 300 million Euros. The total costs for the urban wastewater management amounted to around 900 million Euros per year.

1.2 Wastewater Basics

1.2.1 Water Basics

- H_2O
- dipole
- solid, liquid, gas (0°C, 100°C)
- density (1,000 kg/m^3)
- heat: specific heat 4.18 kj/(kg ˚C); heat of evaporation/enthalpy of vaporization 2,250 kj/kg
- viscosity: 1.0 mPa.s with 20˚
- contaminants: dissolved (< 10 nm), colloidal (10 nm-1 μm), suspended solids (> 1 μm)
- water as a solvent: gases (Henry's Law), liquids (miscibility)
- ionisation: ions, acids, bases
- oxidation-reduction
- biology: bacteria, pathogens, substrate, nutrients

1.2.2 Wastewater Sources

Wastewater that is treated in a municipal wastewater treatment plant (wwtp) or sewage treatment plant (stp) can have various sources. The influent of the typical wwtp consists of mostly wastewater from households and buisinesses (urban/municipal wastewater, or "sewage").

There may also be industrial wastewater discharges to the wwtp. Often large industry and industry parks will pretreat their own wastewater in a special wwtp before it is discharged to the municipal serer or main wwtp. Water is used with many industrial processes and wastewater is produced. The amount and composition of wastewater are strongly related to the type of industry. And even the degree of internal recirculation and water saving has a large influence. If the industrial treatment is adaquate, it may be allowed directly discharge to the surface water.

More information: ME Chapter 3

Wastewater that comes from pure municipal and urban sources is of a fairly constant composition. If industries are also connected to the wwtp, the influent wastewater composition varies depending on the type of industry and the management. The contaminants transported by these wastewater flows determine the (biological) capacity of the wwtp required to treat it.

In addition to the municipal and industrial sources of wastewater that make up the base componant of aflow to a treatment plant (dry weather flow), the drainage of rainwater to the wwtp (storm water flow) will also contributes to the total wastewater flow to be treated at the wwtp. The extent to which stormwater will impact the wwtp is related to the type of collection system in place, as combined systems will carry much more stormwater to the wwtp than seperate systems. This storm water flow determines to a large degree the required hydraulic capacity of the wwtp.

1.2.3 Wastewater Flows

From the moment that drinking water is withdrawn from the pipelines it will, as a rule, be disposed of as wastewater either after a few minutes or with a longer lag. For example the drainage of water from a flushing tank in a toilet, or from a bath, a washing machine or a dishwasher will not occur right when it is drawn from the drinking waster system thus a delay will occur before it reaches the sewer. Also, there will be a levelling out in water consumption and discharge as a result of the population's behavioural pattern; in villages and small cities the behavioural pattern is more uniform than in large cities, so that in large cities, where the water consumption is usually higher than in small cities, the levelling out will be larger.

Finally, a delay and levelling out occur will also in the sewer system since it takes a certain amount of time before the wastewater reaches the pumping station or sewage treatment plant. These types of occurrences will be more significant in low lying areas than in hilly areas.

It should be pointed out that in a combined sewer wastewater flows can be buffered within the system. This can also be true for the wastewater flow from a separate sewer system because a substantial part of these sewers have, for practical reasons, a larger diameter than is deemed hydraulically necessary.

Ultimately, wastewater flows to sewage treatment plant show great fluctuations; however on a daily basis the fluctioations tend to often have a fixed diurnal pattern (see figure 1.2) due to the behavior of the populaton. In some instances additional fluctuations can be predicted based on water consumption, such as during a football match where the use of toilets during half time and after the games can be seen(See figure 1.3). Higher water consumption on warmer days than on colder days is another such predeictable behavioral trend.

With all these factors taken into account one can calculate the maximum amount of dry weather flow per hour hours assuming a factor 1/12 - 1/14 of the average daily amount of tap water. The factor 1/12 is applied for villages or small cities and the factor 1/14 for large cities.

In designing sewage treatment plants the following is most often assumed:

- Qdwf-day; this is the daily averaged discharge in m^3/d
  (i.e. 120-140 l/inh.d)
- Qdwf-max; this is the maximum hourly discharge in m^3/h during dwf
  = commonly Qdwf-day/(12 to 14)
  (i.e. 8 to 10 l/inh.h)
- Qdwf-gen; this is the hourly averaged drainage in m^3/h
  = Qdwf-day/24
  (i.e. circa 5 to 6 l/inh.h)
- Qdwf-min; this the minimum hourly discharge in m^3/h
  = Qdwf-day/48
  (that is. circa 2,5 to 3 l/inh.h)

The maximum amount of wastewater that is processed in the sewage treatment plant depends on the (type of) sewer system. As a reference value it can be said that the maximum hydraulic capacity of the wwtp can often amount to 3 to 5 times the Qdwf-max.

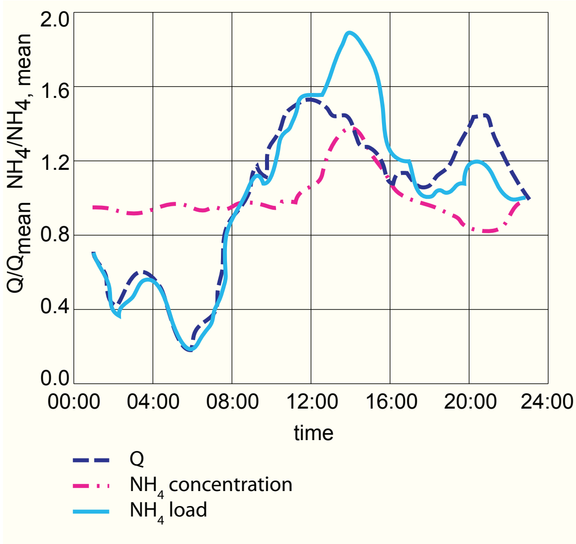


Figure 1.2 - The pattern of incoming domestic wastewaater

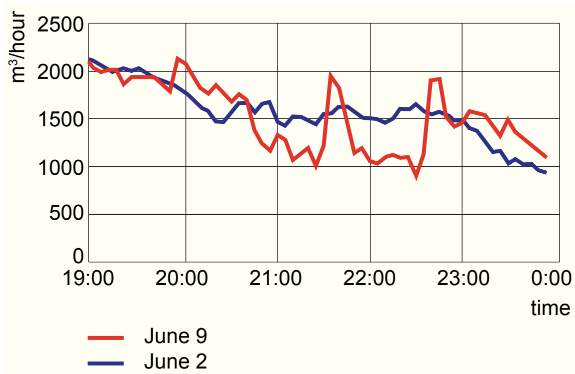


Figure 1.3 - European Championships: water consumption in Gouda and the surrounding area, 9 June 2008, the Netherlands – Italy (with respect to a normal day) [source: Oasen Gouda 2]

In conclusion a wwtp must be able to withstand large hydraulic fluctuations. Likewise it is characteristic that a maximum supply only occurs at 10 to 20% of the time.

1.2.4 Wastewater Composition

The composition of wastewater varies depending on the sewer area (countryside, village, city, industry) to which it is connected, the type of sewer system (separate, combined, pressurised mains, gravity sewer) length and the slope of the sewer system, state of the sewer (leakage or drainage) and the diverse drainage systems.

The most important components in urban wastewater are depicted in table 1.1. The 'average' composition of wastewater in the Netherlands is shown in table 1.2.

Table 1.1 - The most important components in urban wastewater

chemical oxygen demand	COD	g O_2/m^3
biochemical oxygen demand	BOD_5^{20}	g O_2/m^3
total nitrogen	N_{total}	g N/m^3
N-NH_4 (ammonium nitrogen)	NH_4	g N/m^3
nitrite	NO_2	g N/m^3
nitrate	NO_3	g N/m^3
Kjeldahl nitrogen	N-Kj	g N/m^3
total phosphorus	P_{total}	g P/m^3
ortho-phosphate	P_{ortho}	g P/m^3
dry solids	ds	g ds/m^3
total suspended solids	TSS	g ds/m^3
chloride	Cl^-	g $/m^3$
sulfide	S^{2-}	g $/m^3$
sulphate	SO_4^{2-}	g/m^3

More information: ME Chapter 2
N.B. mg/l is often used instead of g/m^3.
The many types of contaminants that are found in wastewater can be classified in a few groups:

Solids
- Inert Solids
- Volitile Solids
- Dissolved, suspended or settleable

Biodegradable compounds:
- organic compounds (protiens carbohydrates, and fats)
- organic nitrogen compounds;
- other organic compounds (e.g. organic sulfides).

Non-biodegradable or slowly biodegradable compounds:
- inorganic compounds (salt, acids, bases, mineral sludge);
- organic compounds, to be distinguished in natural compounds, such as humic acid compounds and xenobiotic compounds, like chlorinated hydrocarbons and other persistent compounds. (hormones, pharmaceuticals)

Nutrients:
- phosphorus compounds;
- nitrogen compounds;
- other nutrients.

Toxic substances:
- inorganic toxic substances, such as compounds from metals and metalloids;
- organic toxic substances, such as pesticides and carcinogenic compounds.

Radioactive substances.
Pathogenic organisms.
Thermal pollution.

These wastewater conbstituents of concern and how they are quanitified will be discussed in more depth in he following section.

Table 1.2 - Composition of wastewater for Dutch wwtps

parameter		1992	1995	1998	1999	2000
BOD_5^{20}	(mg O_2/l)	196	185	173	185	180
COD	(mg O_2/l)	533	510	456	480	470
N-Kj	(mg N/l)	47.5	47.2	41.8	42.2	40.4
P_{total}	(mg P/l)	7.8	7.6	6.7	6.6	6.3
Flow	(10 m^3/d)	4,871	5,071	5,879	5,518	5,506

1.3 Wastewaster Contaminants

1.3.1 Solids

The size of compounds in wastewater differ greatly. Visible particles or undissolved matter are 0.1µm and larger. Compounds with a particle size between 1 and 100 nm are called colloidal particles. Dissolved solids measure from 1 nm or smaller. In table 1.3 an overview is given of the amount of contaminants per inhabitant per day.

The settle-able solids are determined in a sedimentation experiment by putting a sample in a special conical funnel (Imhoff-glass) of 1 litre for 2 hours. The glass must be regularly rotated to prevent the particles from sticking to the glass walls. In this way the settled sludge is determined in ml/l.

The total content of dissolved and dissolved particulate compounds is determined by evaporating a sample and then after that drying the residue at 103˚C; after weighing the residual weight the dried solid content can be calculated in g/l or mg/l.

The undissolved solids can be separated from the dissolved solids through filtration; the remaining material from and on top of the filter, after having dried and weighed the filtration solids in g/l or mg/l (=undissolved solids), can be determined. From the filtrate after evaporating comes the content of dissolved solids.

The amount of inorganic solids present in a sample of wastewater or sludge, indicated by ash content, is assessed by incinerating the evaporation residue or the filtration residue for 45 minutes in an oven temperature of 550˚. The amount of organic matter, that is: the weight loss can be found through subtracting the ash content from the weight of the dried solids. Older instructions still report an incineration temperature of 600˚C, but presently 550˚C is insisted on to prevent the carbonates from decomposing.

Table 1.3 – Contaminants in wastewater of one inhabitant per day in grams

	inorganic	organic	total
settleable	20	40	60
colloidal	10	20	30
dissolved	50	50	100
total	80	110	190

1.3.2 Biodegradeable Organic Substances

Organic compounds are formed mainly from carbon and hydrogen, bonded with other elements. Carbons tyically present in domestic wastewater include:

- carbohydrates generally to be seen as $(CH_20)_n$
- fats (esters of glycerine and fatty acids);
- proteins (compounds made up of C and H and also N and sometimes P and S);
- urea, that gets excreted through urine: $CO(NH_2)_2$.
- Phenols detergents and pesticides

Explicit inorganic substances in wastewater are salt, sand, loam and ash, and cannot be remobed through biodegredation.

Oganic compouunds within wastewater afor the most part biodegradable, thus in the presence of aerobic microorganisms these biodegradable compounds can be broken down, consuming dissolved oxygen in the process.

Oxygen is reasonably soluble in water. The solubility depends on the pressure, the temperature and the content of dissolved substances (see figure 1.4). In general the influence of dissolved compounds is limited, so the oxygen saturation value amounts to 11.3, 10.7 and 9.0 mg O_2/l with a chloride content of respectively 0, 5 and 20 g Cl/l.

Through measuring the oxygen demand/ an impression is obtained of the content of organic compounds. This usually happens with one of the following methods:

- the biochemical oxygen demand (BOD), i.e. by means of bacteria;
- the chemical oxygen demand (COD), with the help of a strong oxidiser such as potassium dichromate.

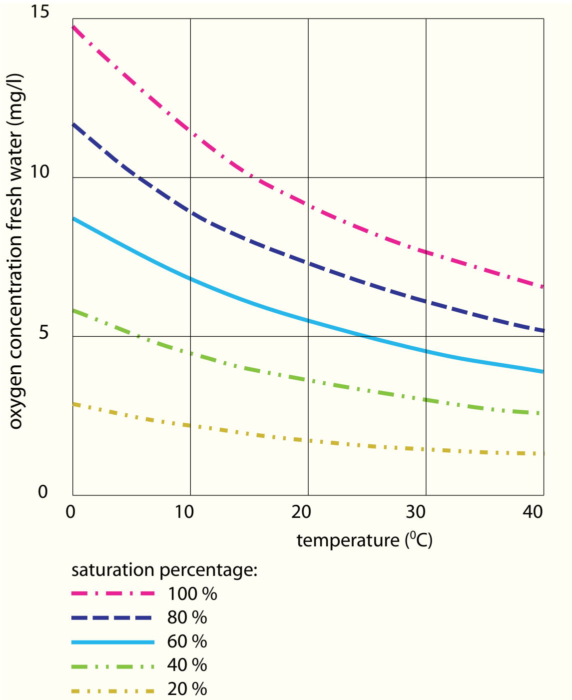


Figure 1.4 – Oxygen concentration and oxygen saturation percentage in relation to the temperature (air pressure 101.3 kPa)

Biochemical Oxygen Demand (BOD)

The biochemical oxygen demand, BOD, is the amount of oxygen in mg that is necessary to transform biochemical oxidizable elements by means of bacteria present in 1 litre of water.

To measure, mix a sample of wastewater with pure water with an oxygen content that is known and determine, after the mixture has been stored (usually) 5 days in a dark place at 20˚C, how much oxygen is used for the oxidation of the organic matter. The test must take place in the dark in order to prevent the occurrence of oxygen producing algae from happening simultaneously. There are two oxygen measurements necessary, namely one at the onset and one at the end of the experiment. The greater the content of the biochemical oxidizable substances, the greater the consumption of oxygen. In practice people usually use the BOD^{20}_{5} that is to say a test that lasts 5 days at a temperature of 20˚C. This means however not, that all biochemical oxidizable elements after 5 days are completely oxidized by the bacteria, as for a complete transformation a lot more time is necessary.

In oxidation through the biochemical process first the biochemical oxidizable elements are oxidized, these are taken in as food facilitated by the bacteria. A schematized course of the oxygen demand with a time-line at 20˚C is shown in figure 1.5.

The biological transformation progresses in two steps. The first flowing curve is called the carbon step. These immediately catch on and are, at 20˚C, and after around 20 days completed. The oxygen demand, exclusively for the oxidation of the carbon, is shown on the dotted line. In the example the BOD_5^{20} = 100 mg/l.
The BOD_{20}^{20} = 146 mg/l = 1.46 BOD^{20}_{5} through the carbon oxidation after 20 days.

The second or the nitrogen stage, also referred to as the nitrification stage, starts after around 10 days and lasts very long. The name nitrification stage is derived from the fact that in this stage along with oxidation of the remaining carbon compounds, various nitrogen compounds are also oxidized.

To prevent interference with nitrification allylthiourea is added (=BOD_{atu}), that stops the nitrification. Per inhabitant an average of 54 g BOD is drained daily, and after sedimentation approximately 35 g BOD is left in the wastewater. These values vary depending on the lifestyles and prosperity per country.

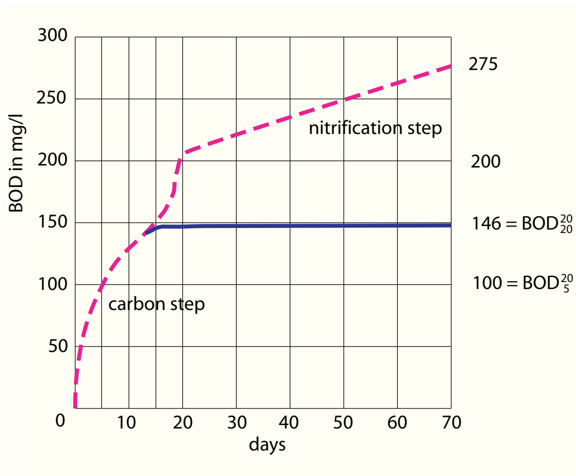


Figure 1.5 – Schematized progress of the biochemical oxygen demand/consumption at 20˚C.

Chemical Oxygen Demand (COD)

In the determination of the COD most organic compounds are nearly all chemically oxidised. Potassium dichromate is used as an oxidizing agent. In determining the COD the following is added in a sample:

- A known amount of potassium dichromate ($K_2Cr_2O_7$);
- A certain amount to silver sulphate (Ag_2SO_4) that serves as a catalyst agent for the oxidation;
- Mercury(II) sulphate ($HgSO_4$) to prevent chloride from oxidizing.

The remaining amount of potassium dichromate is determined after two hours of boiling under back flow cooling in a basket. The consumed amount of oxygen is calculated by comparing the difference between the original amount of potassium dichromate and the remains.
The results obtained are much less susceptible to fluctuations than the results from a BOD determination. The aberrations can be limited to 3-5%. The presence of toxic substances does not influence the analysis results and practically a complete oxidation of the cellular and humic acids takes place.

Comparing BOD and COD

Despite that both (expressed in gO_2/m^3) are a measurement for the concentration of organic matter, BOD and COD have different meanings. The COD represents the exact concentration of organic matter because the chemical oxidation is complete. In fact there is a fundamental connection between the number of electrons that are absorbed onto the organic matter (read: is available) and the measured amount of used oxygen. Therefore the COD is extremely useful for formulating a mass balance: after all no electrons can be lossed or made. A possible complication is that some inorganic substances that are present (such as sulphide) are also chemically oxidized, which means that previously this had to be corrected. For biological wastewater purification is COD determination useful because the non biodegradable organic substances are also chemically oxidized.

The BOD is more representative for the concentration (aerobic) biodegradable components because the assessment is based on the measuring of oxygen consumed through aerobic micro-organisms during the decomposition of the compounds. A special point of interest is that some inorganic compounds, of which ammonium is the most well-known, can also be biochemically oxidized through certain micro-organisms and therefore can add up to the BOD. This must be taken into account or be corrected. The usefulness of the BOD is actually limited since the biochemical oxidation of organic matter is not always complete; also because the time that the micro-organisms is given for the decomposition is chosen fairly arbitrarily. On top of that the micro-organisms, which are used for determining the BOD, also still use a part of the organic compounds for making new biomass (growth). For these reasons the BOD from a wastewater sample is always lower than the COD.

The comparison COD/BOD is otherwise often used as an indication for the biological treatability of wastewater: a ratio of 2 or lower clearly shows aerobic treatability of wastewater, while a ratio from above 2.5 is an indication for less treatable wastewater, for example due to the presence of toxic components. Apart from that these ratios say little about the anaerobic biodegradability of the organic matter: anaerobic micro-organisms work, after all, according to different biochemical decomposition routes than aerobic organisms.

In practice BOD and COD are used alongside each other, depending on the utilization. Furthermore, the employment as a measure of the concentration of organic matter is dependant on the geographical context: in North America BOD is most often used, while Europe prefers to use COD.

The use of the BOD as an indication for wastewater concentration is sometimes doubted. In fact the BOD was developed as a test for the effect of discharge on the river's water quality, where the point is actually about much lower concentrations of polluting compounds than in wastewater. The story goes that the BOD_5 test was developed in England where it was determined that the water in the Thames took an average of 5 days to reach the sea, that is why there is a 5 day incubation time. [source: www.heritagesystemsinc.com]

1.3.3 Nutrients

Nitrogen and phosphorus are the two most common nutrients monitored in wastewater effluent due to their roles as limiting nutrients in eutrophication of marine and freshwater environments respectively.

Nitrogen

Nitrogen can come in various forms, namely as an organically bound nitrogen, such as proteins or the decomposition products from that (amino acids), and in an inorganic form, as NH_3 (ammonia) or NH_4^+ (ammonium). During the mineralisation process the organically bound nitrogen first turns into ammonium ions (or ammonia depending on the pH).

The content of the total of the organically bound nitrogen and the ammonium nitrogen in mg/l is determined according to the Kjeldahl method. This is also referred to as Kjeldahl-nitrogen (N-Kj).

The oxidation of ammonium is separated in two steps:

$$2NH_4^+ + 3O_2 \longrightarrow 2NO_2^- + 2H_2O + 4H^+$$
$$2NO_2^- + O_2 \longrightarrow 2NO_3^-$$

$$2NH_4^+ + 4O_2 \longrightarrow 2NO_3^- + 2H_2O + 4H^+$$

For the first nitrification step the bacteria type nitrosomonas (nitrite forming) is necessary, while the second nitrification step is carried out through the nitrobacter bacteria type.

The nitrifying bacteria develop only slowly and the reactions occur as long as the temperature does not drop too far below 10˚C. The nitrification stops if the oxygen content is dropped to circa 1 mg/l or lower. Observe that the nitrification leads to the forming of H^+, which means that enough buffer capacity must be present to prevent acidification.

Total Nitrogen (TN) is a measurement of both the Kjeldahl-nitrogen and the nitrate and nitrite.

The daily amount of discharged nitrogen per inhabitant can be assumed at 10 g. The chemical equation shows that per one gram of nitrogen 4.57 g oxygen is consumed.

Phosphorus

Monitoring phosphorus is challenging because it involves measuring very low concentrations down to 0.01 milligram per liter (mg/L) or even lower. Even such very low concentrations of phosphorus can have a dramatic impact on streams. Less sensitive methods should be used only to identify serious problem areas.

While there are many tests for phosphorus, only four are likely to be performed by volunteer monitors.

1) The total orthophosphate test is largely a measure of orthophosphate. Because the sample is not filtered, the procedure measures both dissolved and suspended orthophosphate. The EPA-approved method for measuring total orthophosphate is known as the ascorbic acid method. Briefly, a reagent (either liquid or powder) containing ascorbic acid and ammonium molybdate reacts with orthophosphate in the sample to form a blue compound. The intensity of the blue color is directly proportional to the amount of orthophosphate in the water.

2) The total phosphorus test measures all the forms of phosphorus in the sample

(orthophosphate, condensed phosphate, and organic phosphate). This is accomplished by first “digesting” (heating and acidifying) the sample to convert all the other forms to orthophosphate. Then the orthophosphate is measured by the ascorbic acid method. Because the sample is not filtered, the procedure measures both dissolved and suspended orthophosphate.

3) The dissolved phosphorus test measures that fraction of the total phosphorus which is in solution in the water (as opposed to being attached to suspended particles). It is determined by first filtering the sample, then analyzing the filtered sample for total phosphorus.
4) Insoluble phosphorus is calculated by subtracting the dissolved phosphorus result from the total phosphorus result.

All these tests have one thing in common they all depend on measuring orthophosphate. The total orthophosphate test measures the orthophosphate that is already present in the sample. The others measure that which is already present and that which is formed when the other forms of phosphorus are converted to orthophosphate by digestion.

1.3.4 Pathogens

For the bacteriological/ assessment of water in terms of whether it has been contaminated with faeces is often used for checking the presence of the coli-bacteria. 10^6 - 10^8 intestinal bacteria are found in 100 ml. domestic wastewater. A large amount of that in general consists of innocent coli-bacteria. In faeces bacteria can also be present that cause intestinal diseases such as typhoid fever, paratyphoid fever (salmonellosis) in various forms and bacillary dysentery as well as viruses, of which can be the causers of infantile paralysis, jaundice, and worm eggs, for example lint worms and round worms and small intestinal worms. Moreover contact with wastewater can bring about skin reactions.

In view of the diversity of pathogenic organisms and viruses that can occur a routine test of the presence and absence of various types is not possible. Some types come actually in scant numbers, so that there is a good chance that the sample taken does not contain any and thus incorrect conclusions are made. To overcome both difficulties often the coli-content is determined; indeed the presence of coli-bacteria indicates a faecal contamination and that allows for the possibility that also pathogenic bacteria and viruses are present. The organisms that belong to the coli-group, what is referred to as coliform organisms or coliforms, is the real coli-bacteria the Escherichia coli (E-coli) a meticulous characterized organism that occurs in the intestines of warm-blooded organisms. There are various cultivation techniques to reveal these coliform organisms, where culture media are used. Many occur at 37˚C (coliforms) or 44.5˚C (faecal coli, E-coli). The amount of coli-bacteria are expressed in a number of colony forming units (cfu) per 100 ml.

In terms of the order of size some figures can contribute to the understanding about the degree of hygienic quality of wastewater. In wastewater the coliforms count is $3{\cdot}10^5$ per ml; in effluent the count is reduced to $6{\cdot}10^3$ per ml. This is in sharp contrast with the requirements for swimming water (100 cfu/100ml), drinking water (1 cfu/100 ml), coast water (200 cfu/100 ml) and fresh water (1000 cfu/100 ml).

1.3.5 Treatment Requirements

In order to ensure that the contaminants described above do not adversly impact the surface water by direct discharge of wastewater there are typically requirements for wastewater to be treated prior to discharge, and limits given to the amount of said contaminants that can allowbly be discharged in the efluent after treatment.

