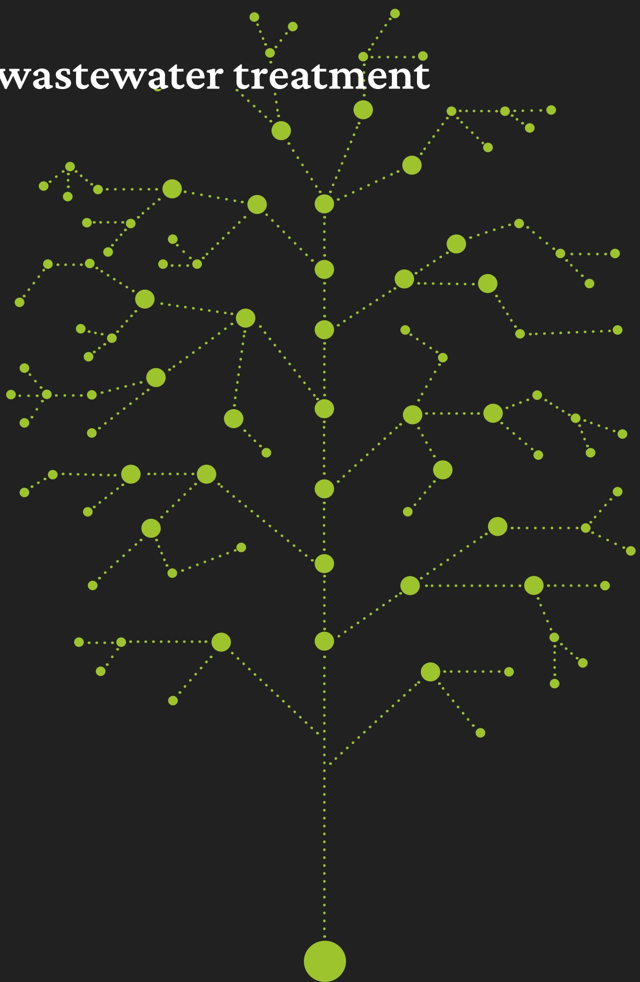
environmental engineering

Introduction to wastewater treatment



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Kraków 2020

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TABLE OF CONTENTS

1. WASTEWATER CHARACTERISTICS

Discharges of untreated or insufficiently treated municipal wastewater have a major impact on the quality of surface water. Modern wastewater treatment plants allow for the effective removal of all major types of pollutants present in wastewater in order to protect the aquatic environment. However, the wastewater management strategy, technologies, and treatment processes must be rationally chosen, correctly designed, and operated appropriately. This book introduces the fundamental concepts related to wastewater treatment and discusses major wastewater management strategies that can be used to prevent surface water pollution on small and large scales. The methods traditionally used for treatment of different types of wastewater are presented, as are new technologies aimed at improving treatment efficiency and increasing the energy recovered from wastewater.

1.1. TYPES OF WASTEWATER

All types of human activity require access to clean water. Water is used in households, public institutions, commercial facilities, industry, and agriculture for different purposes and in different amounts. The majority of consumed water is returned to the environment in a polluted form as wastewater. Wastewater carries pollutants that are characteristic of the type of water use. The composition of domestic wastewater produced in bathrooms and kitchens is rather uniform and very different from the wastewater generated in industrial processes, which can contain a wide range of different pollutants typical of specific industrial processes. Usually, the following types of wastewater are distinguished:

- **Domestic wastewater** is generated by typical human daily activities in the form of effluents from bathrooms and kitchens in households, public institutions, and commercial facilities.
- **Industrial wastewater** originates in various production processes in all types of industry and can contain different pollutants which are sometimes very harmful to the environment even in small amounts.
- **Runoff** from hard surfaces includes rain water, melting snow, and polluted water from street washing.

Modern municipal sewage systems are designed in such way that a mixture of domestic and industrial wastewater (often called '**municipal wastewater**') and runoff is collected in separate drainage systems ('**separate sewerage systems**'). Municipal wastewater is collected in a **sanitary sewerage system** and transported to a wastewater treatment plant for further processing. Although runoff is meant to be collected in a rainwater drainage system, some of it often gets into the sanitary system due to **incidental flows** or illegal connections. On its way to a wastewater treatment plant, municipal wastewater is usually further diluted by underground influents leaking through the sanitary sewers.