The discharge requirements in the Netherlands are laid down on the basis of the WVO (Wet Verontreiniging Oppervlaktewater (Surface Water Pollution Act)), various general enactments by the government (Rijnactieplan (Rhine campaign) and

Noordzeeactieplan (The North Sea campaign)) and the EU-directive for urban wastewater. These

Table 1.4 - Effluent requirements in the Netherlands (units in mg/l) [source: Urban Wastewater Drainage Act 2009]

parameter	requirements	**concerns**
BOD_5^{20}	20	
COD	125	
suspended solids	30	
P_{total}	2.0	WWTP < 100,000 PE
	1.0	WWTP > 100,000 PE
N-Kj	20	
N_{total}	10	new WWTP > 20,000 PE
	15	new WWTP < 20,000 PE
	10	existing WWTP > 20,000 PE
	15	existing WWTP < 20,000 PE

requirements are listed in table 1.4. New WWTP have to fullfill the new WFDs, which could lead to mroe stringent effluent demands, as Ntotal below 5 mg/L and Ptotal as low as 0.3 mg/L

In some situations (mostly in arid areas) it is possible to make use again of purified wastewater, for example irrigation purposes. It is important then to strongly improve the hygienic quality. This can be distinguished by putting in post-treatment steps such as effluent polishing ponds, chlorination and filtration.

Mostly the re-use strategy focuses on avoiding contact with edible washed fruits and vegetables with specific techniques such as drip irrigation. Along with irrigation purified wastewater is being increasingly put to use in other (low quality) applications; consider industrial processes or cooling water usage by industries, rinse water for toilets and water for landscaping.

1.4 Treatment Plant Basics

In most situations the required wastewater treatment is carried aout at wastewater treatment plants. A sewage treatment plant must:

- treat the wastewater to such an extent that meets the effluent requirements.
- be able to handle the inherent variations that are in the wastewater as well as variations in the amount of wastewater.
- be robustly constructed;
- be able to treat the wastewater at a minimum annual cost;
- cause minimum amount of disturbance to the surrounding areas.

In this section the general design primipals and components of these wwtp will be introduced.

1.4.1 Design Loads

A WWTP in general is designed based on the hydraulic intake (hydraulic load) and the degree of contamination in the wastewater (biological load).

Biological

The degree of contamination of the wastewater is determined by the organic material's supply load (BOD or COD) and the nutrient loads (nitrogen and phosphate). The contamination load is commonly expressed in population equivalents (PE) or the pollution equivalent. In general the definition of PE is used for domestic wastewater and the pollution equivalent is used for industrial or company wastewater. Typical definitions of PE are 54 g BOD or 136 g TOD (total oxygen demand) per person per day.

The contamination load in population equivalents is calculated accordingly:

PE per 54 g BOD/d = [(BOD)*Q] / 54

The contamination load in pollution equivalents is calculated accordingly:

PE per 136 g TOD/d = [(COD +4.57 N-KJ)*Q] /136

Where Q is the influent flowrate in m^3/d and COD/BOD and N-KJ are the influent concentrations in g/m^3

Given that the load of contaminating components per day has but a limited variation, the daily load is mostly used as a starting point for the dimensioning, on top of which variations during the day as well as seasonal fluctuations must be taken into account.

Hydraulic

The hydraulic design of the WWTP is determined by the supply flow rate – mostly the amount per hour – during dry weather flow (dwf) and wet weather flow (wwf) (see 1.2.2).

The normative wet weather flow going to the WWTP is determined throughout the dwf and the excess pumping capacity from the last pumping station or the influent pumping station at the WWTP. The design capacity for the WWTP amounts globally to: Q_{max} = 30-50 l/(PE.h)

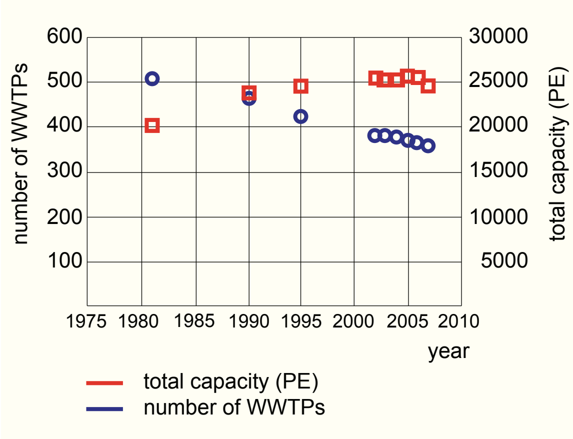


Figure 1.6 - Number of WWTPs and capacity in the Netherlands [Source: CBS 2009]

1.4.2 Dutch WWTP Data

Figure 1.6 shows that the total capacity of the WWTPs in the Netherlands has increased, while the number of plants has reduced, this is a result of continuously constructing larger installations. A few characteristic facts about urban wastewater treatment are given in table 1.5.

Table 1.5 - Data from Dutch WWTPs (2001)

WWTP				
- number	356	#		
- design capacity	24,5	10^6 (population equivalent per 136 g TOD = PE)		
- current taxes	21,7	10^6 PE		
- flow	5,67	10^6 m^3/day		
Influent composition	(average)		**effluent quality**	(average)
COD	471 mg O_2/l		COD	51 mg O_2/l
BOD_5	174 mg O_2/l		BOD_5	7 mg O_2/l
N_{total}	44 mg N/l		$N_{Kjeldahl}$	6,6 mg N/l
			N_{total}	13,5 mg N/l
P_{total}	6,7 mg P/l		P_{total}	1,8 mg P/l
suspended particles	223 mg/l	suspended particles	10 mg/l	10 mg/l
sludge production				
total	339	10^6 kg d.s./year		
per PE	16	kg d.s./year = 44 g d.s./day		
per m^3	164	g d.s.		
energy consumption				
total	520	10^6 kWh/year		
per PE	24	kWh/year = 2.7 Watt		
per m^3	0,25	kWh		

1.4.3 Treatment Process Components

For more information see ME Figure 5-1

As different types of contaminations in wastewater can occur, wastewater is treated in various successive steps (see figure 1.7). A diagram of a WWTP is given in figure 1.8. The round sediment tanks are a quickly recognizable characteristic of a conventional WWTP.

In table 1.6 an overview is given of the contaminants that usually occur in domestic wastewater, the effects that sometimes happen during discharge and the possible processes that can be applied.

Pre-treatment is dedicated to removing coarse material and sand; these particles can cause difficulties further along in the treatment process (blockages, damage to the components, etc.) if they are not removed. Subsequently the settleable substances are separated (mechanical

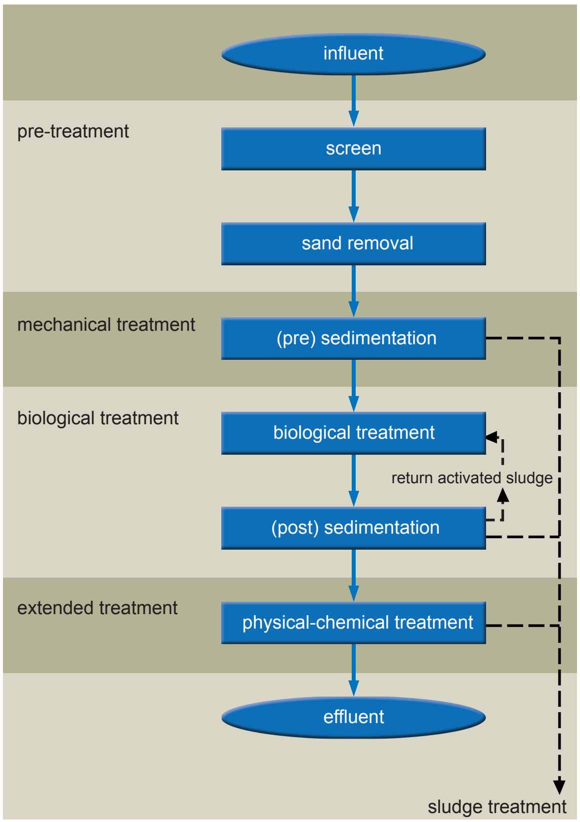


Figure 1.7 – Possible steps in the treatment of wastewater

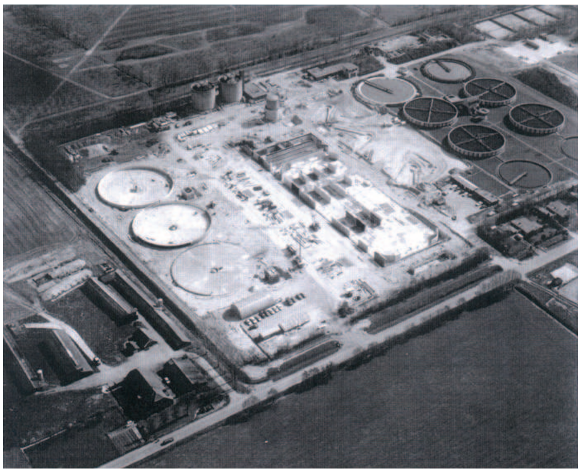


Figure 1.8 – Overview of a WWTP (under construction)

treatment) which is referred to as primary treatment or clarification, however this step can also be omitted in some treatment configurations. After this follows secondary treatment which encompases the removal of the dissolved and suspended organic contaminants through a biological process; nitrogen and phosphorus compounds can also be removed in the secondary process. Then the biological (or activated) solids produced by the secondary process are separated in a secondary clarifier. Next the effluent is either discharged into the surface water or an extended or physical-chemical purification process can be deployed as the last phase prior to discharge. The extended treatment often focuses less on the oxygen demanding substances, but more on other components (e.g. micro pollutants, heavy metals, pathogens, etc.). These processes include active-carbon treatment, filtration, chlorination, membrane filtration, ion exchangers and chemical precipitations. In practice this rarely happens.

1.4.4 Sludge Basics

For more information see ME Chapter 14-1 and 14-2

Sludge is a collective term for settle-able solids that get separated after treating wastewater. This includes primary sludgefrom the sedimentation of raw sewage and the surplus activated sludge (waste activated sludge, WAS) produced during the biological process treatments.

Table 1.6– Overview contaminants, effects on treatment systems

contaminants	effects on discharge	processes
a. coarse particles and settleable solids	sludge sedimentation decompostion oxygen depletion	sieves sediments
b. non settleable, biodegradable substances	oxygen depletion	biological treatment
c. ammonia (Kjeldahl-N)	oxygen depletion toxic for fish negative for drinking water preparation eutrophication	biological nitrification chemical-physical stripping
d. undissolved (suspended particles)	oxygen depletion eutrophication	micro-sieves filtration
e. inorganic nutrients - nitrate - phosphate	eutrophication influencing oxygen content negative for drinking water preparation	biological denitrification chemical precipitation biological removal
f. dissolved, biological resistant organic particles	poisoning destroying biotope accumulation in the food chain negative for drinking water preparation	activated-carbon absorption chemical oxidation
g. dissolved inorganic particals	poisoning destroying biotope accumulation in the food chain negative for drinking water preparation	ion exchangers electro dialysis reversed osmosis distillation
h. pathogenic organisms	worsening hygienic quality	disinfection

Sludge characteristically has a large fraction of organic material (circa 50 - 80% of the total solids). Furthermore a large amount of water is bond throughout the sludge mass (water amount 95-99.5% of the total).

The sludge mass is often expressed is suspended solid units. Suspended solids are the quantity by weight that remains after (filtration followed by) evaporation and drying at a temperature of 103˚C.

The suspended solids can further be subdivided into an organic partand an inorganic part by incineration at 550˚C; the organic part is the weight loss, the inorganic particle is the remaining ash.

The amount of the sludge is expressed in volume units; along with this the quantity of water is also taken into account.

The relation between these two quantities is formed by:

- % dry solids (ds), being the weight fraction of the suspended solids in respect to the total weight (5% ds means 5 g ds on every 100 g total);
- kg ds/m^3, being the mass on the suspended solids per unit of volume, or concentration.

If the sludge is still liquid, the volume quantity can still be reasonably good to determine so that both criteria are then (still) usable; if the sludge takes on a solid form, the concentration loses its practical value.

As a byproduct of the wastewater treatment process, the sludge must also be treated and disposed of in a safe and economic manner by the wwtp.

2 Primary Treatment

2. Primary Treatment

2.1 Screens

For more information see ME Chapter 5 -1

2.1.1 General

Wastewater carries certain solid materials such as wood, plastic and fibrous material. These substances can cause serious problems in the treatment process by plugging the pumps and mains or by forming floating layers in the digestion tanks. The size is usually such that it is possible to remove it through a screen or sieve. The placement in the process is usually directly after the influent pump.

2.1.2 Bar Screens

Bar screens are made up of a number of parallel rods that are equidistant and are positioned at an angle of circa 75˚. Cleaning the screens is carried out by a raked bar screen. This can be done manually as well as automatically. The screen waste, also known as screenings, is often removed through a conveyer belt to a container or to a screenings press. Sometimes a reduction is adjusted ahead of time. Often a bypass pipe is installed at the screen which allows the water to drain if the screen is clogged (see figure 2.1 and 2.2).

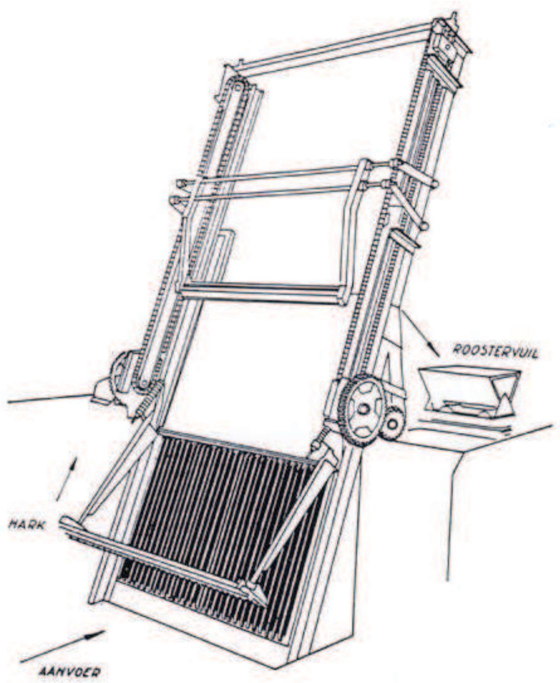


Figure 2.1 - Bar screen

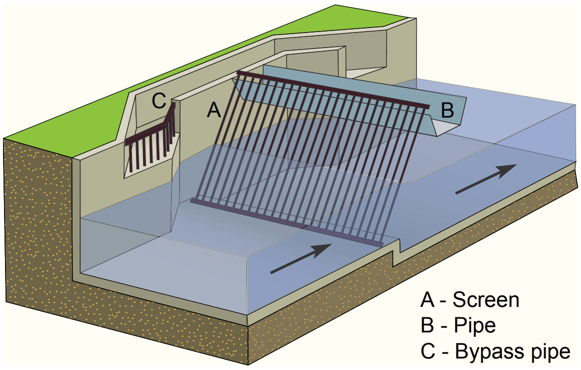


Figure 2.2 - Bar screen with bypass pipe

Design of Bar Screens

One of the most important process parameters with a bar screen is the resistance that the screen creates against the water flow. This resistance will increase as a result of contamination. The resistance loss resulting from the presence of a clean screen is a function of the velocity, the form, thickness and distance of the screen bars and the angle at which the screens are placed. To prevent the resistance loss from increasing to 10 to 20 cm, the velocity through the screen is often kept at a value of maximum 0.6 to 1.0 m/s. The flow velocity in the intake pipe must always be high enough so that no sand settles (> 0.4 to 0.5 m/s).

Bar Screen Opening

Depending on the screen openings (distance between the rods) the bar screens are divided into:

- coarse screens with a opening of 50 to 100 mm
- fine screens with an opening of 5 to 20 mm

The coarse screen is mainly employed for combined sewer systems to hold back the large debris such as rafters, planks etc.; in general it is placed in front of a fine screen.

With a fine screen the openings between the bars are as small as possible, so that the most amount of screen refuse as possible is removed. Otherwise this material causes process disruptions either within the biological part of the treatment plant or it is discharged as floating scum along with the effluent, which is also undesirable.

2.1.3 Continuous Screen

Presently continuous screens are often being installed where the screen moves through the sewage and takes with it the waste; the waste can also be removed from the sewage by a slow combined movement of the screen components. Especially for these types very scant openings (3-8 mm) are installed (see figure 2.3).

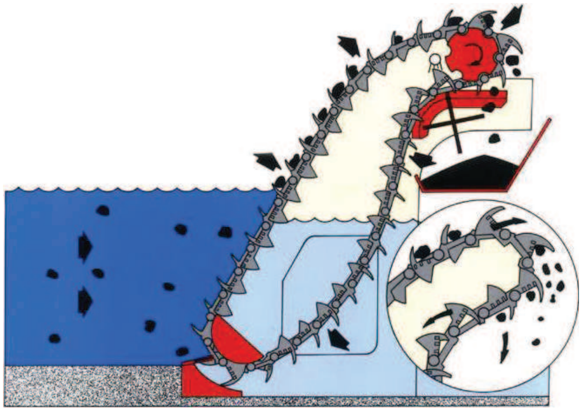


Figure 2.3 - Continuous screen

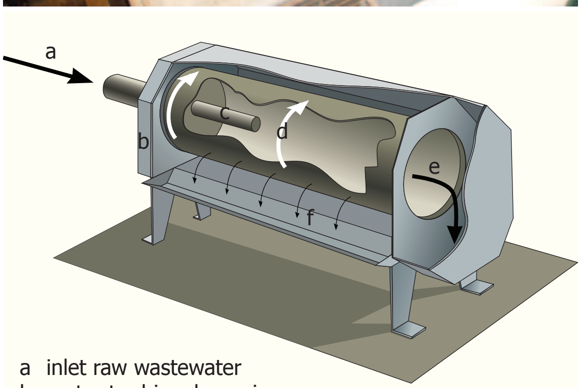


Figure 2.4 - Drum sieve (East-London, South Africa)

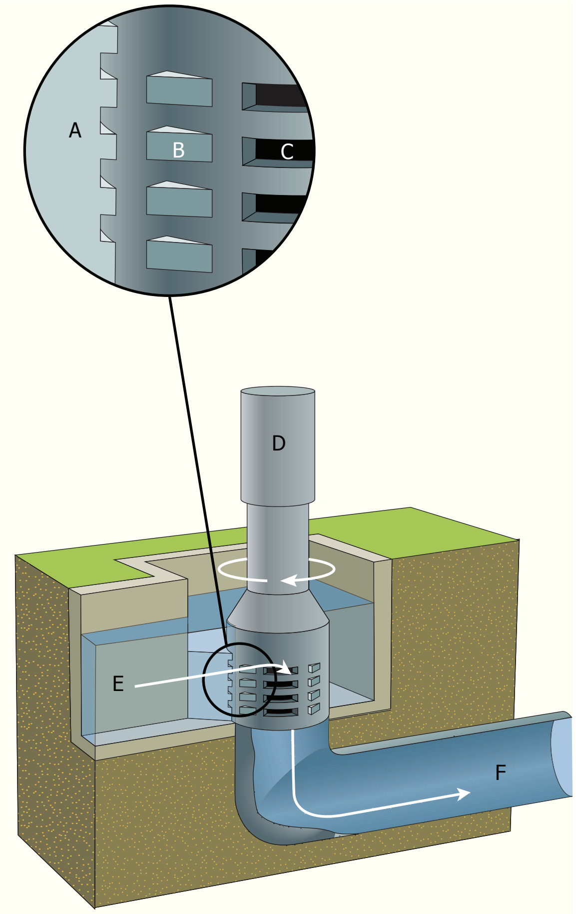
2.1.4 Sieves

Sieves are installed in the treatment of wastewater as fine screens. There are various types in use: One of these is the drum sieve (see figure 2.4). This sieve is used for example for chicken slaughterhouses to remove the feathers and offal. A drum sieve consists of a slow rotating drum that is equipped with small perforations. The drum is driven through a gear box by an electric motor. The wastewater to be treated is brought inside the drum and pushed back out through the perforated casing. The sieved particles stay in the drum and through both the revolving movement of the drum and the internal screw are placed at the end of the sieve screen and then cast out. More small particles from industrial wastewater can be removed from wastewater by sleves than by installing screens.

In some less industrialized countries water treatment consists of a few screens and a drum sieve, after that the effluent is drained into the surface water. Since such a sieve is comprised of very fine pores from 0.1-0.5 mm the largest part of the suspended solids is removed and with that also a significant part of the polluting organic substances.

2.1.5 Comminuting Screens

A comminuting screen is a combination of a screen and a cutting machine, where the refuge is cut finely under water (see figure 2.5). A comminuting screen is, as it were, a drum with horizontal openings that are usually between 8-15 mm wide. These steel cylindrical drums are rotated by an electric motor and turn on a vertical axis. The wastewater flows from outside the drum inside through the openings, which have been installed on the drum wall, and is discharged



A teeth ridge
B cutting tooth
C opening for effluent
D motor
E influent
F effluent

Figure 2.5 - Comminuting screen

underneath. On the outside of the drum there are a few knives made of hard steel, which fit inside the openings between the prongs of a hard steel stationery comb. The held back debris is pushed up by the water pressure against the turning drum, and is cut finely between the comb and the knives and is then discharged from the drum through the slits.

This method is hygienic, and keeping the screen debris under water makes for little opportunity of spreading foul odours. One advantage is also that the installation can be limited in terms of size. To the comminuting screens' disadvantage is that floating debris, such as balls, corks, and plastic objects, are not easily grabbed by the cutting machine. In addition, the coarse materials are not removed from the water but cut; further along within the installation the diverse particles (plastic, hairs, dust e.g.) can cling together (web) and still create blockages.

Currently, in the Netherlands comminuting screens are no longer being installed. Removing coarse objects is the preferred method.

2.1.6 Treatment of Screenings

The amount of screenings, or screen debris, that is kept back through a fine bar screen or a sieve depends on the nature and the composition of the wastewater and as a result can vary greatly. And particularly in heavy rainfall the amount will increase significantly. The amount of screen debris amounts to circa 50 litres per 1,000 inhabitants per day.

The collection and the way in which the screen debris is removed are very important. With screens and sieve installations the screen debris is mostly dewatered in a press that is developed for that purpose, and is then put into a closed container and removed. The leak water and flush water are returned to the sewer. The screen debris can also, after thickening, be packed in artificial foil and brought to a dump or incineration plant.

An important aspect of the processing of screen debris is the avoidance of odours. That is why the screens and the transportation of the screen debris is often covered or stored in a separate building. The entire installation is equipped with a ventilation system that purifies the extracted air.

2.2 Grit Chambers

For more information see ME Chapter 5-6

2.2.1 General

There are various reasons why sand and grit must be removed from wastewater:

- to extend the lifespan of the mechanical components, especially pumps;
- to prevent sand and grit from getting into the pipelines and machinery, which can cause blockages.
- to avoid depositing a sand package at the bottom of the digestion tank, the presence of which would minimize the effective volume and hence the efficiency of the tank.
-

In a grit chamber one tries, through selective sedimentation, to remove grit and similar mineral material with a grain diameter of > 0.15 mm; while from a hygienic standpoint, in relation to the disposal or dumping of the removed sand, the putrescible material in the wastewater should stay behind. The amount of sand, which is delivered to a WWTP, varies depending on the circumstances within the sewer system and is generally in the range of 5 to 12 l/inhabitant per year.

The placement of a grit chamber is preferably completely at the beginning of the purification process. When it is installed in an influent pumping station, the grit chamber is, for technical and practical reasons in general situated, on the surface and often after the screens.

In a plant with sludge digestiona grit chamber can be placed placed in the sludge line, for example between the primary treatment and the sludge thickening or between the sludge thickening and the sludge digestion. As a result of this one can save costs on the size of the tank; in fact the amount of sludge that needs to be treated is noticeably less than the total wastewater flow (see figure 2.6).

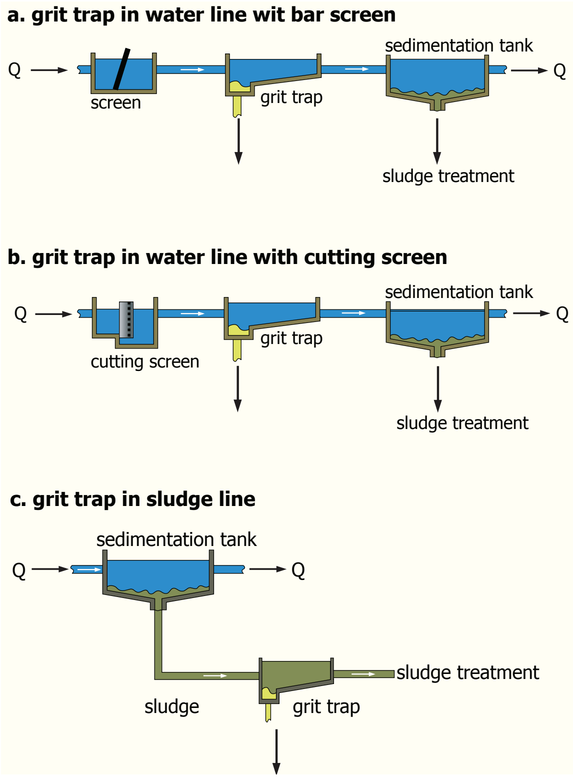


Figure 2.6 – Grit chamber in the water line (a) and in the sludge line (b)

2.2.2 Dimensioning

Sand particles behave as discrete particles during sedimentation. By setting the surface load (v_o) of the grit chambers equal to the settling velocity of the sand particles (v_b) a successful sedimentation is realized. Q/A is also called surface load; the depth only plays a limited role.

$$v_o = \frac{Q\ (\text{maximum flow})}{A\ (\text{surface of the grit chamber})}$$
$$= v_b\ (\text{settling velocity sand particle})$$

2.2.3 Rectangular Horizontal Flow Grit Chamber

The rectangular horizontal flow grit chamber (see figure 2.7) is essentially a long channel that is made up of a rectangular spillway. This type of grit chamber is suitable for one type of hydraulic load. That is why various channels are built parallel to each other, so that – depending on the amount of supplied sewer water- one or more channels can be put to use. The settled sand is removed

Figure 2.7 - Rectangular horizontal flow grit chamber

and dumped. In removing the sand centrifugal pumps are used in the appropriate construction. The pumps are stationed on a wheeled car that is placed lengthwise above the grit chamber. The sand is pumped into a tank; here a washing process occurs which separates the sludge particles from the grains of sand to which it is clinging. The organic compounds are transported back through an overflow to the grit chamber or to the inlet pipes. The sand can be carried off using a container (see figure 2.8).

The dimensioning of a rectangular horizontal flow grit chamber is determined by the maximum surface load of 40 $m^3/(m^2 \cdot h)$; at the same time the horizontal flow velocity must be 0.30 m/s.

However even the (larger) sludge particles in this surface load will settle, these particles will, as a result of the horizontal velocity, be spread over the bottom and will be rinsed out while the sand particles stay behind. To ensure a stable velocity the length/width in proportion to 10:1 to15:1 is taken.

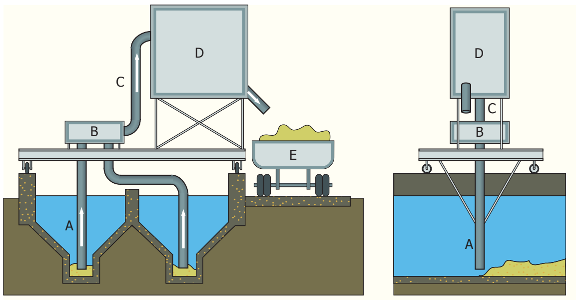


Figure 2.8 – Square horizontal flow grit chamber with air lift

2.2.4 Square Horizontal Flow or Dorr Grit Chamber

The Dorr-grit chamber is comprised of a square bin measuring circa 1 m deep. This grit chamber has a flat, circular bottom where scrapers rotate (see figure 2.9). The wastewater is delivered on one side (D) equally distributed through adjustable baffles (H). Hereafter the water flows trough the tank lengthwise and is discharged via an overflow on the opposite side. The slow rotating two armed scrapers (G) transport the sand that is deposited into a drain on the side wall. Afterwards the sand flows along with the

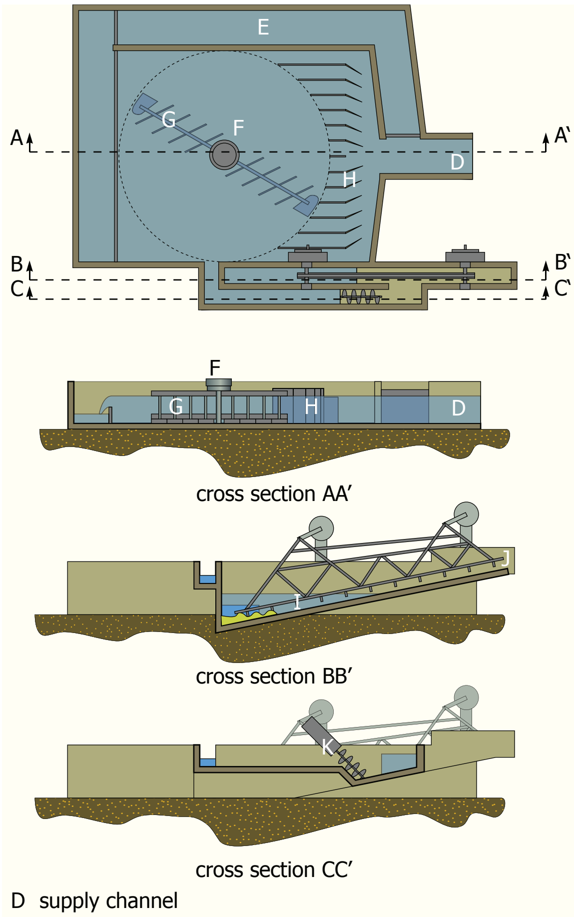


Figure 2.9 – Dorr grit chamber with mechanical grit removal and grit washing equipment (Dorr-Oliver)

rinse water into a steep flushing cistern (BB') where a mechanical washing installation is installed. This washing can take place in a co-current or counterflow. Using more or less rinse water creates control options. The speed and power of this washing process can be influenced by changing the gear or installing baffles.

During changing hydraulic loads, the Dorr-grit chamber can be put to use by means of splitting between the sedimentation and washing of the settled sludge. If the surface load is so low that even the sludge particles settle, these will get washed out from the sand with the help of the washing installation.
Inlet effects and outlet effects and failures through cleaning the sand are taken into account during the design stage. The surface load (based on the square bottom) is kept at 30 $m^3/(m^2 \cdot h)$ with maximum supply. For a practical depth 0.8 to1.0 m is maintained.

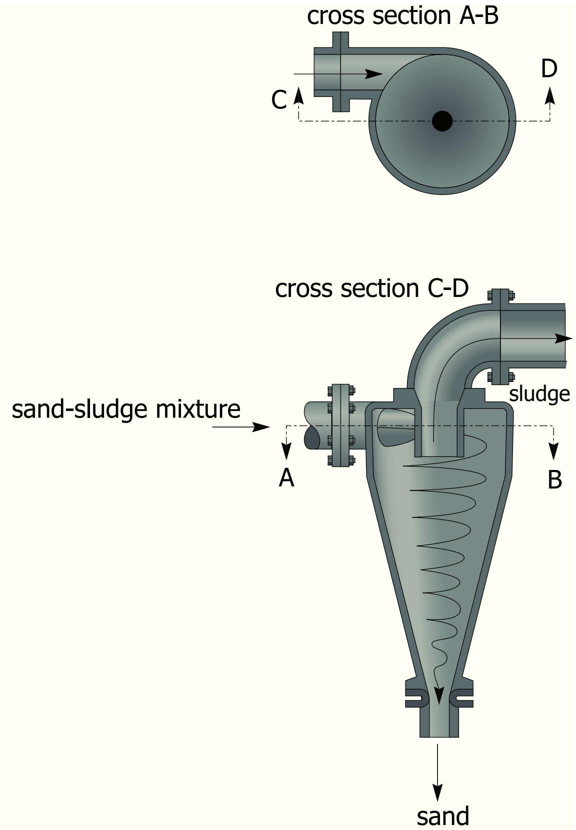


Figure 2.10 - Hydrocyclone

2.2.5 Hydrocyclone

The hydrocyclone is used for separating grit from more concentrated sludge flows, for example between the primary settling tank and the sludge thickening tank or the digestion tank. The hydrocyclone is a cylindrical mechanism with a conically tapering base. The supply of the sludge-grit-water mixture comes in tangentially, that is to say along the circumference (see figure 2.10).

Through this tangential supply, which is separated under a gauge pressure of 5 to 10 atmospheres (circa 500 – 1000 kPa), the cyclone is created and a spiral flow in which the heavier particles through the centrifugal flow are thrown out to the outer rim and by means of the cone wall slide down and are removed at the bottom (underflow). The lighter sludge particles come together at the centre and top of the cone with most of the water by means of an upward flow where they exit through a pipe. By means of a washing installation organic particles can, from the underflow, be separated from the grit and returned to the influent at the installation. Design occurs on the basis of practical experience and rules.

2.3 Primary Sedimentation

For more information see ME Chapter 5-7

2.3.1 Introduction

Usually the primary sedimentation tank comes after the grit chamber. Here as many of the settleable undissolved particles as possible are separated. This sludge is called primary sludge. In some plants (oxidation ditch types) where there is no primary sedimentation tank installed, the undissolved particles are caught in the activated sludge and are stabilized there.

Grit particles are mostly referred to as granular particles that settle unchanged (discreet sedimentation). It is also well known that during the sedimentation process the settling of particles become larger, as a result of particle agglomeration or flocculation (flocculent sedimentation). This process can be stimulated by stirring, adding chemicals and biological agents. Flocculent sedimentation occurs with higher concentrations of undissolved particles; this is certainly the case with primary settling.

2.3.2 Sedimentation in Practice

Different varying factors influence the sedimentation process such as those that occur in practice. The efficiency of the sedimentation depends on:

- the size and the form of the particles; the larger the particle size, the faster sedimentation occurs;
- the density of the particles; if the difference between the particle density and the carrier liquid is larger, then the sedimentation process is faster;
- the composition of the suspension;
- the concentration of the suspension; the larger the concentration , the larger the sedimentation process efficiency.
- the suitability of the particles to flocculation;
- the temperature; with higher temperatures, the viscosity of the liquids is reduced and thus particles settle faster;

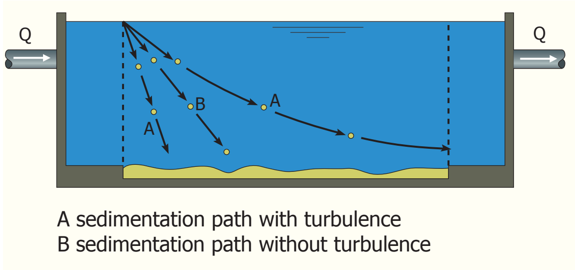


Figure 2.11 - Disturbance by turbulence

- the depth and form of the sedimentation reservoir;
- the distance that the liquid travels through the reservoir;
- the velocity of the liquid;
- the influence of the wind on the liquid's surface;
- the way in which the liquid flows into the reservoir and is removed;
- the occurrence of short-circuits, resulting from the fact that the liquid has already, however small, a difference in density.

Disturbance through Turbulent Flow

In contradiction to sedimentation in an ideal reservoir, in practice sedimentation is influenced by turbulence in the tank. When turbulence occurs a few particles will swirl up and most likely

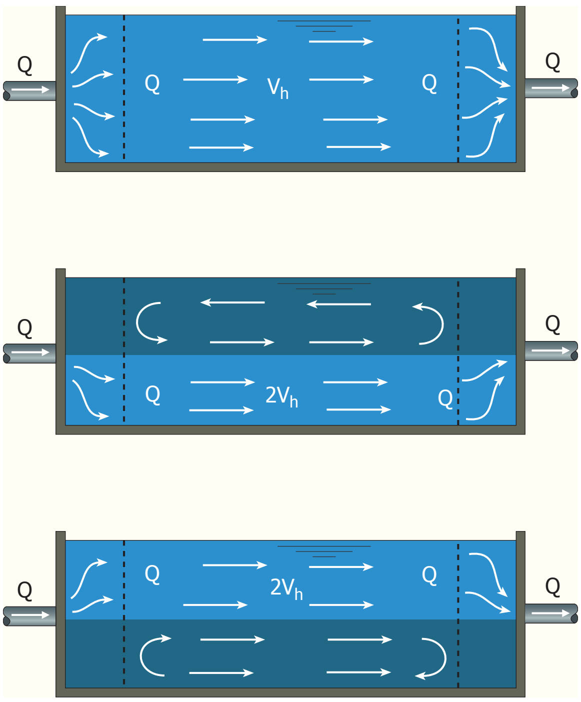


Figure 2.12 - Disturbance by short circuit flows

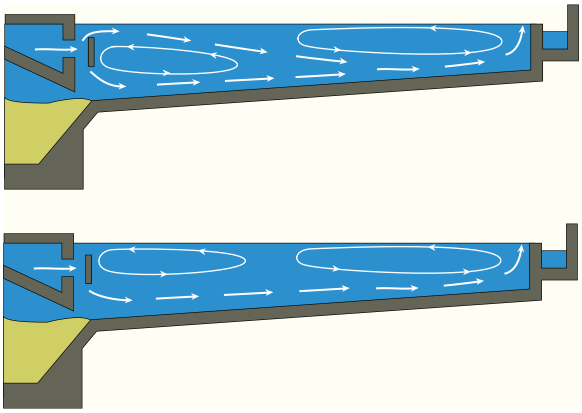


Figure 2.13 - Examples of short circuit flows

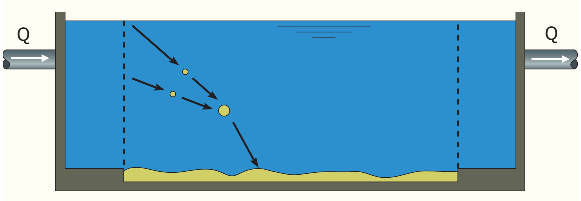


Figure 2.14 - Flocculation

be removed with the effluent and a few particles will reach the sludge zone earlier. The path that a particle takes will no longer be straight but curved, as is schematically shown in figure 2.11. Due to the turbulence the efficiency of the sedimentation process is disadvantageously influenced.
The state of flow is characterized by Reynolds number, that should be lower than 2000 to have laminar flow.

Disturbance through Short CircuitFlows

When short-circuit flows occur in a sedimentation tank, dead zones and eddies occur, the average residence time is shorter than in theory. This is, for example, the case if there is a difference in the liquid density in the reservoir and that which is delivered. Figure 2.12 shows that the residence time can be noticeably shorter than is theoretically expected.

To limit the short circuit flows as much as possible, a good distribution of the incoming and outgoing liquid is necessary as well as a stable flow pattern. A stable flow is understood to be the recovery of the original state of flow after the occurrence of a disturbance. The stability of a flow is higher as the ratio between the force of inertia and gravity is bigger. As the flow becomes more stable, the distribution of the velocity over the cross-section is more constant and the flow will, after the occurrence of a disturbance, recover faster returning to its original state. The ratio between the force of inertia and gravity is reflected in the dimensionless Froude number that should be higher than 10^{-5}.

Disturbance through Scouring

In an ideal sedimentation tank the efficiency of the sedimentation process depends only on the surface load v_o = Q/(B·L) and is therefore independent of the depth of the tank. As the depth becomes more shallow, the horizontal velocity v_h = Q/(B·H) increases, and at a certain time this becomes so large that all the settled material is withdrawn and removed.
Usually the whirling up of the settled particles is prevented through ensuring that the horizontal velocity v_h does not exceed the critical velocity v_s. This v_s, the slip velocity, is 0.30 m/s for grit particles; for primary sludge 0.03 m/s and for activated-sludge 0.02 m/s.

Flocculation

As a result of the velocity gradients two flocculent particles can meet each other and join to become one larger particle, causing the sedimentation velocity to be larger than both the components (see figure 2.14). Flocculation sedimentation is promoted through turbulence.
Apart from disturbances, the surface load does not determine the efficiency alone, the residence time is an important influence in flocculation and thus the depth of the sedimentation reservoir plays a role as well.

2.3.3 Design Aspects

The most important design criteria for sedimentation tanks is the set surface load. In general 1.5 – 2.5 $m^3/(m^2·h)$ is assumed as for maximum load. If this maximum load has little or no effect on the influent flow pattern, then higher values (up to 4.0 $m^3/(m^2·h)$) are used. As a residence time a minimum value of 1 to 1.5 per hour is kept.

The optimal depth is between 1.5 m (small tanks) and 2.5 m (large tanks). With rectangular tanks the ratio depth/length must be approx. 1:20; the width/length ratio must be minimum 1:4 and preferably 1:5 to 1:6.

In terms of the minimum and maximum tank sizes the following is recommended:

- **round tanks**
  diameter minimum 20 m
  maximum 60 m
  optimal 30-40 m
  with smaller tanks (D<30 m) the efficiency is reduced due to interference by influent flow and discharge
  depth 1.5 - 2.5 m
  bottom slope 1:10 to 1:12

- **rectangular tanks**
  length maximum 90 m
  optimal 30-50 m
  width 5-12 m usually 5 à 6 m
  depth 1.5 - 2.5 m
  bottom slope 1:10 to 1:12

The wastewater discharge from a sedimentation tank usually takes place through one or more effluent weir troughs. When leaving the tank the liquid accalerates. In order not to danger the stability of the flow in the tank, the discharge must be evenly separated and the so-called Weir-overflow rate may not exceed certain values. The Weir-overflow rate is the discharge of the effluent (m^3) per unit (of length of the effluent weir troughs/ (m) per unit of time (h). If the Weir-overflow rate is too high, light material can be discharged with the overflow water from the tank. As a maximum acceptable value the Weir-overflow rate of the primary settling tanks is kept at 10 to15 $m^3/(m \cdot h)$.

At the same time the average velocity of the sludge scrapers must be sufficiently low; in any case the scraper must not cause any churning up of the previously settled sludge. Therefore a peripheral velocity for the sludge scraper on round tanks of max. 0.06 to 0.07 m/s is advised, and for a velocity for rectangular tanks approx. 0.03 m/s.

The discharge of the settled sludge usually takes place in a continuous or semi-continuous way; from the collection channel or funnels the sludge is in large amounts pumped regularly into the (primary) sludge thickeners; building up of the sludge in the primary tank must be avoided (malodour, rotting etc.).

2.3.4 Round Tanks

With round sedimentation tanks the wastewater is fed into the middle and is discharged through a trough on the outer periphery. In figure 2.15 a cross-section drawn. Above the tank there is a bridge that slowly rotates with sludge scrapers attached, these move the sludge on the bottom

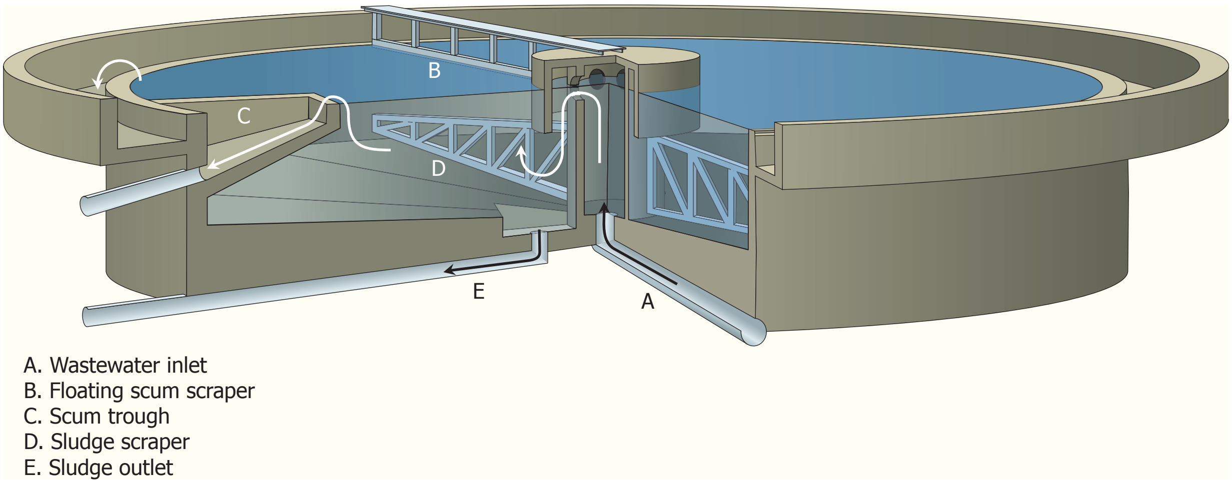


Figure 2.15 - Settling reservoir with a circular layout with sludge scraper and floating scum scraper

slowly towards the central sludge funnel, where it is removed. The bottom is built with a slight slope. For the effluent trough there is a scumboard or baffle for holding back the floating solids. The floating solids (usually fat) are pushed into a scum trough by the floating scum scrapers that are attached to the bridge.

An important aspect is the delivery of the wastewater into the tank. The inflow energy must be destroyed as much and as quickly as possible to prevent swirling (of settled solids). Along with that it must be taken into consideration that the entering liquid should have at least a rate of approx. 1 m/s, which must be reduced to less than 2 to 3 cm/s. For this purpose feedwells and flow splitter constructions are installed.

2.3.5 Rectangular Tanks

Rectangular sedimentation installations have a basin with flat or slightly sloping bottoms, equipped with an incoming and outgoing construction. In addition a scumboard is installed to keep back the floating scum (see figure 2.16).

The sludge that is settled on the bottom is pushed into a sludge funnel that is on the intake side. Once put under hydrostatic pressure it is then removed through a pipe attached to the funnel. Of course the sludge can also be removed with a pump.
For the rectangular primary settling tanks, chain scrapers are usually installed for removing the sludge from the bottom of the tank. These consist of a few parallel slide blades with a blade length equal to the width of the basin. The slide blades are attached on two infinite transport chains.

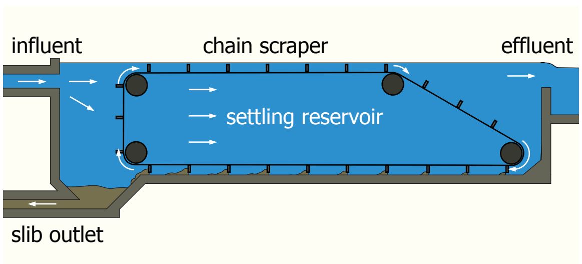


Figure 2.16 - Rectangular settling installation

Both transport chains go over two chain wheels that are mounted under water at the end of the basin. One of the chain wheels is driven through a transmission by an electric motor. The slide blades are dragged along the basin bottom by the transport chains in the opposite direction of the wastewater flow direction and push the sludge continuously into the sludge funnel on the delivery side of the basin. In the returning motion the transport chains usually lie above the water surface of the basin. Some chain scrapers, by means of the slide blades' returning movement, bring any present floating scum to the direction of the basin outlet, where the floating scum is held back by the scumboard. The floating scum is removed by a tilting mechanism.

3 Secondary Treatment

3. Secondary Treatment

For fundamentals of Biological treatment see ME Chapter 7

3.1 Trickling filters

For more information see ME Chapter 9-2

3.1.1 General

In figure 3.1 the construction of a trickling filter is depicted. The trickling filter bed consists of a cylindrical tank 2 – 4 meters high equipped with a perforated bottom. The trickling filter is filled almost entirely with packing material made up of lava slag, gravel or other suitable material. A rotary distributor arm spreads the influent wastewater over the upper layer of the media and then the wastewater trickles down over and in between the packing material through the filter. Through the underdrain and the drainage collection trench the treated water is discharged into the clarifier.

3.1.2 Biofilm

The so-called biological skin, also known as "biofilm" has developed on the surface of the packing material of a stabilised trickling filter. The biofilm consists of an aerobic layer and an anaerobic layer (see figure 3.2). The biofilm is made up of a slimy substance in and on which, apart from bacteria and other organisms, the residues from lysed cells and

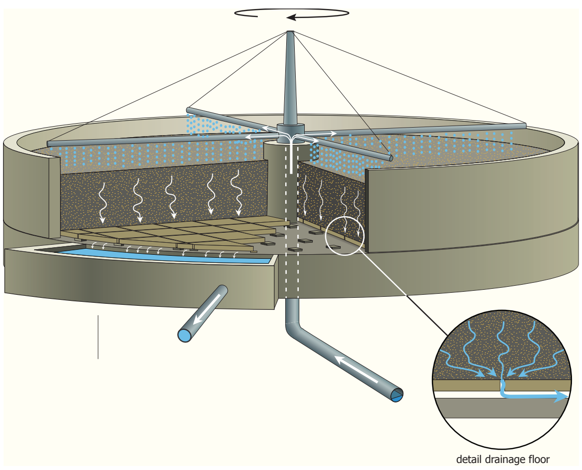


Figure 3.1 - Trickling filter

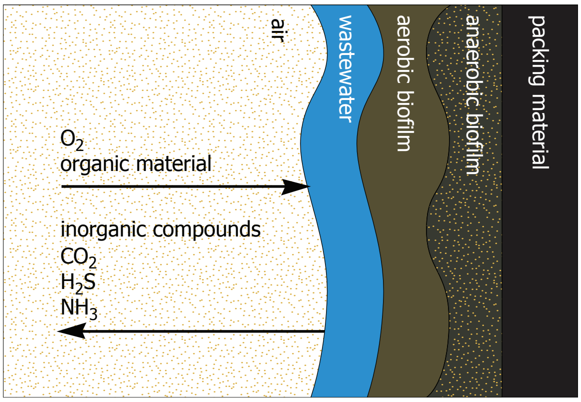


Figure 3.2 - Biofilm on the packing material in a trickling filter

other non-biodegradable or slowly biodegradable compounds like humic substances occur. Protozoa, which consume bacteria, also live in this bacterial biofilm. Furthermore in the packing material algae, larva and larger worms (tubifex) are found. Deeper down in the filter flagellates and protozoa dominate the bacteria.

The settled wastewater flows in thin layers over the aerobic layer of the biofilm. From the passing liquid absorption of the dissolved solids, present in the settled wastewater, takes place in the biofilm, whereas through the liquid film the biodegradable substances are formed by hydrolysis and oxidation and from organic compounds, which were absorbed earlier, are discharged. For the oxidation of the organic compounds in the aerobic layer oxygen that is to be found in the airflow and is pulled through the trickling filter is necessary. The oxidation product CO_2 disappears both with the water discharged from the trickling filter and the discharged air.

The biofilm growth takes place namely where the wastewater comes in contact with the biofilm and thus where the concentration of organic compounds is highest. This explains why the need for oxygen is the greatest at the top layers of the biofilm and that the remaining oxygen that passes through to the deeper layers is limited. This means that the aerobic layer cannot be thicker than 1-3 mm and the layer just below is anaerobic.