Runoff collected in rainwater drainage systems is usually only roughly treated to remove sediments. During an intensive rainfall, it is discharged directly into a nearby river or stream through storm overflows. There are still some sections of old sewerage systems—or even whole systems—where municipal wastewater and runoff are collected together in the same drainage system (a '**combined sewerage system**'). The existence of such system must be considered when designing and operating a wastewater treatment plant, due to the increased variability of flow and the greater dilution of incoming wastewater in such situations.

1.2. QUANTITY OF WASTEWATER

Any design work on wastewater treatment facilities is based on the measurement of quantity of wastewater generated in a specific drainage area and its qualitative characterisation. The overall quantity of municipal wastewater is the sum of domestic wastewater generated (approx. 90% of water consumed by humans is discharged as wastewater), wastewater discharged from industrial activities, incidental and illegal discharges, and infiltration from groundwater into sewers. The proportion of individual sources depends on the local area, the size of the community, weather conditions, legal regulations, and water use standards. However, the quantity of wastewater is usually determined based on dry weather conditions (no rain) with the following measures:

– Average daily flow rate Q_{dav}, the mean of daily flow rates obtained from long-term measurements—for example, over the period of one year—expressed in m^3 per day:

$$Q_{dav} = \frac{\text{sum of daily flows}}{\text{number of days considered}} \left[\frac{m^3}{d}\right]$$

– Average hourly flow rate Q_{hav}, the average daily flow rate divided by 24 hours and expressed in m^3 per hour:

$$Q_{hav} = \frac{Q_{dav}}{24} \left[\frac{m^3}{h}\right]$$

- Maximum daily flow rate $Q_{d\max}$, the largest daily flow rate over the observed period (e.g. one year). The quotient of maximum daily flow rate $Q_{d\max}$ and average daily flow rate Q_{dav} is called the daily maximum irregularity factor $N_{d\max}$:

$$N_{d\max} = \frac{Q_{d\max}}{Q_{dav}}$$

- Maximum hourly flow rate $Q_{h\max}$, the largest hourly flow rate observed during the day of the largest daily flow ($Q_{d\max}$) over the observed period (sometimes also called peak hourly flow rate). The quotient of maximum hourly flow rate $Q_{h\max}$ and average hourly flow rate Q_{hav} is called the hourly maximum irregularity factor $N_{h\max}$:

$$N_{h\max} = \frac{Q_{h\max}}{Q_{hav} \cdot N_{d\max}}$$

Consequently, the relationship between the maximum hourly flow rate $Q_{h\max}$ and the average daily flow rate Q_{dav} can be expressed as follows:

$$Q_{h\max} = \frac{Q_{d\max}}{24} \cdot N_{h\max} = \frac{Q_{dav}}{24} \cdot N_{d\max} \cdot N_{h\max} \left[\frac{\mathrm{m}^3}{\mathrm{h}}\right]$$

- Minimum daily flow rate $Q_{d\min}$ is the smallest daily flow rate over the observed period (e.g. one year).
- Minimum hourly flow rate $Q_{h\min}$ is the smallest hourly flow rate observed during the day of the average daily flow (Q_{dav}) over the observed period. The quotient of minimum hourly flow rate $Q_{h\min}$ and average hourly flow rate Q_{hav} is called the hourly minimum irregularity factor $N_{h\min}$:

$$N_{h\min} = \frac{Q_{h\min}}{Q_{hav}}$$

- Average hourly flow rate during daytime hours Q_{hd} is the mean of the 12 largest consecutive hourly flow rates between 5.00 am and 9.00 pm observed during the day of the largest daily flow ($Q_{d\max}$). The quotient of average hourly flow rate during daytime hours Q_{hd} and the average hourly flow rate Q_{hav} is called the daily irregularity factor for day hours N_{hd}:

$$N_{hd} = \frac{Q_{hd}}{Q_{hav}}$$

Typical values of the irregularity factors are in the following ranges: $N_{d\max} = 1.3–2.0$; $N_{h\max} = 1.5–4.0$; $N_{h\min} = 0.3–0.6$; and $N_{hd} = 1.3–2.4$ [9]. An example variation of wastewater flow rates for a typical medium-sized city is presented in Fig. 1.