Table 3.1 - Characteristics of trickling filters with lava filling (slag) dependant on the load

Low-loaded trickling filters	**Moderately loaded trickling filters**	**Highly loaded trickling filters**
BOD load 0.1-0.2 kg BOD/m^3.d)	BOD load: 0.3-0.5 kg BOD/(m^3.d)	BOD load: 0.7-1.0 kg BOD/(m^3.d)
Hydr. Load: <0.2 m^3/(m^2•h)	Hydr. Load: 0.4-0.8 m^3/(m^2•h)	Hydr. Load: 0.7-1.5 m^3/(m^2•h)
Oxidation of BOD and N-Kj and partial mineralization of sludge	Oxidation of the BOD and limited oxidation of N-Kj	Unquestionable oxidation of the BOD
Mineralised film lets go by itself. For enough nitrogen removal and sludge mineralisation the hydraulic load must be small, thus having no recirculation.	In order to prevent excessive sludge growth resulting in blockages there must be as a result of recirculation a large hydraulic load installed, however not too high to disturb the slowly nitrifying bacteria.	High hydraulic load, thus a thin biofilm. With even higher bio loads there is a large chance of creating a blockage.

The start up of a trickling filter lasts approximately 2 months in the summer, this is faster than in winter because under low temperature conditions higher organisms, such as larvae and worms, cannot grow. The best time to start up a trickling filter is therefore in spring.

In a trickling filter the composition of the flora and fauna change with the depth of the filter. In the upper layer, where the nourishment is greatest, many (heterotrophic) bacteria and protozoa as well as higher organisms start to develop. Towards the bottom the bacteria and protozoa become dominant. Eventually nitrification takes place in the bottom layer where the (autotrophic) nitrifying bacteria develop as the organic fraction is already largely decomposed and since enough oxygen is present. In each zone a complete and specialised flora of micro-organisms can develop as a result of the prevalent environmental conditions.

The growth of the biofilm on the packing material continues up until disposal takes place as a result of:

- the spraying function of the supplied water
- the activity of higher organisms
- the development of gas in the under laying anaerobic layers
- endogenous digestion of the bacterial mass.

As a result of this, a partial of the film residue, the sludge, can occur underneath in the filter. The sludge that is reasonably mineralized or stabilized both aerobically and anaerobically, is referred to as humic sludge

3.1.3 Packing Material

The biofilm where the degradation of the oxidisable compounds in the wastewater takes place must be able to form itself on the trickling filter's present filling material.

For this the following requirements are set:

- good attachment possibilities for the biofilm.
- even distribution of the air and water flows
- large attachable surface per volume-unit
- enough large free void space for the discharge of sprayed biofilm
- the material may not interfere with the growth of the micro-organisms
- the material may not be crushed or show any of abrasion
- chemically and biologically inert to corrosion from the wastewater, micro organisms and secretions.

The most important concepts to characterise packing materials are:

- the specific surface area, the surface availability for growth per volume-unit in m^2/m^3
- the porosity, or the void space in %
- the specific weight of the material in kg/m^3, both in unused and used form
- material characteristics such as:
  - type (smooth or porous)
  - grain size (diameter)
  - uniformity (as even as possible)
  - shape of the fragments

In the Netherlands usually lava is used as a filling material because of the low costs and rough porous structure in which a good attachment and an intense contact between the sludge, air and water is obtained. The size of the lava bits must be a compromise between on the one hand allowing a large void space to prevent clogging up (thus large fragments) and on the other and a large specific surface area, a homogenous distribution of air and water over a long contact time (or small fragments). In general the fragments are 5-8 cm big. The specific surface area is therefore approximately 80- 100 m^2/m^3 and the void space is approximately 40%.

Over the last 20 years more and more artificial packing materials are being used, that are currently being fabricated in many various forms. The advantages are a low specific weight (including biofilm and adhered water 250-400 kg/m^3 compared to 2000 kg/m^3 for lava slag), the size of the void space (90-98%) and the size of the specific surface area (100-230 m^2/m^3). As a result of this lighter constructions can be installed. Furthermore taller trickling filters are being installed (up to 10 m high), which makes the installation more compact, the contact time is greater and the rinsing function is larger. Preferably packing materials are installed with continuous reaction surfaces or random fill rings with a large open structure (see figure 3.3).

3.1.4 Construction Aspects

Walls

The walls of the bed are preferably closed, so that a chimney effect will occur as a result of the difference in temperature in and outside the bed. Furthermore the walls support the packing material and there is less chance of freezing on the periphery of the bed. At the bottom of the wall there must be openings to allow air supply and discharge. The total surface of the openings must be 0.5-1% of the surface floor depending on the type of filter.

Floor

The floor consists of two parts, namely a perforated bottom of prefabricated elements which supports the packing material and a lower lying closed floor that supports the perforated floor and allows for the effluent to be discharged. The collected surface of the perforations must be approximately 15% of the surface floor.

Rotary Distributor

Distributing the water evenly over the surface occurs with the help of fixed distributor arms, rotary distributors for circular tanks, on rolling distributors for rectangular tanks. The movement of the distributors is activated by the excess pressure of the water according to the principle of the Segner wheel. To create this an extra amount of pumped wastewater pressure of 0.6-0.8 m is required. Distributors can also be mechanically

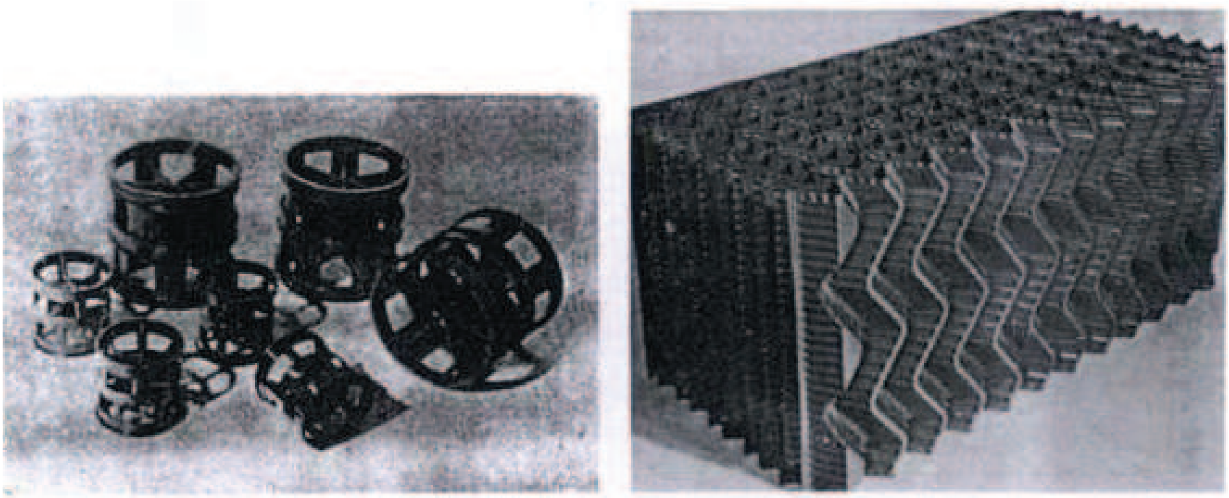


Figure 3.3 - Artificial packing material for trickling filters

powered, and therefore need no extra pump pressure.

Distributing Wastewater

The wastewater is pumped through the influent pipe under the floor of the trickling filter, is raised through a riser pipe in the centre of the bed, and ends up in the distributors. The arms are furnished on one side with small drilled holes allowing the water to flow out. To create an even distribution both the holes must be as small as possible and the amount of holes must also be as large as possible. To prevent blockages the holes must also not be too small. For this reason modern filtering distributors are constructed with holes with a diameter size of 1-1½ cm. Each waterjet flows over a little sheet or table creating a water film in order to obtain a good water distribution and aeration.

In terms of water distribution various factors play a role: such as the resistance losses in the distribution holes, the resistance losses through the pipes, the centripetal (centrifugal) force (that is directly dependant on the rotary velocity and is thus indirectly a function on the exit velocities and the distribution as well as the size of the distribution holes along the length of the arm.

An important aspect is also the variation in the hydraulic supply to the trickling filter. Sometimes the distribution, which functions well with high supply, with low supply becomes noticeably uneven.

Sizes

For construction reasons (particularly in regard to the rotary distributors) the diameter of a trickling filter can be maximum approx. 40-45 m. The height of the packing material is minimum 1.5 m but often around 2 - 2.5 m is kept.

Preliminary Treatment

In order to prevent blockages in the trickling filter the wastewater must be treated previously having removed large particles, sand, fat and settleable solids.

3.1.5 Oxygen Supply

The aerobic degradation processes require sufficient supply of air-oxygen. That is why there needs to be enough air coming in through the upper surface and through the underdrain. The oxygen supply is not a problem in precisely constructed trickling filters as a result of the air circulation (natural ventilation) in the trickling filter. This air circulation can occur in two directions:

a. Upward flow (from under towards the top)
   In the winter the wastewater and the contents within the trickling filter are warmer than the air outside. The air in the trickling filter is warmed up and is raised up (chimney effect).

b. Downward flow
   In summer, the wastewater is cooler than its surroundings and therefore also the content of the filter. The air in the oxidation bed is cooled down and sinks, since cold air is "heavier" than warm air.

Moreover, in the summer the daily flow direction of the air can change:
- at night from under towards the top (effect a)
- during the day from the top to the bottom (effect b).

With a temperature difference of 4ºC an airstream of 18 m^3/m^2 h already occurs, where 10-20 times more oxygen is present than the oxygen requirements from the supplied wastewater. It must be taken into consideration that the (actual) oxygen transfer from this airstream as rule amounts to 10% only.
Stagnation in the oxygen supply from the filter occurs when there is a temperature difference between water and air of approximately 1.9ºC. This situation occurs usually in spring and autumn (when the wastewater temperature and outside temperature is about the same) and this often causes the 'deterioration' of the filter resulting from the anaerobic conditions in the filter.

Recently built trickling filters are usually covered because of odour emissions, resulting from spraying the wastewater. In this case the filter is preferably ventilated from top to bottom. The advantage of this is that the odorous substances that are mostly released during the spraying process can be decomposed in the filter, as a result of the biological activity.

3.1.6 BOD-Load

The operation of a trickling filter is largely determined through the amount of degradable organic material that is allowed on the trickling filter, given the BOD-(volume) load in kg BOD/$m^3 \cdot d$. In formula form:

$$B = \frac{Bd}{V}$$

where

B = volume load or BOD-load (in kg BOD/($m^3 \cdot d$))

Bd = BOD-supply in kg BOD/d, also known as BOD load

V = volume trickling filter (m^3)

The treatment performances in general are reduced if the BOD-load increases; also the (non) occurrence of nitrification (transference of ammonium nitrate) is greatly dependent on the BOD-load.

3.1.7 Rinsing Effect

In order to have an optimum removal of the excess biofilm a well functioning rinsing effect in the trickling filter is very important. On the one hand blockages must be prevented and on the other hand there must be a thick enough biofilm created. The rinsing effect is largely determined through the surface loading rate v_o; this is calculated through the influent flow divided by the surface.

$$v_o = \frac{Q}{A}$$

where

v_o = surface load/surface loading rate in $m^3/(m^2 \cdot h)$

Q = influent flow m^3/h

A = surface trickling filter

For low-loaded trickling filters a surface loading rate is kept at 0.05 - 0.3 $m^3/(m^2 \cdot h)$; for highly loaded trickling filters 0.6 - 2 $m^3/(m^2 \cdot h)$.

3.1.8 Recirculation

In order to obtain the required surface loading rate in a trickling filter under all conditions recirculation is applied. Mostly with high loaded trickling filters a high surface loading rate (minimum 0.6 $m^3/(m^2 \cdot h)$) is important to prevent the void spaces from becoming clogged. Aside from this, recirculation in general has a few advantages, for which it is also desirable for low-loaded trickling filters to install a recirculation system. These advantages are:

- diluting of concentrated wastewater
- eliminating of peek supply and peek concentrations
- reducing the influence of biodegradable toxic substances through dilution
- longer average residence time for the wastewater in the trickling filter
- by recirculation with nitrate containing effluent the heavy odours in the primary settling tank and the trickling filter are prevented or limited, as the odorous substances (usually H_2S) are oxidised with nitrate (denitrification).

The advantages mentioned refer to the fact that applying recirculation mostly results in better effluent quality. Disadvantages of recirculation are:

- higher utilization costs as a result of the constant pumping back of effluent
- cooling off of the wastewater in the winter time, reducing the biomass activity.

The degree of recirculation is described through the recirculation factor R.

$$R = Q_R/Q$$

where

Q = the wastewater influent flow (m^3/h)

Q_R = the flow of recirculated water (m^3/h)

The supply to the trickling filter thus becomes $Q+Q_R$ or $Q \cdot (1+R)$.

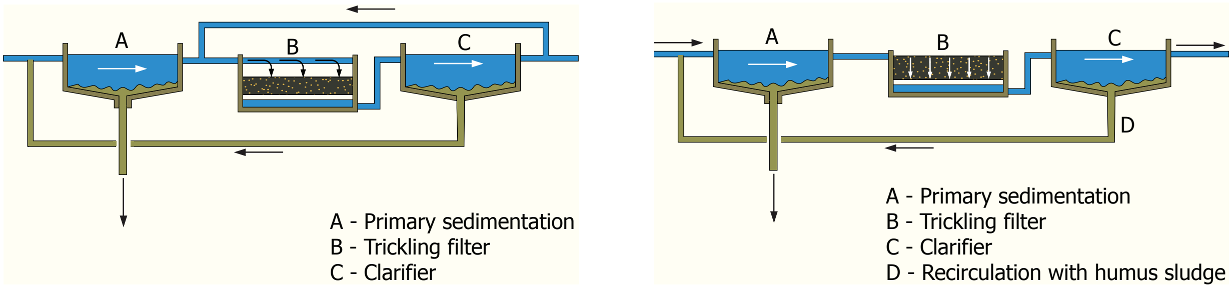


Figure 3.4 - Recirculation methods

There are two main approaches in recirculation: (see figure 3.4).

1. Effluent is added to the primary settled wastewater (end of the primary sedimentation) and the humic sludge with the incoming wastewater (inlet primary settling tank). This method requires separate pipes for the effluent and the sludge.
2. Effluent + humus sludge added together with the incoming wastewater (inlet primary settling tank). With this both the recirculated water with the settled sludge is withdrawn from underneath the final clarifier. This method leads to an extra load for the primary settling tank.

The advantage of the first method is that the total head pressure needed to raise the recirculated water is lower because it is added to the primary settling tank (lower energy costs). The advantage of the second method is that no extra sludge piping is necessary. This method has further to its advantage that the returning feed of the nitrate containing effluent will reduce the odorous emissions from the primary settling tank. In most cases the second method is preferred.
The recirculation can be controlled via the level of the supply pit using a float valve or an electric valve. The recirculation water (effluent + humic sludge) is supplied through a gravity pipe.

If there is no supply pump at the wwtp, for example in the case of wastewater being supplied through a pressurised sewer main, than it must be recirculated with a separate recirculation pump. The amount of recirculation water is usually adjusted to the amount of influent (pump mechanism/controls).
These two situations are displayed in figure 3.5.

3.1.9 Sludge Production and Sedimentation

The amount of sludge or humic acid removed with low and moderately loaded trickling filters is 0.3-0.5 kg suspended solids per kg BOD if domestic wastewater is treated. With highly loaded trickling filters higher values are found, namely 0.6-1 kg suspended solids per kg of removed BOD. In the sludge production however large differences occur during the year.

The effluent of a trickling filter must thus still be treated in a sedimentation process. The formed sludge particles in general have very

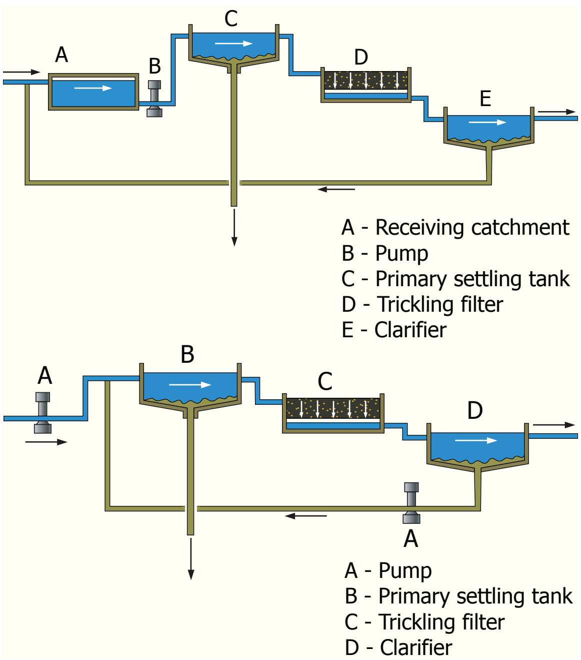


Figure 3.5 - Recirculation with influent supply through gravity sewer or pressure main

good sedimentation characteristics; this way a relatively high surface loading rate of 1.5 $m^3/(m^2·h)$ in the clarifier is applied. The flocculation characteristics of the sludge are however not optimal, which leads to fine flocs that are difficult to remove. This can be seen in the effluent from the secondary settling tanks; mostly amounting to the suspended solid content up to 20-30 mg/l.

3.1.10 Dimensioning

The size of a trickling filter is determined based on the following parameters:

- the BOD supply Bd in kg per day from the settled wastewater (kg BOD/d)
- the maximum flow of pre-settled wastewater Q_{max} in m^3, that is supplied per hour to the trickling filter (in m^3/h)
- the acceptable organic or BOD-load B in kg BOD per m^3 content of a trickling filter per day (kg BOD/(m^3·d)); this parameter determines in fact the treatment performance;
- the maximum acceptable hydraulic or surface loading rate v_{qmax} from the trickling filter, that is: the amount of wastewater in m^3 that per hour on 1 m^2 surface from the bed is allowed ($m^3/(m^2·h)$).

From these parameters the content, the surface and the height of the packing material of the trickling filter are easy to determine:

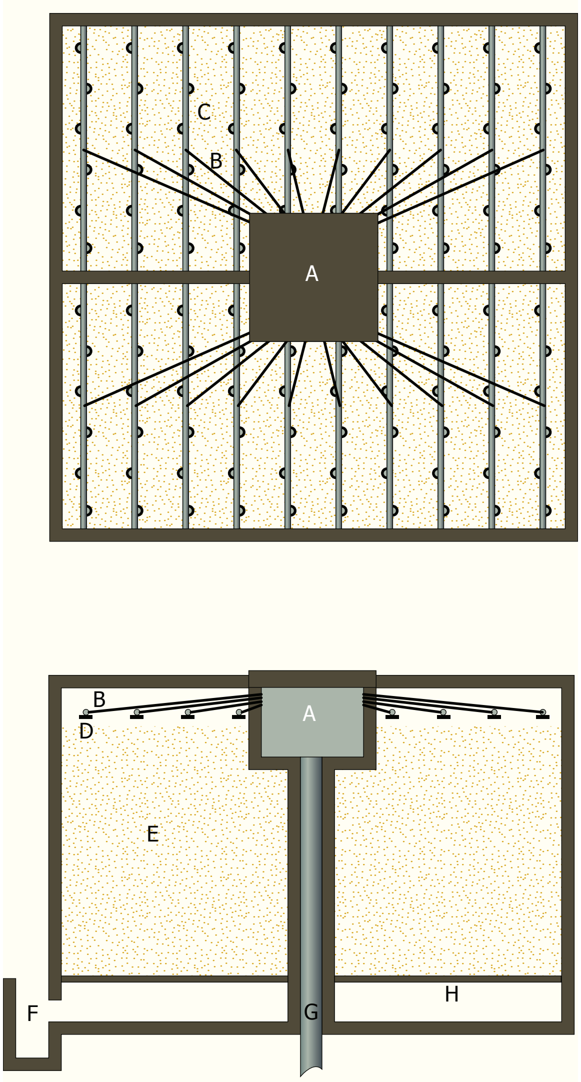
Content V in m^3:	$V = Bd / B$
Surface A in m^2:	$A = Q_{max} / v_{omax}$
Height in m:	$H = V/A = (Bd \cdot v_{omax})/(B \cdot Q_{max})$

Where H with the low-loaded beds may vary between 1.5 and 3 m, with highly loaded beds may amount up to 3 to 4 m and with artificial packing comes up to 8 to 10 m.

In order to create an efficient operation with low supply usually recirculation is installed; the amount is calculated from:

- the minimum of the average hydraulic load $v_{o\,opt}$ ($m^3/(m^2·h)$)
- the average necessary supply to the trickling filter Q_{opt} (m^3/h), where $Q_{opt} = A·v_{o\,opt}$
- the recirculation amount Q_R (m^3/h) is then $Q_R = Q_{opt} - Q$, where Q is the actual (varying) supply flow of the wastewater (m^3/h).

An example of a compact installation is given in figure 3.6.



A Distribution head
B Distribution pipes
C Spray pipes
D Spray heads
E Trickling filter
F Gutter
G Supply pipe
H Stone flooring

Figure 3.6 - Compact trickling filter installation

3.2 Activated Sludge Process

For more information see ME Chapter 8

3.2.1 General

The activated sludge process was developed between 1913-1914 in Manchester by Ardern and Lockett. They discovered that if they exposed wastewater to air for a long enough amount of time that flocs formed in the water and that the upper layer of water, after the flocs had settled, was substantially treated. When this sediment was added again to new wastewater and this mixture was aerated, it was found that the water was noticeably treated faster than if the sludge flocs were not added. These flocs were called activated sludge; they are made up of a slimy material where both bacteria and protozoa live.

This principle is used in many various forms and further developed in the technique of wastewater treatment. In general wastewater is first subjected to a preliminary treatment, this consists of removing large particles, sand, fat and settleable solids. Then the activated sludge process takes place (see figure 3.7); for this purpose the wastewater is lead into an aeration tank. In this aeration tank the wastewater is mixed with earlier made activated sludge and with the help of the aeration equipment oxygen is added. Under these conditions the activated sludge can remove both the organic as well as other whole or partial contaminates from the wastewater. After the aeration tank the wastewater/activated sludge mixture is lead to a secondary settling tank, in which the biologically activated sludge settles and separates from the treated wastewater. This activated sludge is continuously returned to the aeration tank (return activated sludge), where once more it is mixed with new wastewater. As a result of this a high concentration of biomass in the aeration tank can be maintained. The activated sludge mass increases, until a certain amount of this activated sludge must be discharged (excess sludge or surplus activated sludge).

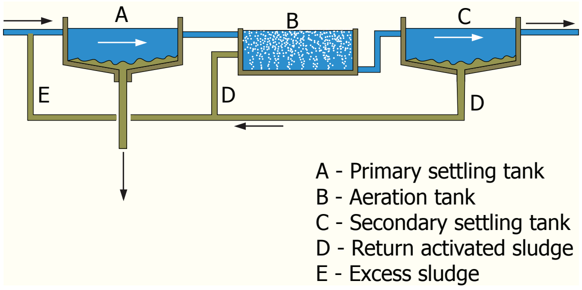


Figure 3.7 – Scheme of an activated sludge installation

3.2.2 Biological Metabolism

In the activated sludge flocs the living bacteria cells ensure that the contaminants are consumed and degraded, these lead to the (partial) treatment of the wastewater.

From the wastewater small organic molecules (with fewer than 8 to 10 C-atoms) are consumed directly through the cell walls into the bacteria. The larger components must first be split into smaller molecules by enzymes; made small enough to be able to pass through the cell walls. These enzymes are produced for that reason by the bacteria cells and excreted. Enzymes are proteins that, in very low concentrations are able to catalyse the decomposition of large molecules allowing them to be taken up by bacteria. Within the cell the so called metabolism takes place (see figure 3.8).

The metabolism comprises all the processes of components consumption by organisms, the changes that the biodegradable matter undergoes within the organism until the excretion of the degraded products. The most important processes are the direct degradation

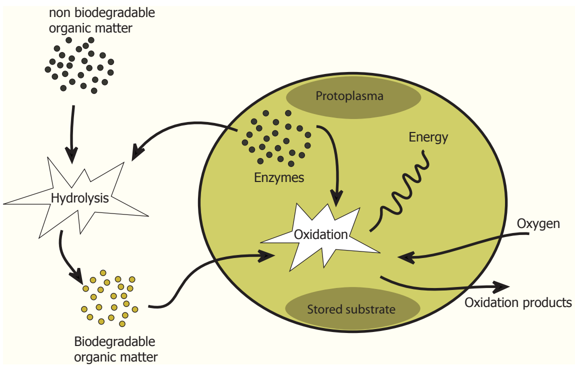


Figure 3.8 - Metabolism of the bacterial cell (extremely schematic)

or dissimilation (in which energy is released), the cell construction or assimilation (where energy is needed) and the cell-digestion or endogenous respiration. In addition to bacteria, other single cell organisms such as protozoa and flagellates develop within the activated sludge; these typically graze on bacteria. Protozoa are particularly present at a relatively low amount of nourishment for the activated sludge. The organic matter in wastewater is partly directly oxidised (substrate oxidation), partly is stored as reserve substrate and for a part used for the building of new cells. The remainder is discharged with the effluent. From the formed cell material and the stored substrate a part is oxidised through endogenous respiration; the residue forms the excess sludge.

3.2.3 Sludge Loading

The treatment performance of an activated sludge installation is to a large degree determined by the daily amount of supplied nutrition and the amount of bacteria. The amount of bacteria is difficult to quantify. As a standard the amount of biomass is therefore expressed as suspended solids (this is thus the dryweight of the undissolved particles from the activated sludge). The supplied nutrition is often expressed in the BOD-supply (Bd). The sludge loading is defined as:

$$B_X = \frac{Bd}{V \cdot X_A}$$

where

B_X = sludge loading in kg BOD/(kg ds.d)

Bd = BOD-supply in kg BOD/d

V = volume of aeration tank in m^3

X_A = sludge concentration in kg ds/m^3 (also g ds/l)

The sludge loading therefore gives an impression of the ratio between the daily supplied nutrition and the total amount of bacteria or biomass (in English literature it is referred to as F/M ratio = food to microorganisms ratio).

The sludge loading can vary from very low (approximately 0.05 kg BOD/(kg ds.d)) to very high (> 1 kg BOD/ (kg ds.d)) (see table 3.2).

The sludge loading is influenced by a few important factors and processes, such as:

- treatment efficiency (see table 3.2)
- sludge growth
- sludge age and with that the degree of stabilisation of the sludge
- nitrification and denitrification
- oxygen requirement of the sludge.

3.2.4 Activated Sludge

Sludge Amount

In the aeration tank the amount of sludge is usually constant at 3 to 5 kg ds/m^3. From the point of view of treatment performance, it should be taken into consideration to maintain the sludge concentration as high as possible, because with a certain aeration volume the sludge loading will be the lowest possible. An upper limit is made by the settleability of the activated sludge. With too high concentrations, the sludge cannot be adequately separated in the secondary settling clarifier.

A measure of the settleability of the sludge is the sludge volume index (SVI), being the sedimentation volume per one gram of activated sludge (in ml/g). It follows from this definition that the concentration of activated sludge will always be smaller than 1,000/SVI. As a result of the differences between the varying measurements in the beaker and the factual practice in an aeration tank or the clarifier, an empirical value

Table 3.2 – Sludge loading for various activated sludge systems

Type	sludge loading in kg BOD/(kg.d.d)	effluent quality BOD mg/l
ultra-low loading (ex. oxidation channel)	0,05	5
low loading activated sludge	0,10-0,40	5-15
- without nitrification	0,40	15
- with nitrification	0,10-0,25	5-10
high loading activated sludge	1 - 3	40-70

Table 3.3 – Production of active sludge per inhabitant in The Netherlands

	sludge loading BX kg BOD/(kg.ds.d)	sludge production g ds/(PE.d)
without primary settling tank	0,05	40 - 60
with primary settling tank	0,1 - 0,2	25 - 30
	0,4	30

of 1,200/SVI is taken for the maximal attainable sludge concentration.

Sludge Growth

The sludge growth is to a large degree dependent on the sludge loading. In addition to this the composition of the wastewater plays a sifnificant role. In general dissolved contaminates give a lower specific sludge production (in kg ds/kg BOD degradation) than undissolved particles. Moreover the sludge growth reduces with increasing temperatures. The dissolved oxygen concentration in the aeration tank has little influence on the sludge growth.

If no primary sedimentation is installed, there will be more undissolved contaminates in the activated sludge. As a result of the flocculating and absorbing characteristics of the sludge, inorganic particles are entrapped by the sludge. The sludge increase in this case will be larger. This is true also if chemicals (dephosphorization) are added that producing solid precipitates.
Figures from practical experience are often related to the population equivalent (PE). A few Dutch values are displayed in table 3.3.

Surplus Activated Sludge

The net sludge growth/increase will have to be removed from the system in order to maintain a constant sludge concentration in the process. This occurs by withdrawing the activated sludge; this can best be separated from the return activated sludge line where the concentration is the highest. This sludge flow is called surplus activated sludge or excess sludge. The excess sludge may be added to the ingoing water flow in the primary settling tank, so that it settles together with the primary sludge and gets treated. The excess sludge can also be thickened separately in a sludge thickener tank or by mechanical equipment.

Sludge age

As a result of the continuous growth, which gets counterbalanced by a proportional discharge of the sludge, an average residence time of the sludge occurs in the system, also known as a sludge age.

In formula form

$$\theta_X = \frac{\text{total sludge storage (kg ds)}}{\text{excess sludge (kg ds/d)}}$$

or

$$\theta_X = \frac{V \cdot X_A}{Q_S \cdot X_R}$$

where

θ_X = the sludge age (in d)
Q_S = is the excess sludge rate (in m^3/d)
X_R = the sludge concentration in the excess sludge (in kg ds/m^3)

With high loaded activated sludge installations the sludge age runs from a few hours to a few days, with a low-loaded activated sludge installation more than 10 days and with an oxidation ditch more than 25 days (see table 3.4).

The sludge age is mostly important for:

- occurence of nitrification;
- the level of mineralisation (stabilisation) from the excess sludge

Return Activated Sludge

After the treatment process in the aeration tank has taken place, the activated sludge must be separated from the wastewater. This mostly takes place by means of a gravity thickener. The sludge that is settled is returned as return activated sludge to the aeration tank (see figure 3.9). By controlling this amount, the sludge concentration in the aeration tank can be maintained at the preferred (relatively high) value.

Table 3.4 – Connection between sludge loading, sludge growth and sludge loading

sludge loading kg BOD/(kg ds.d)		sludge growth Kg ds/kg BOD-supply	sludge age (d)
0,05	oxidation ditch	0,74	27
0,10	activated sludge	0,68	15
0,15	activated sludge	0,78	9
0,20	activated sludge	0,86	6
0,50	activated sludge	0,90	2
1,0	activated sludge	0,93	1,2
2,0	activated sludge	1,0	0,5

If equilibrium occurs, then sludge that comes from the aeration tank with a concentration of X_A and a flow of $Q + Q_R$ in the primary settling tank, is returned again and supplied with a concentration X_R and a flow Q_R. Thus

$$(Q + Q_R) \cdot X_A = Q_R \cdot X_R$$

and accordingly

$$X_R = X_A \frac{Q+Q_R}{Q_R} = X_A \frac{R+1}{R}$$

where

Q = wastewater flow (m³/h)

Q_R = return activated sludge flow (m³/h)

X_A = activated sludge concentration in aeration (kg ds/m³)

X_R = return activated sludge concentration (kg ds/m³)

R = return activated sludge ratio.

From this it follows that X_R is always higher than X_A. In the secondary settling tank the sludge is concentrated to the necessary amount. However, the return activated sludge concentration cannot be higher than 1,200/SVI (see above).
For the return activated sludge ratio is

$$R = \frac{X_A}{X_R - X_A}$$

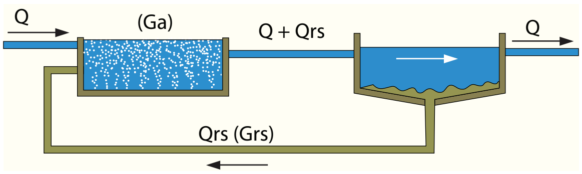


Figure 3.9 - Sludge and waterbalance over aeration tank and secondary settling tank

The minimum return activated sludge ratio is attained with a maximum value for the return activated sludge concentration, or

$$Rmin = \frac{X_A}{X_{Rmax} - X_A} \quad \text{met } X_{Rmax} = 1200/SVI$$

With most of the larger activated sludge installations the return activated sludge flow is adjustable and automatically connected to the influent-flow. A common value under dwf (dry weather flow) conditions is $Q_R/Q = 1$. With maximum pump capacity the adjustability for $Q_R = (0.5$ à $0.7)\ Q_{max}$ is common. The adjustment usually takes place in phases. Managing the adjustments takes place through the supply pumps or through the flow meter.

3.2.5 Dimensioning

There are many choices and possibilities when designing an activated sludge process; it depends mostly on the desired results. The following aspects apply:

- biological load
  - calculation of BOD-load, Bd (kg BOD/d)
- volume
  - choose sludge loading Bx (kg BOD/(kg ds.d))

    [NB in fact, this determines the effluent quality and sludge concentration X_A (kg ds/m³)]
  - calculate volume $V = Bd/(B_X \cdot X_A)$
- secondary sedimentation
  - choose/determine settled sludge quality SVI (sludge volume index) (ml/g)
  - calculate maximum supply Q_{max} (m³/h)

- choose maximum sludge loading volume $v_{s\,max}$ = 300 à 400 l/(m²·h) (zie 9.9.)
- calculate surface $A = \frac{Q_{max}}{v_{smax}} \cdot X_A \cdot SVI$

• return sludge
  - determine maximum concentration of return activated sludge X_{Rmax} (kg ds/m³) usually approximately 10 kg/m³
  - calculate $Q_R = Q \cdot \frac{X_A}{X_{Rmax} - X_A}$

• excess sludge
  - estimate the sludge growth (yield (Y))
    a. from BOD-load (Y.Bd) and Y ~ 0.5 to 0.8
    b. from design parameters (20 à 30 g ds/d per PE).

3.2.6 Oxygen Demand

The total oxygen demand in an activated sludge system is based on various factors

$OD = O_e + O_s + O_n + O_o + O_z$

where

OD = total oxygen demand (in kg O_2/d)
O_e = oxygen demand from the endogenous respiration (in kg O_2/d)
O_s = oxygen demand resulting in the substrate respiration (in kg O_2/d)
O_n = oxygen demand resulting in nitrification including denitrification (in kg O_2/d)
O_o = oxygen demand for the transformation of quickly oxydizable components such as Fe^{2+} and S^{2-} (in kg O_2/d)
O_z = discharge of dissolved oxygen with the effluent (in kg O_2/d).

Under normal conditions are O_o and O_z negligible.

If an amount of mixed liquor is continuously aerated without adding any substrate (in this case wastewater), then the oxygen consumption will decrease to a constant base level. In this situation the endogenous respiration takes place: the degradation decayed cell mass (other bacteria) for the benefit of the production of energy that is necessary for the primary vital functions (for example movement) of the cell. The endogenous respiration O_e is calculated as the product of the specific endogenous respiration b and the total mass of the activated sludge. In formula form:

$O_e = b \cdot V \cdot X_A$

where

O_e = endogenous respiration in kg O_2/d
b = specific endogenous respiration factor in kg O_2/(kg ds.d)
V = volume of aerationtank in m³
X_A = sludge amount in kg ds/m³.

The specific endogenous respiration b is dependant upon the sludge loading and the temperature; for design purposes usually b = 0.06 kg O_2/(kg ds.d) is maintained for B_x = 0.05 kg BOD/(kg ds.d) and b = 0.10 kg O_2/(kg ds.d) for higher sludge loadings.

If substrate is added to activated sludge during endogenous respiration, then the oxygen demand increases rapidly. The amount of oxygen that is consumed during the substrate degradetion depends on the amount and nature of the substrate. With the treatment of domestic wastewater these oxygen requirements are at 0.5 kg O_2 per kg of removed BOD.

For the oxidation of 1 kg ammonia nitrogen 4.57 kg O_2 is necessary stoichiometrically. (But because a part of the nitrogen gets built into the bacteria cells in reality less than 4.33 kg O_2 per kg N is necessary. The oxygen requirement O_n can then also directly be calculated if the amount of nitrogen that must be nitrified is known. For an exact calculation the oxygen equivalent for denitrification must be taken into account. This amounts to 2.86 O_2 per kg NO_3-nitrogen. For this a clear analysis and insight into the nitrogen balance for an activated sludge system is required. (Chapter 4)

3.2.7 Aeration Systems

Functions

The aeration of activated sludge and wastewater in an aeration tank has two functions:

- addition of the required (air) oxygen to treat the wastewater
- the ensuring for enough turbulence (velocity) so that the sludge stays in contact with the wastewater.

There are also systems in which the functions are split. The aeration system supplies the necessary oxygen, while a separate mechanical system (agitator/propeller) ensures that the sludge is kept in suspension.

The following criteria are employed to meet the necessary mixing requirements:

a. energy input or energy-density (W/m^3); depending on the shape of the tank 2 to 10 W/m^3 may be required for sufficient mixing;
b. flow rate (m/s); to prevent the sludge from settling: it is often required that the entire tank (including very close to the bottom) a minimum velocity of 0.20 m/s occurs.
c. air-loading ($m^3/m^2.h$); this is the amount of air that per surface aeration system can be added to the mixed liquor (only fine bubble aeration) In practice the value for fine bubble aeration is often kept at a minimum of 2 $m^3/(m^2.h)$.

The following factors are important for the choice of type and operation of aeration systems: energy costs, operational safety, maintenance and adjustability.

On the whole there are two main systems for aeration, namely the surface aeration system and fine bubble aeration.

Micro Scale

Even on the micro scale there must be enough turbulence present, so that the dissolved oxygen can be transfered to the sludge flocs. Turbulence on the micro scale is realised by using macro turbulence.
Depending on the turbulence, the sludge flocs will have an oxygen rich boundary layer where the biochemical conversions can occur quickly, and under which another biomass layer exists that has less or no oxygen and where other processes can take place. The turbulence can also influence the size of the flocs and thus influence the settling characteristics.

Oxygen Supply Capacity

The oxygen supply capacity OC (oxygenation capacity) of an aeration system is defined as the amount of oxygen that this system can bring in per hour in 1 m^3 oxygen free clean water with a temperature of 10˚C and under an atmosphericpressure of 101.3 kPa; the OC is expressed in kg $O_2/m^3.h$ and is therefore a characteristic quality of a specific aeration system or device.
The oxygen supply under practical conditions deviates from the OC under the stated standard conditions. This is calculated with the so called α-factor.

α = OC in activated sludge / OC clean water

The α factor is mostly influenced by the surfactants and depends on the type of wastewater, the activated sludge process and the aeration system. Experimentally found values of α vary between 0.6 and 0.8 for fine bubble aeration and are mostly approx. 1.0 for surface aeration.

Oxygen demand – oxygen supply

The actual oxygen transfer depends still on the oxygen concentration that usually deviates from 0 mg/l. For an actual oxygen concentration of C mg/l is the actual oxygen transference, OC_{act}:

$$OC_{act} = \alpha \cdot OC \cdot (C_s - C)/C_s$$

where C_s is the oxygen saturation concentration in mg/l at the actual process temperature.
From this it follows that the amount of oxygen carried over increases, as the oxygen deficit is larger. A low oxygen concentration is therefore advantageous (from an economical point of view). However the reaction rate of the oxidation processes reduces with lower concentrations of oxygen.
The relationship between the oxygen demand OD and the necessary OC can be laid as:

$\alpha OC = (OD/24) \cdot p \cdot C_s/(C_s - C)$

Here p is a peek factor that depends on the fluctuations that can occur in the oxygen requirements. The daily occurring fluctuations depend on the variations in the supply and the set up and the size of the aeration tanks; for this a peek factor of 1.0 to 1.5 can be employed. Sometimes during low loading periods (seasonal influences) fluctuations occur that necesscitate a peek factor (1.1 - 2.0)

Surface Aeration

Aeration occurs through mechanical forces onto the liquid and is carried out by horizontal rotors or vertical turbines.
The oxygen is brought into the liquid via: the movement of the liquid surface, the bubbles that are brought into the liquid, the sprinkled liquid and the air-liquid mixture at the spot of the aerator, where the air is forced into the liquid.

The amount of oxygen, that is brought into the liquid, is influenced by the diameter of the revolving element, the rotational speed, the immersion depth and the shape and placing of the baffles.
Aside from bringing in oxygen, the aerators must circulate flow in the aeration tanks to prevent the activated sludge from settling. Shape, size and contents of the aeration tank must be related to the aerator in such a way that in adding in the required amount of oxygen the circulation is sufficient.

Vertical Axis Surface Aerator

A vertical axis surface aerator consists of a funnel or dish shaped fan with diameter of 0.5 - 4 m, which is driven by a vertical axle. The aerator acts as a sort of pump with a large capacity and a very small head loss.

The various constructions of the aerator cones nearly all work according to the same principle: in the tank there is a vertical circulation flow, in which the water/sludge mixtures axially sucked up from the bottom and radially dispersed over the water surface. With all the aerator cones the oxygen transfer primarily takes place in the turbulent area on the surface around the aerator cone. In addition oxygen transfer takes place as a result of dragging bubbles with the circulation flow in the direction of the bottom. Examples of cone aerators are given in figure 3.10 and 3.11.

For the analysis of the total performance of a vertical aerator the oxygen transfer yield is important, being

$$\eta_{O2} = \frac{OC}{N_{tot}} \cdot V$$

where

η_{O2} = the oxygen transfer efficiency in kgO_2/kWh

N_{tot} = the total power in KW; here the axel capacity, the efficiency of the electrical power and the electric motor and possible cable losses are taken into account

The oxygen transfer efficiency of the modern vertical axis aerators amounts to approximately 1.8 - 2.2 kg O_2/ kWh. With older types these values are at 1.3 – 1.8 O_2/KWh.

In some situations vibration problems can occur; these are usually caused by the twisting and turning of the water, which can create a wave. These problems can mostly be remedied by placing horizontal or vertical break walls.

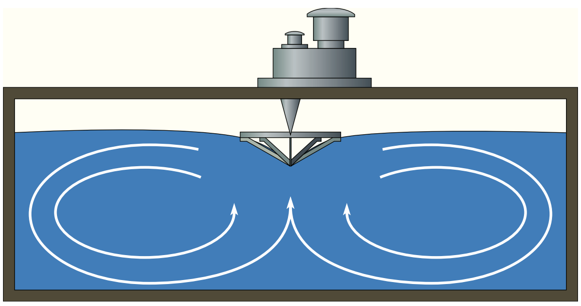


Figure 3.10 - Simcar-aerator, example of an aerator cone

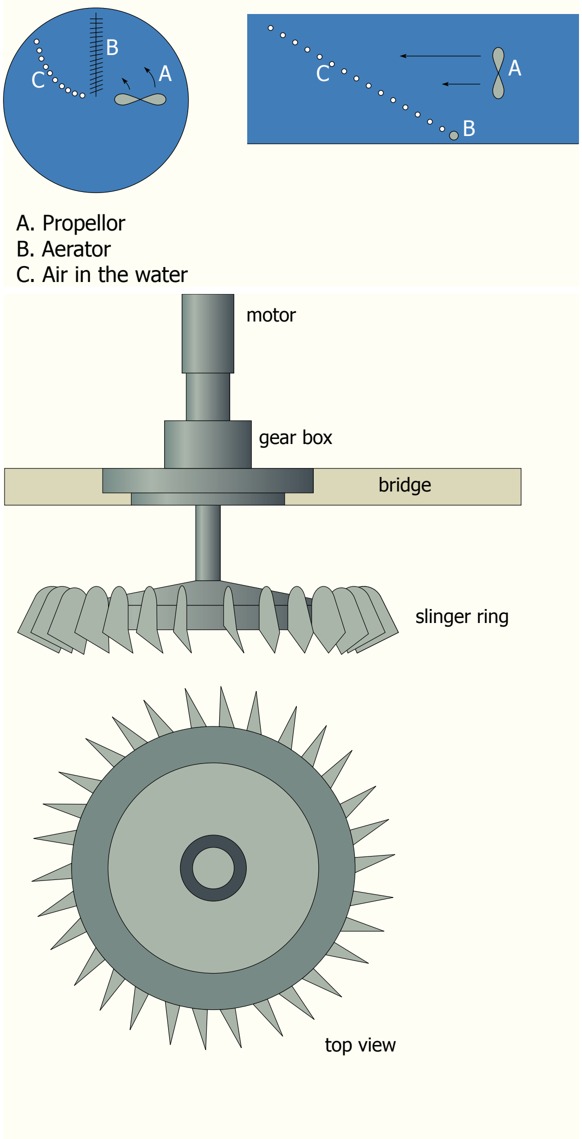


Figure 3.11 - Various aerator cones

Rotors or Brush Aerators

Rotors consist of a horizontally placed rotating axle with combs sticking out, upon which slabs or plates are fixed. Cage rotors are often installed especially in oxidation ditches. The axle consists of a pipe with a diameter of approximately 170 mm. The round plates are fitted directly on top of this axle, where the support frames are mounted evenly on the axle, from which a cylindrical cage exists. Plates are welded every other 5 cm from 5.15 cm onto the support frames. The total diameter of the cage rotoris 0.70 m. The total length amounts to maximum 6 – 8 meters. The immersion depth amounts to approximately maximum 22 cm. A drawing of the cage rotor/ is provided in figure 3-12.

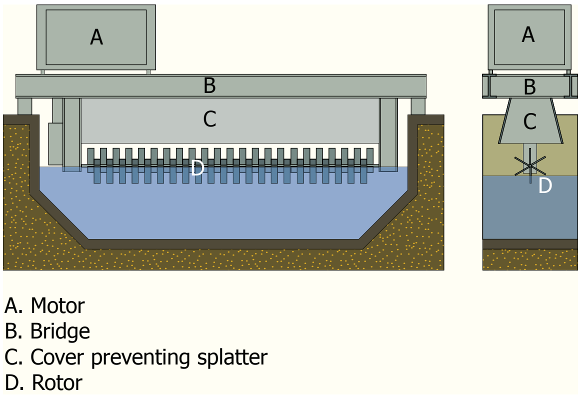


Figure 3.12 - Aeration rotor (Passavant)

The oxygen transfer capacity of the rotors depends on the rotational speed, the immersion depth, the liquid velocity and the length of the rotor. The rotational speed of the rotors amounts mostly from 75 to 100/min. As a result of the rotor the horizontal liquid velocity is stimulated; to operate a velocity of approximately 0.30 m/s a specific capacity of 10-12 W/m^3 is needed.

The oxygen transfer varies per type and amounts to 2.5-8.0 kg O_2/h.mr (mr refers to running meter of the rotor). The oxygen transfer efficiency in the working area amounts to approximately 1.7 kg O_2/kWh.

Fine Bubble Aeration

For fine bubble aeration air is sucked into compressors (or blowers), compressed and blown into diffusor elements situated at the bottom of the aeration tank. The aeration depth) depends on the depth of the tank and amounts mostly to 3- 5 m (sometimes even from 8 -10 m). The aeration elements create fine bubbles, sized from 2 - 6 mm; earlier systems were installed with larger bubbles but because of their poor efficiency have all but disappeared. The placing of the elements in the tank can have a large influence on the oxygen transfer efficiency (see figure 3.13). Suppressing the column effect leads to higher efficiency; that is why now an even as possible distribution of the aeration elements on the bottom of the tank is installed.

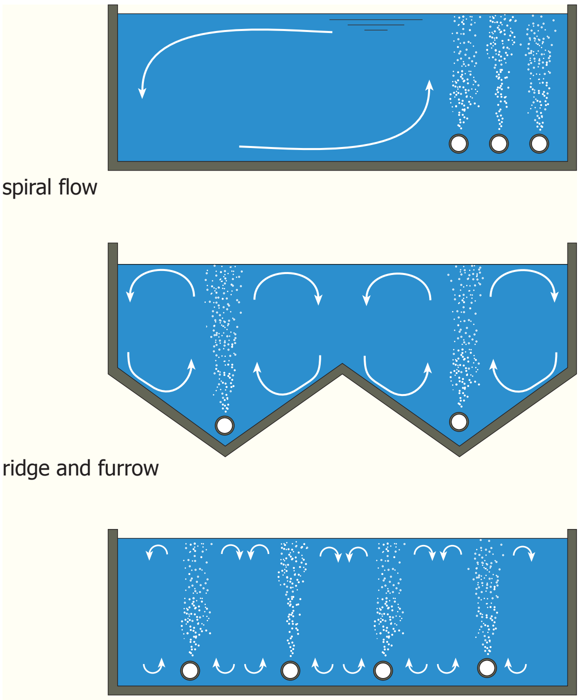


Figure 3.13 - distribution of the elements over the bottom creates spiral flows (whirlpools);
a. spiral flow; b. ridge and furrow; c. bottom aeration

For the fine blowing of the fine bubble aeration (bubble diameter 2 – 6 mm) are aeration elements of porous material with a pore size of 0.2 - 0.3 mm necessary. These elements are usually made of ceramic or plastic.

The shape can be:

- cylindrical with a length of 0.50 - 1.0 m (see figure 3.14)
- round dishes (domes) with a diameter 0.25 - 0.50 m (see figure 3.15).

The diffusers (aeration elements) are fixed onto the air distribution tubes (from steel or plastic) on the bottom of the aeration tanks.