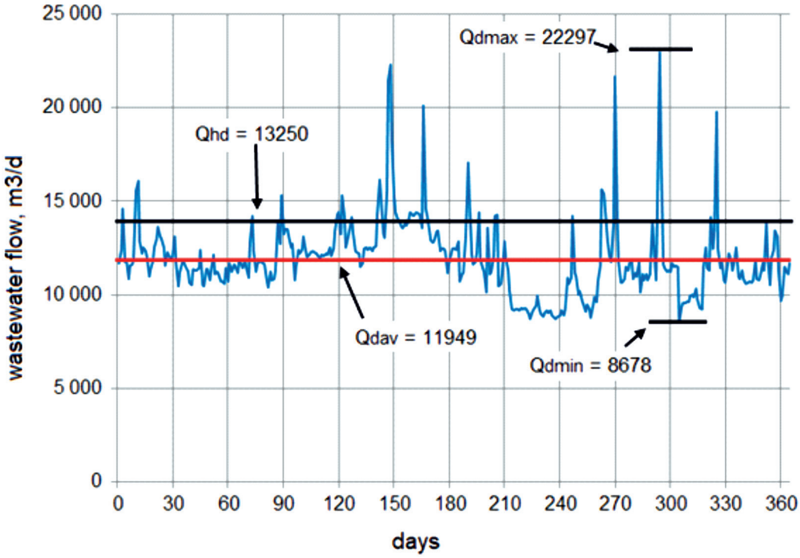

Fig. 1. Wastewater flow variation during a year and typical flow measures

1.3. THE COMPOSITION OF WASTEWATER

Pollutants can be found in municipal wastewater in suspended, colloidal, and dissolved forms. The **total solids (TS)** includes the combined mass of both the suspended and dissolved forms. The sum of pollutants in suspended form is called the **total suspended solids (TSS)**. The TSS comprises both organic pollutants described as **volatile suspended solids (VSS)** and inorganic suspended material; that is the difference between TSS and VSS. Regardless of the pollutant form, wastewater composition is characterised by pollution indicators that can be categorised into several groups: organic, nutrients, microbiological, and other (inorganic, gasses, heavy metals, micropollutants, etc.).

1.3.1. ORGANIC POLLUTANTS

The category of **organic pollutants** includes a wide variety of different chemical compounds, and it is therefore described by indirect indicators such as **biochemical oxygen demand (BOD)** and **chemical oxygen demand (COD)**. BOD is a measure of the amount of dissolved oxygen needed by aerobic biological organisms to decompose the organic material (carbonaceous BOD) and ammonia (non-carbonaceous BOD) present in a given water sample at certain temperature over a specific time period. Usually, a test is carried out over a period of 5 days (BOD_5). During this time, about 68% of the organic material is decomposed. In 20 days (BOD_{20}) almost all biodegradable organic material (99%) is exhausted.