The most important parameter is the air flow per unit of time and unit of length; this amounts to for tubes approx. 2 – 10 m^3/h.mT (mT meter tube) and for dishes 0.9 – 2.5 m^3/h.dish. As the air flow increases, the efficiency decreases (larger bubbles). The specific efficiency of the fine bubble aeration is given as

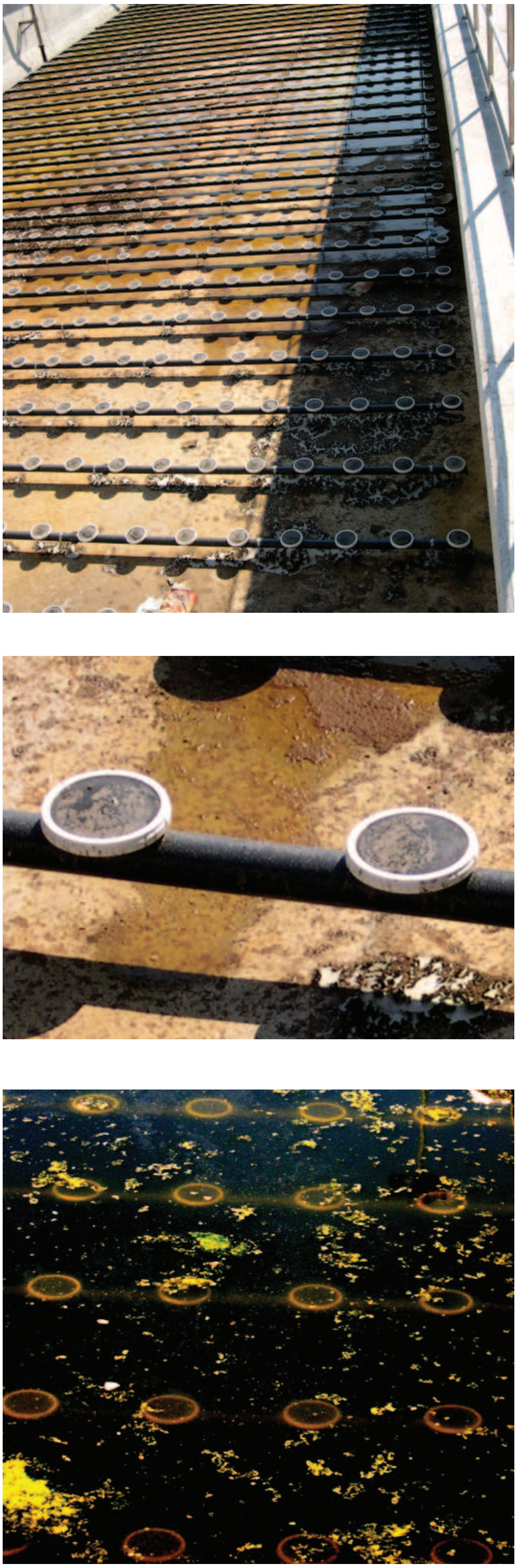


Figure 3.14 - Dish shaped aeration elements

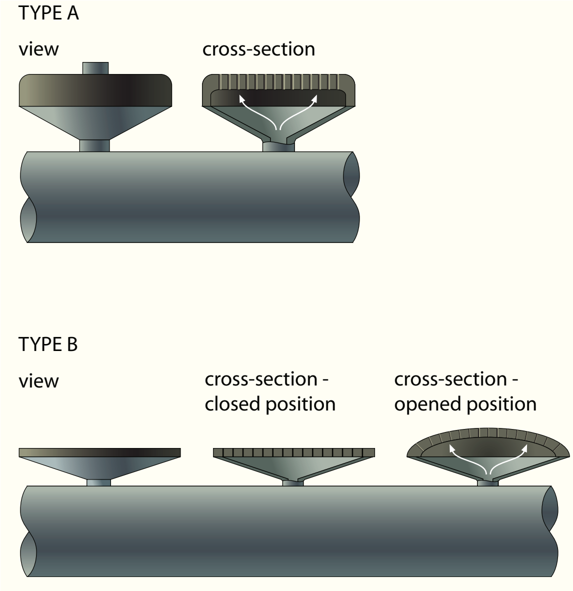


Figure 3.15 – Various types of aeration domes

$$OC_{spec} = \frac{OC}{Q_{AIR} . H_L}$$

where
OC_{spec} = the specific oxygen transfer in kg O_2/(Nm^3.m)
Q_{AIR} = the air flow in Nm^3/h
H_L = the head loss in m.
The specific oxygen transfer amounts mostly from 15-25 g O_2/(Nm^3.m).

The total efficiency can be calculated from the OC and the power Nb to blow in the air; this last factor is calculated with

$$N_b = \frac{Q_L \, x \, H}{367 \, x \, \eta}$$

where H is the aeration depth (plus any pipe losses) in m and η is the compressor efficiency, mostly approx. 0.7. The oxygen transfer efficiency for fine bubble aeration amounts mostly 3.0 - 4.0 kg O_2/kWh in clean water or 2.0 - 3.0 kgO_2/kWh in activated sludge.

It is necessary to filter the air before it is sucked up to prevent clogging on the inside of the diffusers. With suspension of the aeration process the diffusers can become clogged with sludge. That is why it is essential to apply a certain pressure on the system if the aeration is ceased. For discontinuing operations there are rubber or (preferably) synthetic elements for the purpose of sealing off the air supply.

Horizontal Flow

The residence time of the bubbles in the water can also be increased by adding a horizontal flow. Examples of this are given in figure 3.16 and 3.17. For these systems it is nevertheless possible to realise a high oxygen transfer efficiency in tanks that are relatively large and have a low aeration density for example with ultra-low loading activated sludge systems. The power consumption of the propulsion system amounts mostly to 1.5 – 2.0 W/m³.

3.2.8 Forms of Construction

Activated sludge systems almost always consist of one or more aeration tanks and secondary settling tanks and a return activated sludge system. Usually a primary settling tank is used in advance. The aeration tank can be built in various ways. Usually the tanks are 4 à 5 m deep; the total contents can amount to 50,000 à 100,000 m^3. The tank can also be comprised of more compartments. The most important aspects are formed by the supply of wastewater and return

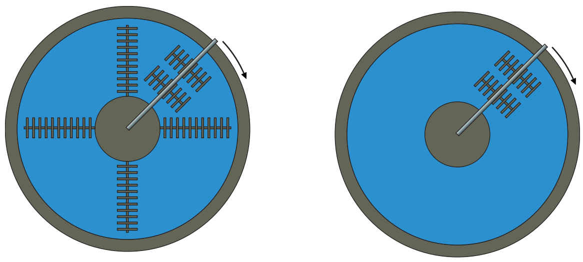
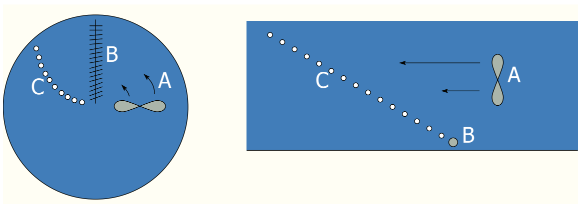


Figure 3.16 – aeration combined with horizontal flow, stirred up by a rotating bridge (Schreiber).



A. Propellor
B. Aerator
C. Air in the water

Figure 3.17 – Aeration combined with horizontal flow, stirred up by a propeller (Rotoflow).

activated sludge, the compartmentalizing and the way in which it is aerated, including the control options. About the construction aspects: a trend has been observed towards the implementation of round tanks (lower construction costs).

Complete Mixed Aeration Tank

For this system an even distribution of the wastewater and sludge in the total aeration tank is aimed for (see figure 3.18).
The advantages to this are that variations in the wastewater supply are distributed evenly over the contents of the aeration tank. In this way a direct dilution with the entire tank contents takes place. The activity in the aeration tank is the same over all. Complete mixing is moderately approached in a square aeration tank with one aerator cone. Concerning measurement and control, a complete mixed aeration tank is by far the easiest to operate.

Plug Flow

In this system wastewater and return activated sludge are added in at the head of the aeration area (see figure 3.19). A characteristic of the plug flow system is that in the aeration tank, the sludge loading is the highest and the largest part of the supplied contaminants (organic particles: substrate) are absorbed and degraded. In the direction of the end of the tank the processes

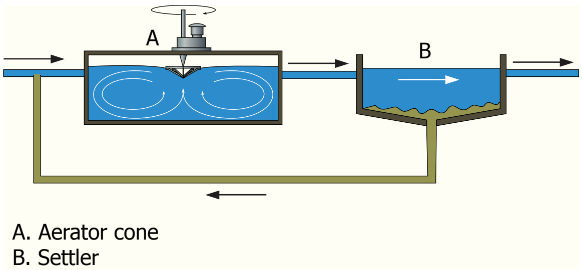


Figure 3.18 - Complete mixed aeration tank

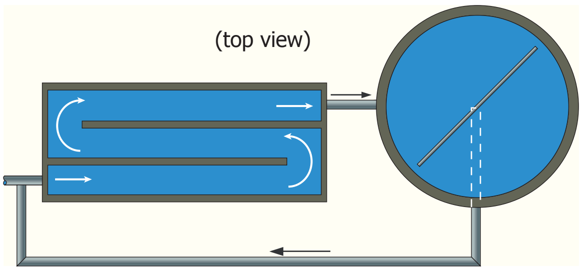


Figure 3.19 - Plug flow system

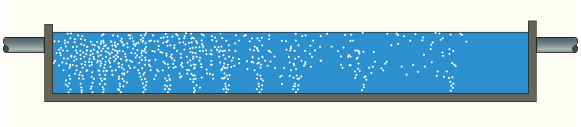


Figure 3.20 – 'Tapered aeration'

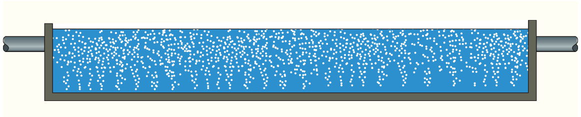


Figure 3.21 – Uniform bottom aeration

become less intense, where after the activated sludge at the end of the aeration area even could face endogenous respiration.

In the field of wastewater treatment plug flow is usually well approaced in long rectangular aeration tanks. The oxygen demand in the beginning of the aeration tank is noticeably larger than further downstream. If the oxygen supply is adjusted well to the oxygen demand it is referred to as 'tapered aeration' (see figure 3.20), this contrasts with uniform bottom aeration (see figure 3.21).

In terms of measurement and control, the plug flow system is difficult to operate; variations in the concentrations move farther along as a 'plug'. Shock loads and toxic compounds can influence the process negatively. The following process techniques can be advantageous in regards to plug flow over totally mixed systems:

- no chance of short circuits;
- less chance of bulking sludge. (see section 9.9)
- More extensive treatment within an even sized aeration volume

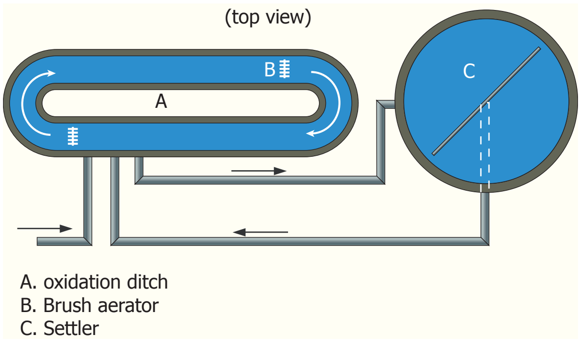


Figure 3.22 – Continuous oxidation ditch

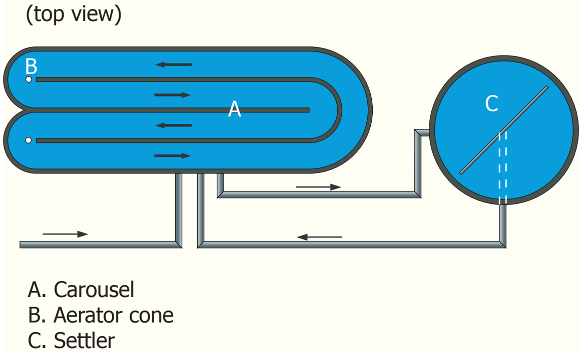


Figure 3.23 - Carousel system

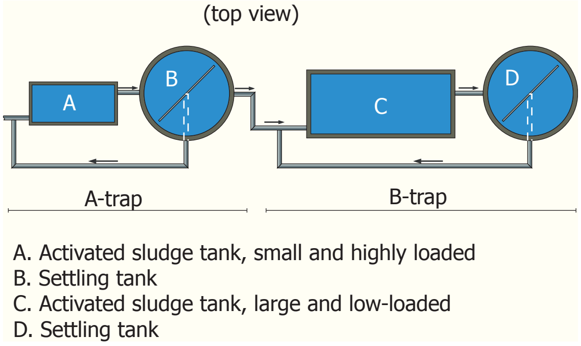


Figure 3.24 - Secondary activated sludge system (AB-system)

Furthermore the plug flow could also be realized by connecting diverse compartments in a series.

Bypass Systems

The characteristics of a bypass system are that the wastewater/activated sludge mixture goes around many times in the aeration area, before the treated wastewater leaves the installation. Examples of bypass systems are the traditional oxidation ditch (see figure 3.22) and the carrousel (see figure 3.23). The amount of activated sludge that flows through the channel of an oxidation ditch or carrousel is many times greater than the supplied wastewater flow (for example 50 times more).

This results in largely diluted wastewater at the moment that it enters the aeration tank. The BOD, N-Kj and NO_3-N concentrations are then nearly equal in the entire aeration area. By local aeration the oxygen concentration varies during the circulation; because of this, favourable conditions exist for nitrification and denitrification.

The following are advantages of the bypass system:

- limited chance of short circuiting, the influent is strongly diluted.
- good nitrification and denitrification possibilities;
- relatively simple to operate concerning measuring and control

A disadvantage is:

- it is a completely mixed system; the chance of bulking sludge is therefore greater than with a plug flow system.

Two-stage Activated Sludge System

A two-stage activated sludge system consists of a high-load primary treatment with an intermediate settling tank and a low-load secondary treatment with a final clarifier. It is possible to install another primary settling tank before the primary treatment. Characteristic of this system is that primary treatment as well as secondary treatment are both have a seperate return activated sludge system where two completely separated sludge systems exist. (Figure 3.24)

The most well known are the AB-systems: in the primary treatment, the absorption stage, amounts to the BOD sludge loading approximately 2 kg BOD/(kg ds.d). The BOD removal efficiency from the primary treatment amounts to 70%; the energy costs of the primary treatment are relatively low. In the secondary treatment, the biodegradation stage (actually from the German “Belebung”) with a BOD sludge loading of 0.15 kg BOD/(kg ds.d) the further BOD-degradation and nitrification take place.

An important advantage to the AB-system is that in both the A-stage as in the B-stage good sludge settleability exists. Disadvantages are the limited possibilities of denitrification and for biological dephosphorization (see chapter 4).

3.2.9 Secondary Sedimentation

The biological sludge produced during the secondary process is settled with clarifiers and intermediate settling tanks where the sludge settles in thinly formed flocs. During the settling process smaller flocs fuse and form larger flocs. At certain suspended solid concentrations, the flocs will prevent each other from settling. As a measure for the settleability the sludge volume index (SVI) is used. (see 3.2.4)

With secondary clarifiers the loading capacity is determined by:

- surface loading rate, v_o in $m^3/(m^2 \cdot h)$
- suspended solids concentration in the inflow, X_A in g/l
- sludge volume index, SVI in ml/g.

On the basis of many practical measurements in both round and rectangular tanks, it has been shown that the sludge volume surface loading may amount to no more than 0.3-0.4 $m^3/m^2.h$ The sludge volume surface loading (vs) is equal to

$$v_s = v_o \cdot X_A \cdot SVI/1000$$

If we make the sludge volume (settled sludge) V_a equal to X_A.SVI a connection is found between the surface loading rate V_o and the settled sludge V_a (see figure 3.25).

Besides separating the settleable sludge, the clarifier also has a buffering function. During dry weather the clarifier usually is under loaded and will have relatively little sludge stored in

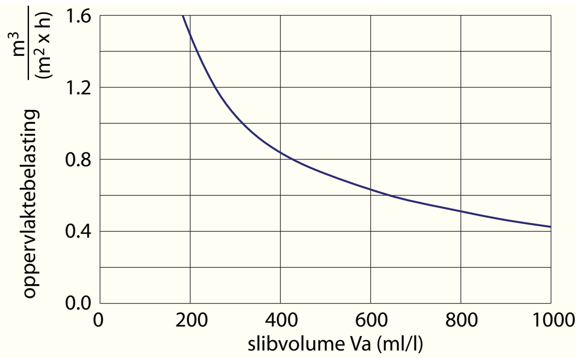


Figure 3.25 - Relation surface loading and sludge volume

the clarifier. If the amount of wastewater (Q) increases, for example during a rainfall, then the suspended solid supply to the clarifier from the aeration tank will increase as well. This will become $(Q+Q_R)_{wwf} \cdot X_A$. Since the sludge thickening is somewhat delayed, the return activated sludge concentration (X_R) will also increase less quickly and an imbalance occurs. More sludge goes to the clarifier than is taken out and sludge stays behind in the clarifier. Increasing Q_R does not correct these imbalances immediately, because by increasing Q_R also the supply of suspended solids will increase and moreover, turbulences will occur. From retaining sludge in the clarifier a sludge blanket comes into existence. The top layer of this sludge blanket, that is to say the separation of water and sludge, is called the sludge blanket level. Because with high supply the sludge layer in the clarifier gets thicker, the sludge will get thickened more easily and in that way X_R increases. The sludge discharge from the aeration area to the clarifier during rainy weather will decrease X_A. This decline is significant. If the clarifier is not over loaded, then a new balance situation occurs. The sludge blanket level comes up to a higher level to rest and X_A and X_R barely change anymore. It is also now true that:

$$(Q + Q_R) \cdot X_{Awwf} = Q_R \cdot X_{Rwwf}$$

X_A with wwf is larger than with dwf. (wwf and dwf denote wet (rain) weather flow and dry weather flow, respectively)

It is also possible that the clarifier is loaded in such a way that the sludge blanket level continues to increase and finally after a few hours sludge flows over the weir of the clarifier.

It is also possible that in the clarifier buffered sludge can be used to compensate for loading peaks: With a higher supply of organic particles the sludge concentration in the aeration tank can be raised by increasing the recirculation rate. By taking such action the sludge load (kg BOD/kg ds.d) is kept constant.

Under certain circumstances some wwtp (periodically) can have the problem of “bulking sludge”. Bulking sludge is the result of massive growth of the filamentous micro-organisms that negatively influence the settelabiility of the sludge. This results in higher sludge and thickening volumes in the clarifier and mostly with a higher hydraulic load, sludge will washout with the effluent.

4 Additional Treatment

4. Additional Treatment

4.1 Nitrogen Removal

For more information see ME Chapter 8-5

4.1.1 General

Nitrogen in wastewater is mainly present in the form of ammonium and organic nitrogen. In the activated sludge process it is possible to transform the nitrogen biologically. In a first step it is converted through nitrification, or that is to say the oxidative conversion of ammonium to nitrate (nitrification); then the nitrate can be reduced to nitrogen gas (denitrification).

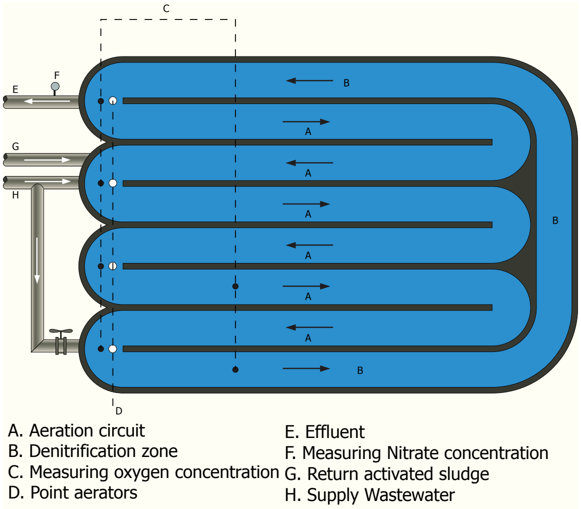


Figure 4.1 - Carousel with denitrification zone

4.1.2 Nitrification

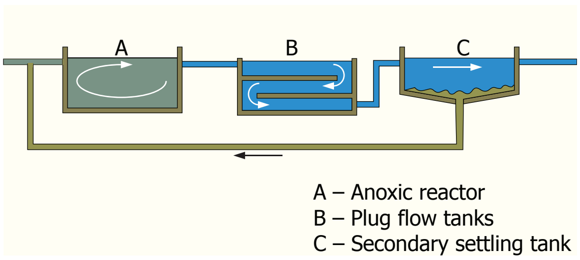
Transferring ammonium into nitrate takes place in two stages with nitrite as an intermediate; the first stage is the slowest and is performed by a type of bacteria known as Nitrosomonas. These bacteria grow slowly and will only appear in the sludge if the sludge age is large enough (and the sludge loading is low enough); furthermore they grow more slowly in winter with lower temperatures than in summer. With a sludge loading of 0.1 kg BOD / (kg ds.d) nitrification can occur down to 7˚C. In addition the oxygen concentration in the activated sludge must be high enough, minimum 0.5 – 1.0 mgO_2/l.

4.1.3 Denitrification

If the dissolved oxygen concentration is very low (in an anoxic environment), a large number of types of bacteria is capable of converting organic matter in an oxidative reaction using the present oxygen in either nitrate or nitrite for its respiration processes.These facultative bacteria are thus able to change from using free dissolved (air) oxygen into using nitrate oxygen.
The nitrate will, as a result of the absence of the dissolved oxygen, and in the presence of organic compounds (oxygen demand) be reduced to nitrogen gas (N_2). The formed nitrogen gas leaves the liquid in the form of bubbles. Organic matter acts as a driving factor for the occurrence of denitrification.. For this process both low oxygen (anoxic) conditions are necessary as well as the presence of readily biodegradable soluble organic material.

4.1.4 Implementations

Nitrification and denitrification are built into the activated sludge process in various ways. In the first place the processes can take place next to each other and after each other by creating oxygen rich and oxygen poor (anoxic) zones in the aeration tank (simultaneous nitrification / denitrification). This procedure in practice works well in oxidation ditches; nitrogen removal with efficiency of more than 90% is possible.



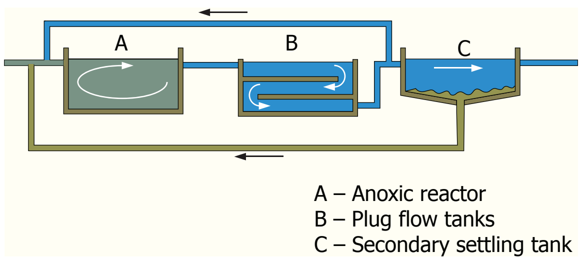


Figure 4.2 – Plug flow reactor with primary denitrification; b with extra recirculation

In figure 4.1 a carousel is shown as an example where the oxygen supply capacity of the aerators is controlled by the dissolved oxygen sensors. The aeration tank is divided into a part with oxygen sulpy (aeration) and a part without oxygen supply.

With plug flow systems denitrification can be realized by limiting the oxygen supply in the first part (oxygen concentration in any case lower than 0.5 mg/l) (see figure 4.2). In this case the necessary nitrate for this so called pre denitrification configuration is supplied through the
return activated sludge flow. The efficiency of the nitrogen removal is thus dependent on the return activated sludge ratio. This mostly becomes larger or an extra recirculation at the end of the aeration tank is installed.

Another construction is the post denitrification, where anoxic conditions are created in a part after nitrification. It is then not necessary to recycle nitrate because this is directly supplied from the nitrification part. One complication with this configuration is that there may be not enough organic material remaining for a complete denitrification. In that case an external "carbon source" (e.g. methanol) must be added.

4.1.5 Dimensioning

Nitrification:

- high enough sludge age because of the slow growth rate of nitrifying bacteria;
- strongly dependent on temperature, so high enough temperature.

Denitrification:

- anoxic zone (approximately 30% of total).

For Dutch conditions this leads to a sludge loading rate of up to 0.15 kg BOD/kg TS / d)).

4.2 Phosphate Removal

For more information see ME Chapter 6-4 and 8-6

4.2.1 General

Phosphate can be removed by adding iron or aluminium salts which then forms deposits. Phosphate can also be biochemically bound in the activated sludge. There are many various designs for both principles.

4.2.2 Chemical Precipitation

With chemical dephosphating, the phosphate binds with polyvalent metal ions to form an undissolved complex. The metal ions are dosed in the form of iron salts, aluminium salts or lime (calcium) . The chemicals can be added in various places in the WWTP (see figure 4.3). To distinguish are:

- preprecipitation: dosing of trivalent metal salts (e.g. $FeCl_3$) on the inlet of the primary sedimentation. The precipitation takes place in the primary settling tank. A disadvantage is the (too) high removal of readily biodegradable material that is actually necessary for denitrification in the activated sludge process;
- simultaneous precipitation: dosing of iron or aluminium salts (ferrous sulphate mainly) and precipitation of phosphate sludge mixed with activated sludge. As a result of this the settleability increases but the total activity of the activated sludge decreases:
- post precipitation: adding the trivalent metal salts and separating them from the treated water in a filter unit or settling tank. This is only economically feasible with low phosphate concentrations;
- chemical precipitation with biological phosphate sludge

4.2.3 Biological Phosphorus Removal

Biological phosphorus removal is a microbiological method to remove phosphorus from wastewater by means of phosphate accumulating bacteria. These aerobic bacteria, which are found in the activated sludge, are not only capable of forming the normal amount of phosphorus necessa ry for cell growth, but also of taking up the extra phosphorus in the form of polyphosphate into the cell, the so called 'luxury uptake'.