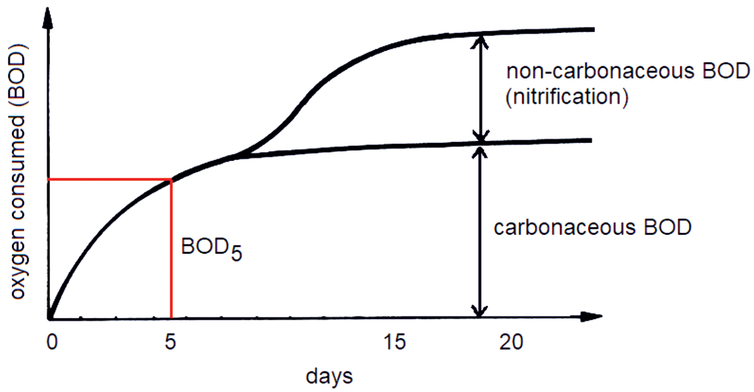

Fig. 2. Carbonaceous and nitrogenous biochemical oxygen demand

COD is a measure of almost all organic compounds that can be fully oxidised to carbon dioxide with a strong oxidising agent (such as e.g. potassium dichromate) under acidic conditions. The COD value is always larger than the BOD value for the same sample. A major advantage of the COD test over the BOD one is that such test can be completed in minutes instead of taking several days. The ratio of COD to BOD_5 in raw wastewater can be used as a rough measure of the wastewater's susceptibility for biological treatment. Typical values of the COD/BOD_5 ratio in raw municipal wastewater are between 1.5 and 2.5 [9]. Another measure of organic pollutants is **total organic carbon (TOC)**. The TOC is measured by oxidising the organic carbon to CO_2 and H_2O under high temperature (680–950°C) and measuring the amount of CO_2 gas produced using an infrared carbon analyser.

The major advantage of COD over the other measures of organic matter (BOD, TOC, and TSS) is that it directly measures the demand for **electron acceptors** for oxidation of organic material present in the wastewater, which allows for easy determination of the oxygen demand for the processes of biological treatment of the wastewater. The detailed characteristics of wastewater with COD can provide valuable information about the forms of organic compounds and their structure, which can help in designing efficient treatment processes. The organic compounds forming total COD can occur in dissolved, colloidal, or suspended forms and can be readily biodegradable or non-biodegradable. These are called the '**COD fractions'**. In its simplified form, the total COD consists of the following fractions:

$$\text{Total COD} = S_S + S_I + X_S + X_I + X_H$$

where

S_S – readily biodegradable soluble fraction of total COD,
S_I – non-biodegradable soluble fraction of total COD,
X_I – non-biodegradable suspended fraction of total COD,
X_S – slowly biodegradable suspended fraction of total COD, and
X_H – biomass of heterotrophic bacteria as a fraction of total COD.

The proportions of individual COD fractions can be determined by measuring COD in filtrated and coagulated wastewater samples and executing a series of respirometric tests (oxygen uptake rate) on wastewater samples. Although the structures of COD fractions vary among different wastewaters, and it changes during the treatment process (Table 1), for raw municipal wastewater the average values are as follows: S_S – 17%; S_I – 7%; X_S – 46%; X_I –17%; X_H – 13%; and volatile fatty acids (VFA) – approx. 4%.

Table 1

Summary of COD fractions in raw wastewater in different countries [17]

Country, region	S_I %	S_S %	X_I %	X_S %	X_H %	$X_S(+X_H)$** %	VFA %	References
N. America	10.5	14.1	27.9			44.3	0.0	Hydromantis, 2007
S. Africa	5.0	20.0	13.0			62.0		Ekama *et al.*, 1986
Switzerland	14.0	9.0	9.0	56.0	12.0	68.0		Kappeler and Gujer, 1992
Denmark	2.0	20.0	18.0	40.0	20.0	60.0		Henze, 1992
Sweden	15.0	27.0	17.0	33.0	8.0	41.0		Xu and Hultman, 1996
Denmark	7.6	20.3	13.0	51.5	7.2	58.7	8.1	Gernaey and Jorgensen, 2004
Denmark	5.0	35.0	10.0	35.0	15.0	50.0		Chachaut *et al.*, 2005
N. America	12.0*	15.0	14.5*			59.0	1.4	Melcer *et al.*, 2003
Netherlands	6.0	26.0	39.0			28.0		Roeleveld and Loosdrecht, 2002
N. America	5.0	16.0	13.0			66.0	2.4	EnviroSim, 2005
France	4.1	3.0	19.0			73.9		Marquot *et al.*, 2006
Germany	6.4	18.3	11.3	49.3	14.7	64.0		Wichern *et al.*, 2003
Germany	6.1	14.8	13.0	55.4	10.8	66.2		Wichern *et al.*, 2001
Italy	6.0	15.0	8.0	56.0	15.0	71.0		Carucci *et al.*, 1999
Spain	8.5	18.3	24.9	33.3	15.0	48.3		Del la Sota *et al.*, 1994
Denmark	10.0*	15.0*	20.0	40.0*	15.0	55.0*	8.8	Henze, 1992
Switzerland	4.0	10.0	20.0	54.1	11.9	66.0		Rieger *et al.*, 2001
Average	7.5	17.5	17.1	45.8	13.1	57.9	4.1	

* Estimate based on the article; ** Biomass COD is included in the slowly biodegradable substrate fraction

1.3.2. NITROGEN AND PHOSPHORUS

Nitrogen and phosphorus compounds are essential to the growth of biological organisms, which is why they are often called **nutrients**. They are present in wastewater in both organic and mineral forms. The principal sources of nitrogen in municipal wastewater are nitrogenous compounds of plant and animal origin, sodium nitrate, and atmospheric nitrogen [11]. The mineral forms of nitrogen are usually **ammonia ions** (NH_4^+) or **free ammonia** (NH_3), which exist in equilibrium depending upon the pH value. Other inorganic forms of nitrogen, such as **nitrates** and **nitrites**, are present in municipal wastewater at low concentrations. The sum of ammonia nitrogen, nitrates, nitrites, and organic nitrogen (in soluble and suspended forms) yields the **total nitrogen** (total N).

The most common forms of inorganic phosphorus in wastewater are soluble **orthophosphates** and **polyphosphates**. The orthophosphates may be present in different forms (PO_4^{3-}, HPO_4^{2-}, $H_2PO_4^-$ and H_3PO_4) and, unlike polyphosphates, they are available for biological metabolism without further breakdown. Polyphosphates can be hydrolysed to the orthophosphate form, but the process is very slow [11]. The sum of orthophosphates, polyphosphates, and organic phosphorus yields the **total**

phosphorus in wastewater (total P). Municipal wastewater usually contains between 4 and 16 g/m^3 of total phosphorus, the majority of which (approx. 70%–80%) is in the form of orthophosphates.

Table 2

Definition of various form of nitrogen present in raw wastewater (adapted from Metcalf and Eddy)

Form of nitrogen	Abbreviation	Definition	Typical % of total nitrogen
Ammonia gas	NH_3	NH_3	depends on pH
Ammonia ion	NH_4^+	NH_4^+	depends on pH
Total ammonia nitrogen		$NH_3 + NH_4^+$	60–80
Nitrite	NO_2	NO_2	1–2
Nitrate	NO_3^-	NO_3^-	1–3
Total Kjeldahl nitrogen	TKN	Organic N + NH_3 + NH_4^+	95–98
Organic N	Organic N	Soluble organic N + particulate organic N	15–35
Total nitrogen	Total N, TN	Organic N + NH_3 + NH_4^+ + NO_2 + NO_3^-	100

1.3.3. MICROBIOLOGICAL POLLUTANTS

Municipal wastewater also contains bacteria, viruses, protozoa, fungi, zooplankton, phytoplankton, and other pathogenic microorganisms. The microbiological pollution in wastewater may pose a serious risk to employees at wastewater treatment plants and to the environment. A commonly used indicator of bacterial contamination of wastewater is the number of **total coliform bacteria** that are always present in the digestive tracts of animals, including humans, and are found in their excreta. **Faecal coliforms** make up the group of total coliforms that are considered to be present specifically in the gut and faeces of warm-blooded animals. Because the origins of faecal coliforms are more specific than the origins of the more general total coliform group of bacteria, faecal coliforms are considered to be a more accurate indication of the presence of animal or human waste. *Escherichia coli* is the main species in the faecal coliform group. Of the five general groups of bacteria that comprise the total coliforms, only *E. coli* is generally not found growing and reproducing in the environment. Consequently, *E. coli* is considered to be the species of coliform bacteria that is the best indicator of faecal pollution and the possible presence of pathogens.

1.3.4. OTHER POLLUTANTS

Other pollutants that may be present in municipal wastewater include various ions, heavy metals, gasses, hazardous organic and inorganic compounds, and various microorganisms. The range of ions may be very wide and depends on the types of compounds present in wastewater, mainly salts such as **chlorides**, **sulphates,** and **hydrocarbons**, as well as **sodium**, **magnesium**, and **potassium**. Some of them (e.g. magnesium, potassium, and calcium) are necessary as microelements for the growth of bacteria and other biological organisms. Sources of metals in wastewater include the discharge from residential areas, groundwater infiltration, and commercial and industrial discharges [11]. Gases present in wastewater include mainly nitrogen (N_2), carbon dioxide (CO_2), hydrogen sulphide (H_2S), ammonia (NH_3), and methane (CH_4). The latter three are produced during the decomposition of organic matter present in wastewater and are of concern in regard to human health and work safety. The array of hazardous compounds that may be present in wastewater is very wide. They may be organic or inorganic in nature and include volatile organic compounds (VOC), by-products of disinfection, heavy metals, pesticides and agricultural chemicals, and a variety of emerging organic compounds. Many of them can cause carcinogenic, mutagenic, teratogenic, or acute toxicity effects.