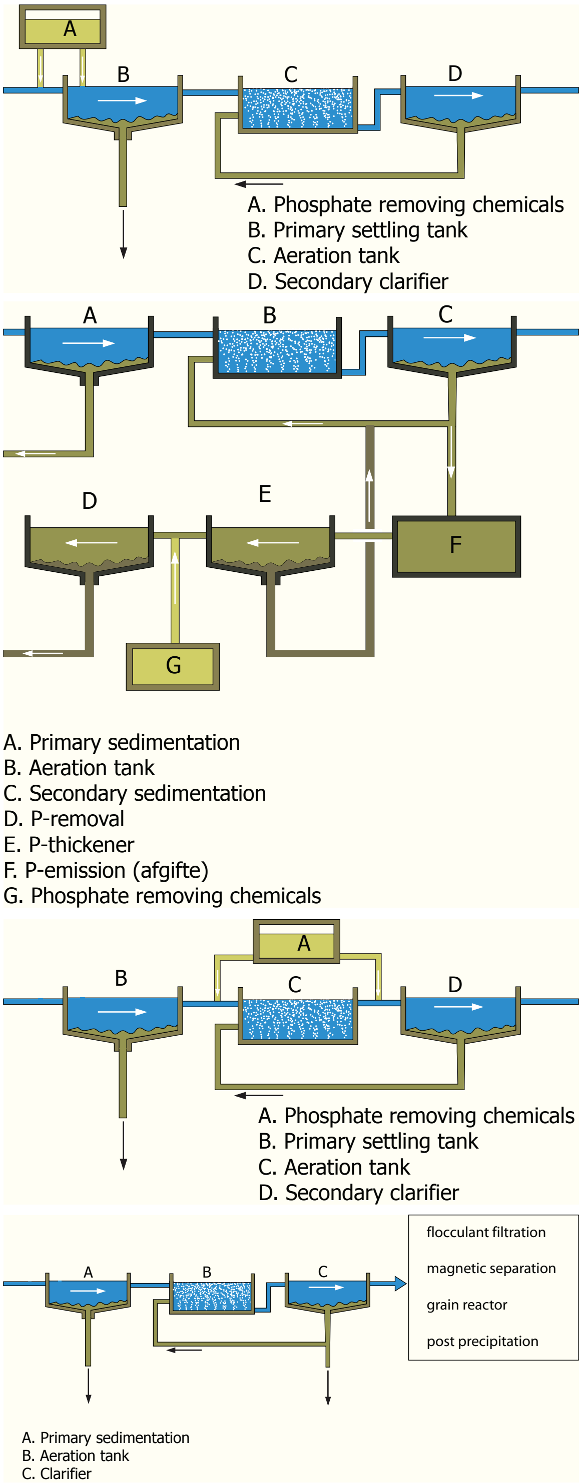


Figure 4.3 - Construction forms of chemical precipitation

To be able to make use of the characteristics of the phosphate accumulating bacteria at a wwtp it is essential that an alternating anaerobic and aerobic environment is created.

- an anaerobic environment stimulates the selection of the phosphate accumulating bacteria. Along with this it is important that this environment is not only free of oxygen but also of nitrate. I.e. that biological phosphate removal can only take place if nitrate removal also takes place in the treatment system;
- an aerobic environment stimulates the phosphate uptake from the wastewater.

As a realization for the biological phosphate removal two principles are described (see figure 4.4):

- the head flow process
- the split flow process.

For biological phosphate removal in the head flow the uptake of phosphate takes place by the activated sludge in the aeration tank. The extra phosphate removal is realized through the discharge of the excess (phosphate-rich) sludge. In the split flow process the uptake of phosphate takes place in the aeration tank as well, but this is removed from the sludge in a split flow. The treatment of the activated sludge in the split flow consists of bringing the phosphate back into the solution ('stripping'). The phosphate-poor sludge is separated from the phosphate-rich liquid. The

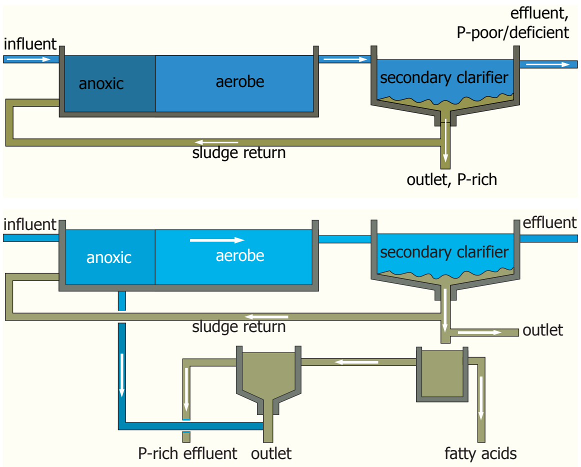


Figure 4.4- Principle of biological phosphate removal Head flow process (above) and split flow process (below)

phosphate is then removed from the water phase by means of a chemical method. The phosphate-poor sludge goes back to the head flow where it can absorb the phosphate again.

4.2.4 Dimensioning

Chemical precipitation

- Me/P ratio 1.5- 2 mol/mol
- Extra sludge production 15 - 20 gds/d.PE

Biological phosphorus removal

- head flow process: anaerobic tank until approximately 1 hour residence time
- split flow process:
  - split flow 30 - 50 % of return sludge
  - residence time approximately 2 - 3 hours
  - sludge water separation residence time 1 - 2 hours
- the process must consist of a minimum of an anaerobic, anoxic and aerobic process.

In practise it is common to remove as much phosphate as possible through the biological head flow process; if this is not enough than the chemical precipitation can take place.

4.3 Extended treatment

For more information see ME Chapter 11

4.3.1 General

For extended treatments there are multiple mainly physical-chemical processes available.

First of all attention will be given to the removal of particles (suspended solids). In the first place there are filtration techniques available that are not supported by chemical precipitation and or coagulation/flocculation.
Membrane filtration focuses on particles smaller than 10 µm. Successively through microfiltration the micro particles and bacteria, with ultrafiltration colloids and viruses, and with nano- and hyperfiltration even ions and molecules can be removed (see table 4.1).

The hygienic quality can be further improved by adding specific disinfection processes such as chlorination, ozone treatment and UV radiation. Activated carbon treatment is an absorption technique in which many particles, mainly of an organic nature (micro pollutants, hormone disturbing substances), can be removed.
For the removal of nitrogen, which occurs as either ammonium or nitrate, there are many techniques available. Ammonium can be removed through ammonia stripping or ion exchange. Nitrate is most easily removed through biological denitrification. With the absorption techniques the handling of the resulting solids poses many problems.

Table 4.1 – Removal of particles through filtration

particle size	ion molecules	macro molecules	micro particles	fine particles
µm	0,0001 · 0,001	0,01 · 0,1	1	10 · 100 · 1000
particles:	dissolved salts · sugar	viruses · colloids	bacteria	pollen · sand
process:	hyper-filtration · nano-filtration	ultra-filtration	micro-filtration	particles filtration

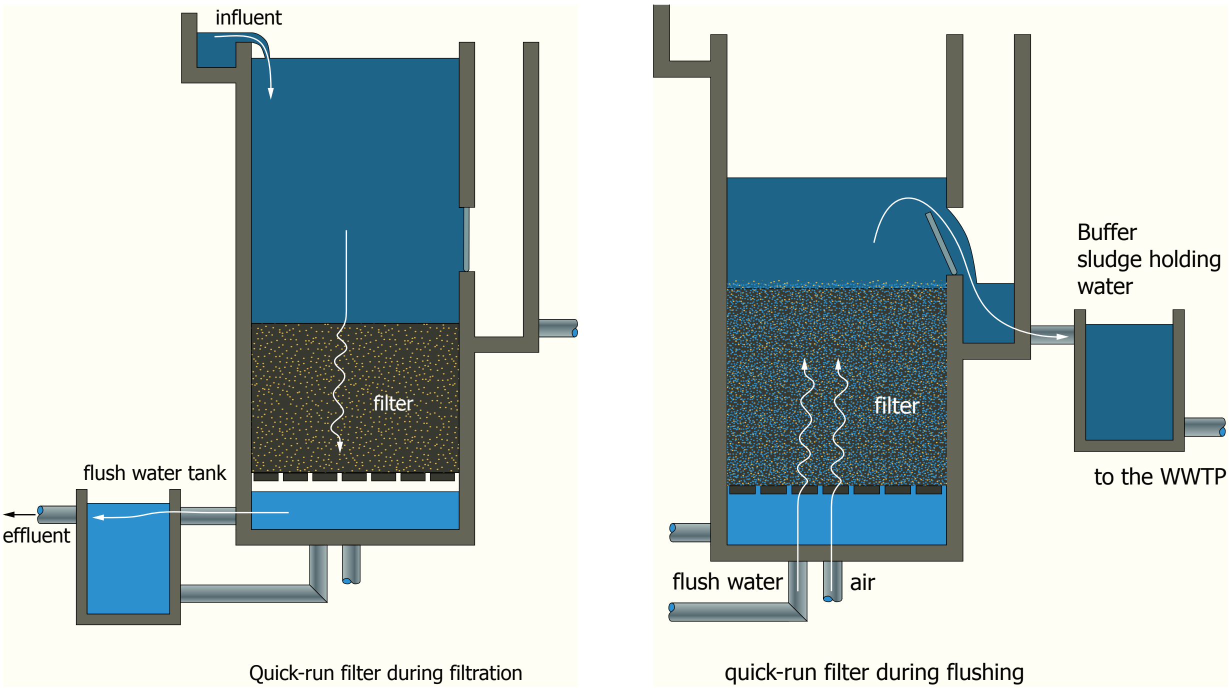


Figure 4.5 - Quick-run filter, single layer, under gravity

4.3.2 Deep bed filter

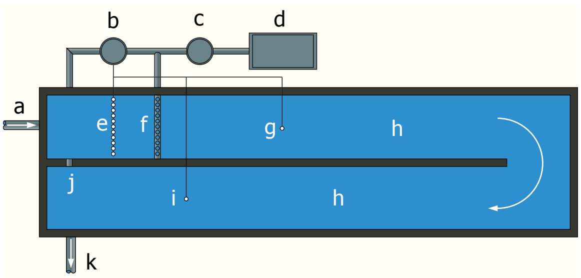
Deep bed filters are mostly used for effluent filtration and usually implemented as a double layer filter. These are open filters that work under the influence of gravity. In filters with an ascending supernatant level the resistance that occurs over time in the filter bed is compensated by the rising supernatant level, which increases the pressure in the filter. The advantage of these types of filters is that they are very simple to operated. A disadvantage is the required height necessary to allow the supernatant level to rise 1 to 2 m above the filter bed (height 1-2m). The large height makes these filters expensive to construct.

The filtration velocity amounts to approximately 5 – 12 m/h; sometimes running up to 20 – 30 m/h. The following materials are usually installed in layers:

- anthracite	80 cm	2.0 – 4.0 mm
- sand	40 cm	1.5 – 2.2 mm
- granite sand	30 cm	0.5 – 0.8 mm

4.3.3 Chlorination

To destroy the pathogenic organisms that occur in large amounts in the effluent often chlorine is added as a disinfectant. The chlorine is often dosed as sodium hypochlorite solution. After mixing it in the effluent a contact time of approximately 0.5 hours is necessary for a proper inactivation. The required dose for well treated effluent amounts to approximately 3 - 4 g Cl_2 / m^3.



a. inflow: effluent from the wastewater treatment plant
b. mixing pump for diluting chlorine
c. chlorine dosage
d. chlorine store
e. measuring treshold (first regulation of c.)
f. mixing
g. chlorine measurement (correction of the regulation of d)
h. contact area
i. measurement residual chlorine (control measurement)
j. short circuits
k. discharge

Figure 4.6 – Scheme of a disinfection installation

5 Sludge Treatment

5. Sludge Treatment

5.1 Sludge Thickening

For more information see ME Chapter 14-6

5.1.1 General

During the treatment of wastewater sludge is produced. This consists mainly of water (typically >95%). The sludge can go through multiple treatments, from which one of the most important is the thickening process. In essence the thickening is only a removal of a part of the water from the sludge. Prior to and after the thickening the sludge mixture is liquid. The separation from the water occurs under the influence of certain dewatering forces; the water is only lightly bonded to the sludge.

5.1.2 Sludge Composition

Sludge from a wwtp is difficult to define. The composition and workability cohere with the type of wastewater and the kind of wastewater treatment system. Dry wwtp sludge consists of among other things:

- 45-60% organic material (loss on ignition)
- 25-55% inert material (ash content)
- 2-4% nitrogen
- 0.5-1.0% phosphorus
- 0.2-0.4% sulphur, chlorine
- 0.1-0.2% heavy metals
- organic micro-contaminants

Within the biological wastewater treatment during the various process stages different types of sludge are produced.

- Primary sludge
  Sludge that is separated in the primary settling tank at a wastewater treatment plant and that consists of settleable suspended solids. The water percentage is between the 93 and 97 %. (suspended solids 3-7 %)
- Secondary sludge (also excess sludge/surplus sludge)
  Biological sludge that is removed from the activated sludge installation (either aerobic or anaerobic sludge, or a mixture of both); the amount of suspended solids is between 0.8 and 1%.
- Mixed sludge
  Sludge that exists as the secondary sludge is mixed with the primary sludge.

5.1.3 Sludge Amounts

The amount of sludge that is produced at a treatment installation depends on:

- the nature of the wastewater (domestic/industrial)
- sewarage system (with or without settling)
- treatment process.

The production of the sludge per population equivalent depends on the treatment process (See table 5.1).

The binding characteristics of the water in the sludge differs and thus, it is bonded with various binding forces to the sludge. We describe free, colloidal, capillair and cellular bonded water (in sequence of increasing binding). The free water can mainly be removed in a simple way. With complete separation a suspended solids content of 15 to 20% is achieved; in practice it seems that with thickening 5 to 8% suspended solids content is realizable at the end.

5.1.4 Thickening

With sludge thickening a large part of the so called free supernatant liquor, which has no binding forces with sludge and makes up approximately 70% of the total water content, is separated. As a result of this, the suspended

Table 5.1 – Production of sludge per population equivalent

Treatment process		gds/d	Organic part (%)
primary sludge (from primary treatment)		40-50	70
trickling filter	highly loaded	20	65
	low loaded	13	60
activated sludge		25-30	75
phosphate-sludge (extra)		10-20	10
oxidation ditch		40-60	60
digested		55	50

solids content of the sludge increases and the sludge volume is reduced. This is very important on the one hand for the subsequent sludge treatment steps, such as for example sludge digestion and sludge dewatering, and on the other hand for the transportation of liquid sludge to a central processing plant.

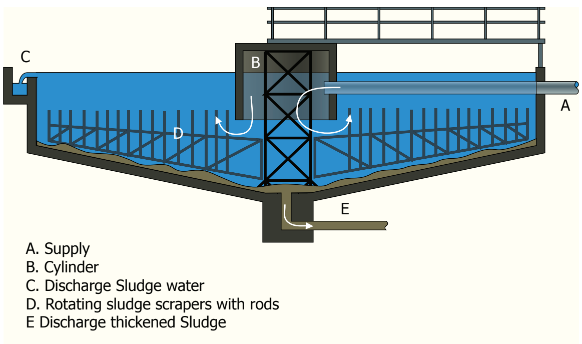
Technical implementations of sludge thickening are:

- gravitational thickening
- mechanical thickening (=thickening table)
- mechanical thickening with drum sieve
- mechanical thickening with centrifuges
- mechanical thickening by means of flotation (= flotation thickening).

5.1.5 Gravitational Thickening

Gravitational thickeners are usually constructed as round tanks with a sloping or flat bottom, according to the same principle as a settling tank. The thickener is equipped with a slowly rotating agitator; the round rods serve to very lightly agitate and to mix (degassing). In general at the surface, the floating scum is held back by a scum baffle and is removed through a discharge drain. Implemented diameters for thickeners vary from 5 to 20 - 25 m; the sidewater depth amounts to minimum 3 m and for practical reasons maximum 5 m.

The supply of sludge (from the primary settling tank) is often continuous; the sludge is then still relatively unconcentrated (10 - 20 gds/s, that is to say 1 - 2% ds); the discharge takes place discontinuously throughout the day.



Figur 5.1 - Gravitational thickener

Apart from the actual thickening process in a thickener, normally also homogenization and buffering take place.

In terms of designing, the nature and quality of the sludge must be taken into account. The most important variables are the suspended solids load V_{ds} and the sludge retention time.

$$v_{ds} = \frac{Q_s \cdot X_{sl}}{A}$$

where

Q_s = flow in m^3/d
X_{sl} = sludge concentration in kg/m^3
A = surface in m^2.

For one certain type of sludge the thickening results reduce as the suspended solids load increases; that is why the suitable suspended solids load at the design and operation are tied to a maximal value. Depending on the sort of sludge various values can be used (see table 5.2).

Also the retention time of the sludge is important. The retention time of the sludge is related to the sludge layer thickness (retention time = volume of the sludge layer divided by the flow of the thickened sludge). This translates itself usually in a side water depth of 3 - 6 m. The retention time may not be too short because then the required thickening is (still) not achieved. If the retention time is too long then the sludge will digest (acidification, rotting). The production of gas bubbles can disturb the thickening process. One day can be kept as a maximum retention time, because longer thickening barely even brings about a reduction in volume. Mixed primary sludge with a high content of organic

Table 5.2 – Suspended solids load with various types of sludge

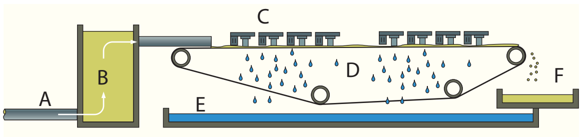
sludge type	ds load in kg ds/($m^2 \cdot d$)
primary	100-150
surplus (activated)	20-30
primary + surplus	30-50
digested	30-50
oxidation ditch	30

particles may not stay in the thickener too long considering the danger of rotting.
The hydraulic retention time (= volume of the thickener divided by the supply flow) has no influence on the production of the thickener.

5.1.6 Mechanical Thickening

By means of mechanical thickening suspended solid contents of 5% ds - 7% ds for both the primary as well as the secondary sludge are obtained. In contrast to gravitational thickening, energy and conditioning agents (in the form of poly-electrolyte) are used along with mechanical thickening. Mechanical thickening systems are nevertheless clearly more compact than the relatively large primary sludge thickeners. The most commonly administered mechanical thickening techniques in urban wastewater treatment are the belt filters and the drum sieves.

Currently in the Netherlands there is large scale usage of gravitational thickeners, but because of the advancement of biological phosphate removal increasingly more mechanical thickening techniques are being applied. Since the sludge has a longer retention time under anaerobic conditions in gravitational thickeners, the release of the biologically bound phosphate takes place. This phosphate returns directly to in the water line via the overflow water from the thickener, which further loads the wwtp than is necessary. To prevent P emissions, as a rule of thumb, a maximal residence time of 24 hours is employed (in practice it seems however that even in this relatively short residence time phosphate release can take place).



A. Sludge supply
B. Flocculation tank
C. Ploughs (ploegen)
D. Gravitational dewatering
E. Water discharge
F. Sludge discharge

Figure 5.2 – Sieve belt for mechanical thickening of wwtp sludge

5.2 Sludge Digestion

For more information see ME Chapter 7-12 and 14-9

5.2.1 General

Mixed treated primary sludge consists of a large fraction of decaying organic material. If it is not quickly processed, than it will start to acidify and an unbearable odour will be developed. Moreover, the sludge is the perfect environment for germs, in which it is not necessarily hygienic. These problematic characteristics can mainly be handled by stabilizing the sludge. In this microbiological process the organic sludge particles are broken down further. In the anaerobic stabilization or sludge digestion this occurs under oxygen free conditions.

With sludge digestion the following is intended:

- the acquiring of a material, that is better suited for further processing
- the fraction of organic material to be reduced and to increase the percentage of suspended solids in the sludge; this means that the sludge volume is reduced
- the elimination of odorous causing components in the sludge
- the improvement of the hygienic quality.

Through the digestion process a stabilized material is obtained. Sludge stabilization in this case can be understood as an extended anaerobic degradation, resulting in sludge that after the removal from the sludge digestion tanks (the digestate) is no longer digesting or rotting and thus no odour will be spread.

Fresh, mixed sludge is a drab grey colour, cannot easily be dried on drying fields, attracts flies, is infectious and spreads, after complete transition in the rotting phase, an enormous amount of bad odours. Digested sludge is black, smells like soil and can be dried on dry fields. Although the hygienic quality is improved, the digested sludge is nevertheless in no way free of germs.

A stabilized sludge can also be obtained in an aerobic way. This is mostly done in the activated sludge process, implemented as a very low-loaded system with a sludge age of more than 25 days (oxidation ditch). This route uses a lot of (aeration) energy, so usually sludge digestion is chosen for the larger plants.

Along with sludge stabilisation during the digestion biogas is produced, which can be used for generating energy.

5.2.2 Theory

Sludge digestion consists of a network of process stages, which is carried out by various types of bacteria. On the whole the degradation process is divided in three sequential steps: hydrolysis, acidification and methane formation.

The large sludge molecules are split into smaller building blocks outside the bacteria cells by hydrolysis-enzymes, which then can be consumed by the cell bacteria. The necessary enzymes are secreted by the so called acidifying bacteria. Because the waste activated sludge consists of many complex, slowly biodegradable components it goes through hydrolysis relatively slowly and incompletely.

The transformation of the hydrolysis products takes place inside the cells of the acidifying bacteria and this goes fairly quickly. Through diverse degradation phases, in which the intermediate product from the one to the other type of bacteria can be passed, acetic acid, CO_2 and H_2 are formed.

The end products of the acidification are transferred by the methanogenic microorganisms into a mixture of methane and CO_2. The methanogenic microorganisms, or methanogens, have a complete other metabolism than the acidifying bacteria.They grow very slowly and are sensitive for certain toxic particles (see also figure 5.3 and figure 5.4).

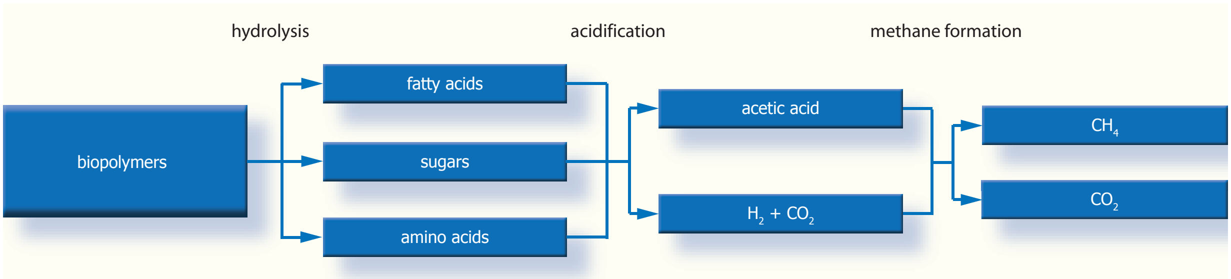


Figure 5.3 – Schematic depiction of the degradation steps with sludge digestion

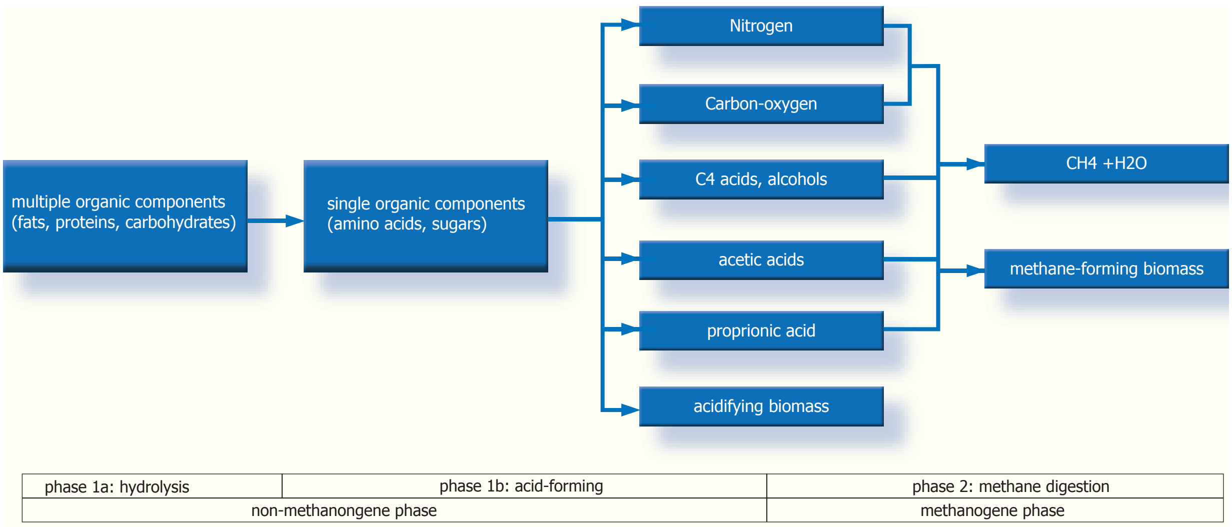


Figure 5.4 - Schematic depiction of the degradation stages with sludge digestion

In normal situations the methanogens are abundantly capable of transferring the formed intermediate products. If however a disturbance occurs in the methane formation, the acidification

will continue and then due to the low pH the methane-forming is further ceased. Then an accumulation of the intermediate products comes about, the so called acidification.

In sludge digestion systems in general the hydrolysis of slowly degradable macro molecules is the limiting process for the ultimate sludge degradation. For obtaining a higher gas production a higher yield is necessary from the first stage of the degradation (the hydrolysis) and not from the actual methane formation.

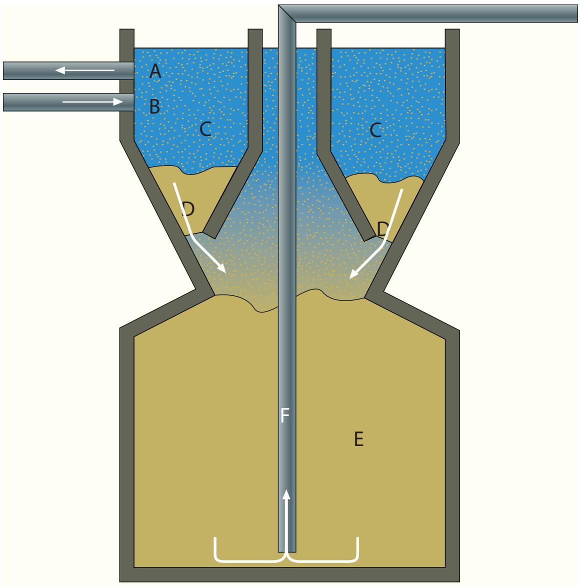
On the other hand, the sensitivity and the low growth of the methane bacteria determine the required process conditions. This is true namely for temperature, pH, minimal residence time and the absence of toxic substances. If these factors are unfavourable, the gas production can come to a stop. The acidification products accumulate and the feared reactor acidification sour digester' occurs.