1.3.5. TYPICAL COMPOSITION OF WASTEWATER

Wastewaters are different, as they originate from various sources and are generated under different economic, social, cultural, and climatic conditions. The population of the served community and the contribution of industrial influents to the sewerage system are also important factors. Moreover, the composition of wastewater fluctuates over time in daily, weekly, monthly, seasonal, and yearly cycles. Therefore, there is no uniform composition of wastewater, even for a specific community. However, based on measurements performed in different regions, an approximation of the typical composition of domestic wastewater can be drawn up as shown in Table 3.

Table 3

Typical composition of domestic wastewater (adapted from Metcalf and Eddy)

Contaminants	Unit	Concentration		
		Low strength	Medium strength	High strength
Solids, total (TS)	mg l^{-1}	390	720	1230
Dissolved, total (TDS)	mg l^{-1}	270	500	860
Suspended solids, total (TSS)	mg l^{-1}	120	210	400
Biochemical Oxygen Demand (BOD_5)	mg l^{-1}	110	190	350
Chemical Oxygen Demand (COD)	mg l^{-1}	250	430	800
Nitrogen (total as N)	mg l^{-1}	20	40	70
Nitrates	mg l^{-1}	0	0	0
Phosphorus (total as P)	mg l^{-1}	4	7	12
Total coliform	No./100ml	10^6-10^8	10^7-10^9	10^7-10^{10}
Faecal coliform	No./100ml	10^3-10^5	10^4-10^6	10^5-10^8

1.4. THE EFFECTS OF WASTEWATER DISCHARGE ON NATURAL WATERS

The majority of medium-sized and large wastewater treatment plants discharge their effluents into inland water bodies, which can cause pollution in the recipient water body. Major water pollutant indicators include oxygen demanding materials, infectious agents (protozoa, bacteria, viruses, and helminths), nutrients (nitrogen and phosphorus), toxic substances, sediments, and hazardous substances. The dissolved oxygen (DO) concentration is a major indicator of a water body's ability to support desirable aquatic life and capacity to assimilate pollution. Therefore, BOD is usually used as an indicator to describe water quality.

The quality of water in a river or a stream varies along its length, starting from the point of pollution discharge. In the **degradation zone**, that is, directly downstream of the discharge point, the concentration of dissolved oxygen decreases, the BOD value in water increases, and the diversity of animal and plant species is reduced. In the next zone, called the **active decomposition zone**, the dissolved oxygen concentration may be decreased further, even reaching zero. Water turbidity and odour increase and the variety of biological life is limited. In the **recovery zone**, the process is gradually reversed and the water returns to normal conditions.