5.2.3 Construction Forms (Historical)

One of the simplest constructions of the sludge digestion process is the septic tank (figure 5.5); the purpose of the septic tank is to separate the settleable (and floating) solids from the domestic wastewater, followed by a (partial) anaerobic digestion of these solids.

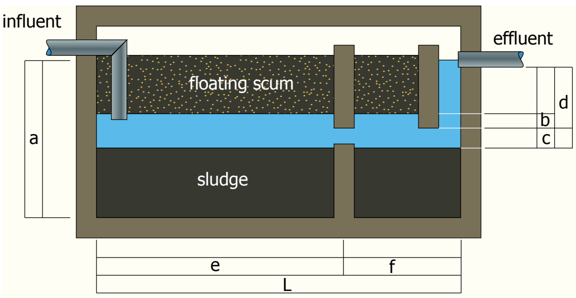
An improved septic tank has space in which sedimentation and digestion takes place in two separate parts. Examples of this are the Imhoff tank and the Clarigester (see figures 5.6 and 5.7).

Later on, from 1930, construction separate digestion tanks became common. These tanks can be heated which allows the process to go faster; also the mixing can be influenced. In



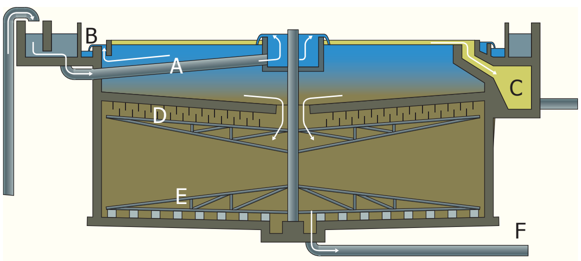
A. Discharge pipe sludge water
B. Supply pipe
C. Sedimentation room
D. Sludge discharge opening
E. Sludge digestion tank
F. Sludge discharge pipe

Figure 5.6 - Imhoff tank



a. liquid level
b. minimal 7.5 cm
c. minimal 30 cm
d. 40 % of the liquid level
e. 2/3 L
f. 1/3 L

Figure 5.5 - Septic tank



A. Supply pipe
B. Discharge Centrate
C. Discharge floating scum
D. Floating scum baffle
E. Scraper
F. Sludge discharge pipe

Figure 5.7 - Clarigester

the first instance digestion tanks were built in which the water separation took place within the digestion tank applying regular settling phases within the digestor (see figure 5.8).

Finally this was changed to intensive mixing of the contents in the sludge digestion tank; the effluent of this tank must then still be thickened in a post

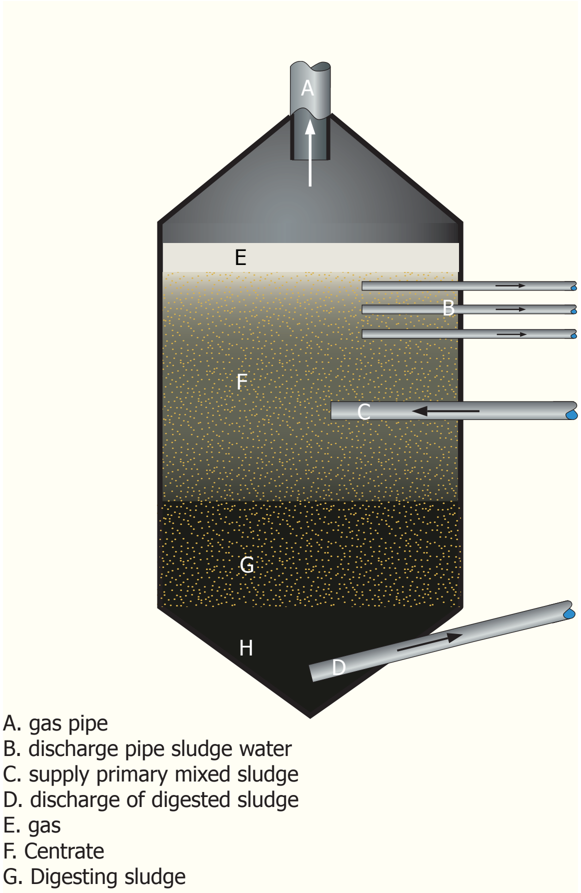


Figure 5.8 – Sludge digestion with water removal
1. supply primary mixed sludge, 2. discharge of digested sludge, 3. discharge of centrate, 4. discharge of gas

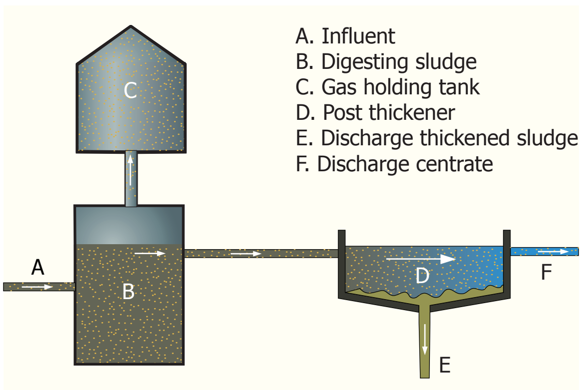


Figure 5.9- Intensive mixed digestion tank

thickener. The digester gas is collected and lead to a gas holder (see figure 5.9).

5.2.4 Solids Degradation

In the sludge digestion process organic sludge particles are converted into anaerobic microbial cell material (very small quantities), digester gas and water. Macroscopically we we observe a reduction of the total amount of suspended solids along with a corresponding production of digester gas. The degradation of the organic matter (40 – 60%) manifests itself in an increase of the ash content and a reduction of the total suspended solids content of the sludge. It is essential to understand that during the sludge digestion processes inorganic sludge is not affected (no degradation, no deposition in the tank). After the sludge digestion the digested sludge allows itself to be moderately thickened (to 4 - 6% ds).

5.2.5 Dimensioning

The most important parameter of dimensioning or designing is the residence time, which is extremely dependent on the applied temperature. In general a temperature of 30 - 35˚C is applied with a residence time of approximately 20 days. (see figure 5.10)

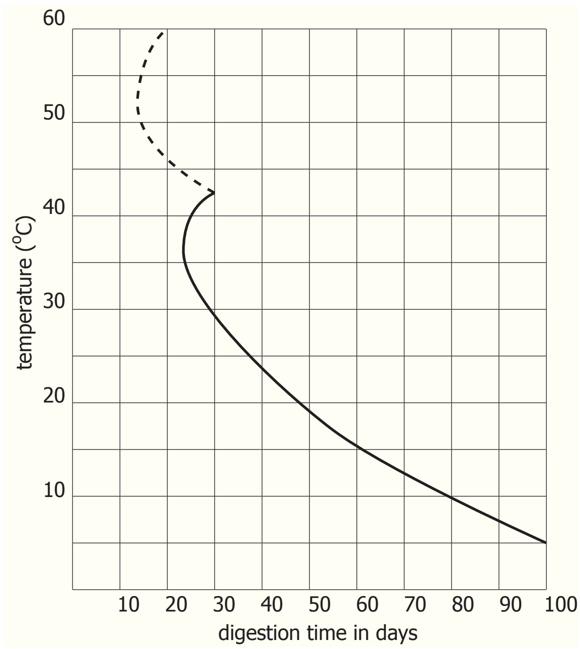


Figure 5.10 – Connection between the digestion temperature and the digestion time

If the daily amount of sludge is known, then the necessary volume of the digestion tanks is determined. This means that it is very important to have thickened the mixed primary sludge as much as possible (4 - 5% ds minimal).

5.2.6 Forms of Construction

Digestion Tank

With the sludge digestion process that is mainly implemented at larger wwtps, the sludge digestion tanks are mostly large in terms of volume (1,000 – 10,000 m^3). These tanks are mostly situated on the ground level. The height to diameter ratio is usually 1:1 - 1:1.5. The tank walls are insulated well to limit the heat loss.

Sludge Supply

The mixed sludge supply is spread regularly throughout the day with the help of a sludge pump (with large conveying height: approximately 15 to 20 m). The level in the digestion tanks is kept constant via an overflow weir. Simultaneous to the supply, discharge of the digested sludge takes place.

Heating

The heating of the tank contents occurs in general through the transference of heat by circulating hot water by means of heat exchangers (see figure 5.11). The heat exchanger is usually placed outside the digestion tank.

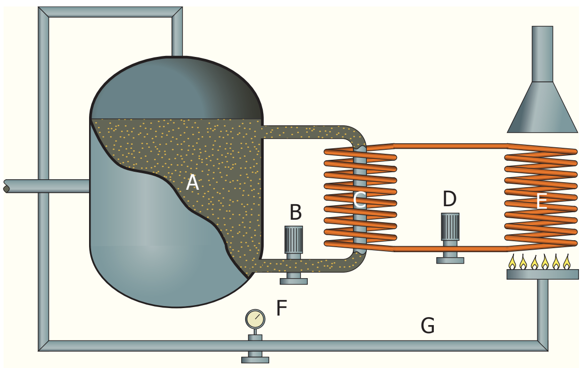
Heat exchangers for heating the sludge must be constructed in this way, so that there is little pollution and after pollution does occur it can be simply and quickly cleaned up. As a result of possible sludge deposition on the sludge side of the heat exchanger, clogging can occur as well as reduction of the heat transfer.

Gas Purging

Mixing the tank contents takes place mostly by blowing in biogas. By means of a gas compressor the digester gas is compressed and blown in under in the tank as larger bubbles. The agitation caused by bubble flow ensures proper mixing this.

Generating Energy

The digester gas can be employed to provide the necessary heat, but can also be used to generate electrical energy (cogeneration /combined heat and power CHP). This happens by means of gas motor-generator combinations, in which the produced electricity generally at the wwtp is used (providing in 50 to 70% of the total needs). The heat produced with this CHP is used agian to warm up the digestion tank is put to use.



A. Digestion tank
B. Sludge pump
C. Heat exchanger
D. Water pump
E. Hot water heater
F. Measuring and safety apparatus
G. Gas pipeline

Figure 5.11 - Heating with sludge digestion

5.3 Sludge Processing

For more information see OD Chapter 9

5.3.1 General

After the stabilization, sludge dewatering takes place. This occurs with especially developed equipment such as centrifuges, belt filter presses and filter presses. The sludge is not directly suited for dewatering, but must first be conditioned. This is accomplished by adding chemicals (polymeric flocculants or ferric chloride) or by heat treatment. After dewatering the sludge has a solid content of 20 to 30 (weight) %.

As last step in the reduction of volume many thermal processes may follow, such as drying and incinerating. This is done at regional level at large centralized installations. The solid residue is dumped or treated to become construction material for example for road construction.

5.3.2 Conditioning

Chemical Conditioning

Adding chemicals to improve the dewatering ability:

- inorganic types: lime and ferric chloride (300 g/kg ds)
- polymeric cation-active flocculants (3 to 8 g/kg ds)

Thermal Conditioning

Heating at a high temperature (200 ˚C) and pressure; with or without adding air. A part of the sludge goes in the solution.

5.3.3 Sludge Dewatering

The purpose of dewatering sludge is extended volume reduction of the sludge by driving out the bound water. The dewatered sludge is discharged and further treated (incinerated, dried, dumped, cement industry).

In sludge dewatering in the Netherlands the following is applied:

- centrifuges

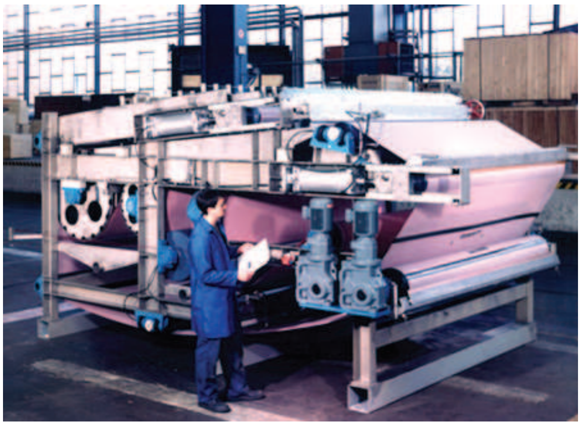


Figure 5.12– Belt filter press

- sieve belt presses
- and a few chamber filter presses.

Digested sludge (capable of dewatering until approximately 27% ds) is better suited for dewatering than fresh mixed primary and secondairy sludge (until approximately 23% ds).

Centrifuge:

Attainable suspended solids content =
approx. 25% ds
Poly electrolyte-usage =
approx. 10kg active Polyelectrolyte/ton ds
Energy usage =
approx. 100 kWh/ ton ds
Production and maintenance =
average (0.4 fte)

Belt Filter Press:

Attainable suspended solids content =
approx. 25% ds
Poly-electrolyte-usage =
approx. 7kg active Poly-electrolyte/ton ds
Energy usage =
approx. 80 kWh/ton ds
Production and maintenance =
average (0.2 fte)

Chamber Filter Press

When dewatering with a chamber filter press the solids content of the sludge is increased from 25% (undigested, fresh, sludge) to 30% (digested sludge). The chamber filter press is made up of plates with an open edge and an escape hole in

the middle. Each plate is wrapped with an artificial cloth. When the pressing starts up, the plates slide against each other. As a result of this and the open sides, a space of about 1.5 cm deep appears between the plates, these are the so called chambers. On the closed plate packet there is a pressure equivalent to approximately 360 bar. The chambers are pumped up with sludge by means of the mohno-pumps (a low and high pressure pump) through the filling hole. The rotational speed of these pumps is controlled by a reduction gear box (variator). A solution of lime, ferric chloride or poly-electrolytes (PE) is added to this sludge to enhance the dewatering capacity.
Attainable solids content =
approx. 35% ds
PE-usage =
approx. 5kg active PE/ton ds
Energy usage =
approx. 10 FecCl/ton ds and or wood chips
Production and maintenance =
intensive (minimal 1 fte).

5.3.4 Thermal Treatment

- drying, direct or indirect, suspended solids 20%→95%
- incinerating, fluidised bed furnace, cleaning degassing, possibly in combination with drying, ash content
- melting, processing ash content, vitrification.

5.3.5 Sludge Reduction

Over the last few years there has been more interest in the development of new techniques to reduce the amount of excess sludge even at the activated sludge process level; this is due mostly to reduce the costs of sludge treatment (approximately € 500 per ton). One of the techniques that is in use in the United States is the Cannibal system by Siemens Water Technologies (figure 5.13). This technique is based on implementing a so called exchange reactor that is placed parallel to the return sludge line, and where by batch processing an amount of return sludge is treated in alternately aerobic and anaerobic conditions. These alternating conditions make sure that facultative aerobic

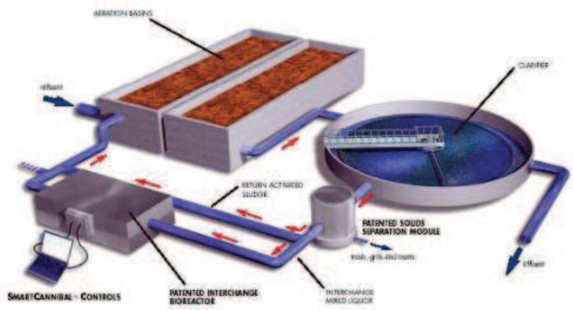


Figure 5.13 - Cannibal

bacteria grow in the reactor; these degrade the digested aerobic bacteria and convert their decomposition products into biodegradable organic compounds that in their turn will be consumed in the aeration tank. Using this technique, a sludge reduction of around 70% is attainable.

6 Practical Considerations

6. Practical Considerations

6.1 Operation

6.1.1 Introduction

Given that in the treatment of wastewater a large number of processes are involved, a good and efficient operation is essential for optimum performance of all the treatment plants. Mostly the plants are automated to a large extent. The operation itself focusses mostly on regulating the processes and when necessary adjusting the controlling parameters.

In terms of the operation various issues are important such as:

- energy;
- chemicals;
- personnel;
- environmental footprint /environmental hindrance (odour and noise);
- discharge of waste materials such as sludge, grit and screen waste;
- automation.

A few of these aspects will be discussed here in more detail.

6.1.2 Energy

Energy is used in many places throughout the treatment process. Mostly this is in the form of electricity which drives the engines.

With this two aspects are important:

- connection load; this corresponds to the power that is drawn from the net's capacity;
- average usage; this depends on the number of operating hours (sometimes 24 h/d) and the average power of an engine.

In total the installed power capacity can be roughly estimated at 7-10 W/PE; with a diversity factor of 0.7 means a maximal power demand (connection load) of 5-7 W/PE. The average usage lies at approximately 25 kWh/PE.j or that is to say at 2.5-3.0 W/PE.

Most important usage (approx. 60%) is created by the energy needed for the aeration process (1.5 – 2.0 W/PE). Here, optimal implementation can allow for great energy savings.

Important energy usage also lies naturally in the pumping of water and sludge (10 to 20% of the total). The necessary pump capacity can be approximated with:

$$N_p = Q \cdot H / (367 \cdot \eta)$$

where

N_p = pump capacity in kW

Q = flow in m^3/h

H = pump head in m (liquid column)

η = pump efficiency (mostly 0.6 à 0.7).

Calculation example

wwtp of 100,000 PE, influent pump

average supply 15,000 m^3/d

maximal supply 4,000 m^3/h

pump head 10 m

efficiency 0.7

=> installed capacity 4000•10/(367•0.7) = 156 kW

=> energy usage 213,000 kWh/j ~ 24 kW specific 2.1 kWh/PE j ~ 0.24 W/PE

Other components continuously running equipment concern

- screen (rake) 2 to 3 KW
- grit chamber (scraper/washer) 1.5 to 2 kW
- scraper sedimentation tank 0.5 to 1.5 kW
- scraper thickener 1 to 1.5 kW

6.1.3 Chemicals

At wwtps chemicals are, to a limited extent, deployed. For example aids for chemical phosphorus removal, sludge dewatering and (incidental) disinfection.

The chemicals involved naturally require their own specific treatment. An estimation of the usage can only take place for the specific processes. In general it can be said that per person equivalent (PE)

- phosphate removal
  iron sulphate 3-6 kg/j
- sludge dewatering
  polyelectrolyte 0.1 kg/j

From this it seems that amount of chemicals is limited, this as a result of implementing physical and biological processes.

6.1.4 Personnel

Most all wwtps run unmanned. For the daily duties such as cleaning, checking, operations, maintenance and local administration only a few staff members are necessary. In general there is a filling in of 0.2 to 0.4 staff member per 10,000 PE.

Aside from this there are jobs that are outsourced by central services; under this fall reparations, large maintenance, laboratories, technological aspects, computerization, and administration. These activities are mostly classified within the central services by the treatment managers. The total machinery of a wastewater treatment board, that naturally has more than one function, is in terms of its size in estimation around 0.8 to 2.0 staff per 10,000 PE.

6.1.5 Environmental Damage

A wwtp today has to meet very severe demands in terms of the hindrance of odour and noise. For this mostly very extreme measures are necessary. This usually refers to covering, insulating, treating of gaseous effluent and the like.

6.1.6 Waste Products

The most important waste product that is produced at a wwtp is the sludge. In terms of the amount the degree of dewatering naturally plays an important role. Even the demands that are set by the market, determine to a large degree the result.
The largest part of the sludge is incinerated in the Netherlands. As final residue approx. 10 kg/(PE year) is left over as ash content with a water content of 10 to 15%; this product will need to be dumped (or perhaps a useful purpose should be found – vitrification phosphate recovery?)

6.1.7 Automation

As it is already mentioned, the implemented processes are mostly continuously in use. This requires not only a large degree of robustness and reliability but also the implementation of automation. The process of automation and computerization is in full swing; an important aspect with this is the management of flow of information that goes to the central registration units. Data management and process control are usually handled by means of so called SCADA (Supervisory Control and Data Acquisition) systems.

6.2 Design Process

6.2.1 Introduction

In the design and treatment techniques many issues play a role. Firstly the whole process has a multidisciplinary character, in which civil and mechanical engineering, technical processes, chemical and biological expertise are brought in. That is why a structured approach is vitally important.

6.2.2 Project Management

To get a good structure for the design and realization process, it is necessary to apply the principles of the project management.
This means:

- planning in phases
  Mostly the following phases are distinguished:
  - initiative (idea, master plan, treatment plan)
  - definition (programme of requirements)
  - design (preliminary design, final design)
  - preparation (scope, contract documents)
  << - conclude contract (tender, adjudication order) >>
  - execution (realization, inspection)
  - commencement of operations (starting up, maintenance)
  Each phase should be finished with a conclusive document; in successive phases decisions made from the previous phases can not be changed.
- management aspects: time, money, quality information, organisation

6.2.3 Resources

In the design process there are various resources necessary such as

- block diagram
- process flow diagram PFD
- lay-out
- process and instrumentation diagram P&ID
- hydraulic profile
- feasibility studies
- aspects studies
- drawings (outline/detailed)
- standardization
- documents (principle plan, EIS (Environmental Impact Study), project documents)
- schedule

6.2.4 Alternatives/Choices

When designing, making responsible decisions is vitally important. In the first place the choices are of a broad nature; later they are elaborately worked out, and detailed.

Namely the choices in the beginning (system start up, technical implementation, lay-out) determine to a large degree the costs of the total project; 90% of the costs are set in the first 10% of the project time.

Aspects that play a role in the decision making process are:

- pros and cons
- weight of the arguments
- costs
- sensitivity analysis
- environmental impacts

Examples of choices affect

- process start up
- integration of existing components
- type reactor/technical implementation tanks
- hydraulic aspects
- sludge line <=> water line
- sludge treatment
- aeration system
- grit removal
- course/fine waste removal
- odour suppression
- generating energy
- one or more units/components
- depth of the tanks
- construction methods
- altitude of the components
- hydraulic circulation provisions
- safety
- measurements, regulating
- sampleing, tests automation
- pump choice.

6.2.5 Research, Tests

Designs are fed by the experiences wrought by comparable systems or situations. It is important then that not only a good evaluation of these experiences is made but also an accurate extrapolation towards the design in question.

If inadequate details are available or doubts about the process conditions results from this, it is necessary to make experiments beforehand. This can take place on a small scale basis (laboratory) but mostly this occurs actually on a semi-technical scale. Important decisions regarding system choices, factories, feasibility and usability of the specific situation, can make up the basis here. This is namely true for new techniques.

In the realization phase many tests take place, such as

- capacity measurements (pumps, often in the factory)
- important process technical performance such as OC-measurements, sludge dewatering e.g.
- guarantee measurements
- hydraulic measurements (losses, discharge heads).

The start up phase is an important phase in which various issues finally can be optimized. After this an evaluation can take place to the extent that the practical results correspond with the original starting points. Naturally during the implementation of operations many practical experiences are gained. Through careful registration of these and aftercare feedback about the design process can take place.

In addition, computer simulation based on process models is being increasingly used to support plant design and perform scenario studies.

6.4.6 Permits

Important permits for operations at a wwtp refer mostly to ground water withdrawal and Environmental Management Law. In some special cases an EIS (Environmental Impact Study, or Assessment) procedure must be followed.

6.2.7 Civil Aspects

The role of the civil engineer in the total design process is mostly that of an integrator (or manager). This is based on the fact that a large part of the realization operations occur at the civil technical (construction, structural) area; the costs of the structural part amount mostly to more than 60% of the total.

More specific issues affect

- hydraulic (large water flows, headloss, distribution problems)
- constructive design
- detailing components
- cost aspects

More generally the integrating role will come to find its place with system choices and the project management.

6.3 Costs

6.3.1 Introduction

The treatment of wastewater is a continuous process, in which the costs and the passing on of these are very important. It must be clear that with these there are two types of costs, one time costs, investment for the realization of the project and continuous costs, being the operational costs that return daily.

6.3.2 Investments

In realizing a project one time costs are made: (CAPEX – Capital Expenditures) . These concern the physical building or construction, engineering and also various other issues such as permits, taxes and financial aspects.

A broad break down is given below.

- building costs 100
- advice/supervision 15-20
- permits, insurances 1-3
- taxes (VAT) 20
- interest during the construction 5-10
- unexpected costs 10-20

The investments depend of course to a large degree on the implemented processes. The investment cost level per PE currently amounts to € 300 to € 600 (all in).

6.3.3 Operating Costs

The operating costs (OPEX – Operating Expenditures) encompass all the costs that are necessary to allow the installation to function well. Under this fall

- maintenance; these can vary during the life time of a project; average values are
  - 0.5% annual of the building costs of the civil technical issues
  - 2.0% annual of the acquisition costs of the electrical mechanical installations
- personnel
- energy, especially electricity
- chemicals
- sludge handling and discharge

The total operating costs amount to approximately € 10 to € 15 per PE annually.

6.3.4 Total Costs

Often knowing the ‘total’ costs of a project is preferred. For this usually two methods are employed.

Operating Costs

With this the investment costs such as annual capital burdens are taken into consideration; on top of that the operational costs are added.
That is to say ‘interest + depreciation’ + ‘operational costs’.
Interest and depreciation can be linearly described on an annual basis; as a depreciation period of 30-40 years is kept for civil technical, 15 years for mechanical engineering and 10 years for electrical components.

Cash Value

With this all the costs during the running of the project can be calculated back (realized) to the beginning. It is important to which degree inflation and interest develop (sometimes a real interest of 2-3% is set). Also the development of energy prices is very important.
For important choices the variables or alternatives must be calculated in this way. In general a preference is shown for the cash value method.
The total operating costs currently amount to € 25.00 to € 40.00 per PE annually; this comes up to approximately € 0.5/m^3 wastewater.

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