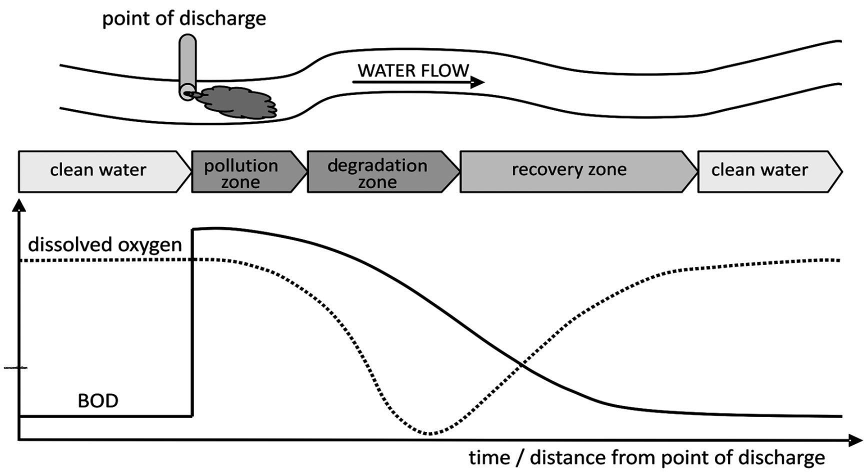

Fig. 3. The effect of organic pollution discharge into a watercourse

The discharge of an effluent with a higher content of nutrients, primarily phosphorus, may cause **eutrophication**, that is, excessive growth and decay of simple algae and plankton. The decomposition process deprives the deeper waters of oxygen and causes a severe deterioration in water quality. A reduced concentration of dissolved oxygen may cause a release of hydrogen sulphide, ammonia, and methane.

1.5. WASTEWATER EFFLUENT STANDARDS

The European legal regulations regarding the discharge of treated wastewater into the environment are largely based on the following assumptions: i) maintaining the suitability of the receiver's water for a specific purpose, ii) protecting the receiver's water and its biocenosis, or iii) limiting the permissible values of contamination indicators in discharged wastewater. In regard to municipal wastewater, minimum requirements on effluent quality are set in the *Council Directive concerning urban waste water treatment (91/271/EEC)* [3]. The directive has been adopted by all EU member countries and incorporated into their national legislations. The **basic requirements** apply to three indicators: BOD_5, COD, and TSS. The requirements are presented in two versions—concentrations and percentages of reduction—as presented in Table 4 and 5. One or both parameters may be applied depending on the local situation.

Table 4

The requirements for discharge from urban wastewater treatment plants subject to Directive 91/271/EEC [3]

Parameters	Concentration	Minimum percentage of reduction (%)[a]	Reference method of measurement
Biochemical oxygen demand (BOD_5 at 20°C) without nitrification[b]	25 mg/l O_2	70–90 40 under Article 4[b]	Homogenised, unfiltered, undecanted sample. Determination of dissolved oxygen before and after 5-day incubation at 20°C ± 1°C, in complete darkness. Addition of a nitrification inhibitor.
Chemical oxygen demand (COD)	125 mg/l O_2	75	Homogenised, unfiltered, undecanted sample. Potassium dichromate.
Total suspended solids	35 mg/l[c] 35 under Article 4[b] (more than 10,000 p.e.) 60 under Article 4[b] (2,000–10,000 p.e.)	90[c] 90 under Article 4[b] (more than 10,000 p.e.) 70 under Article 4[b] (2,000–10,000 p.e.)	– Filtering of a representative sample through a 0.45-μm filter membrane. Drying at 105°C and weighing. – Centrifuging of a representative sample (for at least 5 min with a mean acceleration of 2,800–3,200 g), drying at 105°C and weighing.

[a] The reduction is in relation to the load of the influent.

[b] The parameter can be replaced by another parameter, total organic carbon (TOC) or total oxygen demand (TOD), if a relationship can be established between BOD5 and the substitute parameter.

[c] This requirement is optional.

The directive also requires that the discharge of nitrogen and phosphorus with treated wastewater be regulated in all areas susceptible to eutrophication. The member countries themselves define such areas. The discharge requirements vary with the size of the population (population equivalent [p.e.]) in agglomerations where a given wastewater treatment plant is situated. National regulations are often more detailed and stricter than the resolutions of Directive 91/271/EEC, especially in regard to the removal of nitrogen and phosphorus compounds. For example, in Poland, the entire territory of the country has been defined as eutrophication-sensitive, so all municipal wastewater must undergo treatment for removal of nutrients.[1] The discharge regulations in Poland are in general more stringent than those required by Directive 91/271/EEC, as presented in Table 6.

Table 5

The requirements for discharge from urban wastewater treatment plants to areas which are sensitive to eutrophication [3]

Parameters	Concentration	Minimum percentage of reduction (%)[a]	Reference method of measurement
Total phosphorus	2 mg/l (10,000–100,000 p.e.)	80	Molecular absorption spectrophotometry
	1 mg/l (more than 100 000 p.e.)		
Total nitrogen[b]	15 mg/l (10,000–100,000 p.e.)[c]	70–80	Molecular absorption spectrophotometry
	10 mg/l (more than 100,000 p.e.)[c]		

[a] The reduction is in relation to the load of the influent.
[b] Total nitrogen means the sum of total Kjeldahl nitrogen (organic and ammoniac nitrogen), nitrate nitrogen, and nitrite nitrogen.
[c] These values for concentrations are annual means as referred to in Annex I, paragraph D.4(c). However, the requirements for nitrogen may be checked using daily averages when it is proved, in accordance with Annex I, paragraph D.1, that the same level of protection is obtained. In this case, the daily average must not exceed 20 mg/l of total nitrogen for all the samples when the temperature from the effluent in the biological reactor is higher than or equal to 12°C. The conditions concerning temperature could be replaced by a limitation on the time of operation to take into account regional climatic conditions.

Table 6

The summary of Polish discharge standards for urban wastewater (based on [14])

Parameter	Unit	Maximum permissible values of pollution indicators or minimum percentage of pollution reduction for wastewater discharged into waters or into the ground from wastewater treatment plants (for p.e. < 2,000) or in agglomeration (for p.e. ≥ 2,000)				
		Population equivalent (p.e.)				
		<2,000	2,000–9,999	10,000–14,999	15,000–99,999	≥100,000
Carbonaceous BOD_5	g O_2/m^3 or min. % red.	40	25 or 70%–90%	25 or 70%–90%	15 or 70%–90%	15 or 70%–90%
COD	g O_2/m^3 or min. % red.	150	125 or 75%	125 or 75%	125 or 75%	125 or 75%
Total suspended solids	g/m^3 or min. % red.	50	35 or 90%	35 or 90%	35 or 90%	35 or 90%
Total nitrogen	g N/m^3 or min. % red.	30[*)]	15[*)]	15 or 70%–80%	15 or 70%–80%	10 or 70%–80%
Total phosphorus	g P/m^3 or min. % red.	5[*)]	2[*)]	2 or 80%	2 or 80%	1 or 80%

* only when discharged into lakes and reservoirs

[1] With some exceptions for the smallest treatment units.

2. OVERVIEW OF WASTEWATER TREATMENT STRATEGIES AND TECHNOLOGIES

2.1. CENTRALISED VS DECENTRALISED WASTEWATER MANAGEMENT

The main purpose of wastewater treatment is to protect the natural environment. The environmental and health risks associated with the discharge of treated wastewater to surface water must be minimised. A major issue in the selection of wastewater management strategy is the choice between a centralised and a decentralised system. While the concept of centralised systems is usually well-understood, there is little agreement on the precise definition of decentralised systems. The concept of decentralised systems is often associated with a number of small wastewater treatment plants that serve a defined area using a variety of treatment technologies. This is not always true, as even the definition of a small wastewater treatment plant is unclear and depends on local and regional conditions. Decentralised treatment can be also defined by the fact that raw wastewater is treated next to the source where it is generated.

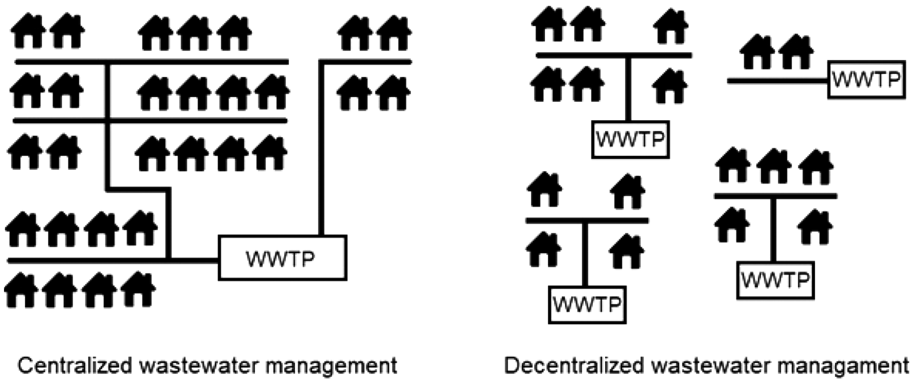

Fig. 4. Centralised vs decentralised approach to wastewater management
WWTP – wastewater treatment plant

Wastewater treatment strategies vary from conventional, highly centralised systems to entirely decentralised ones, with cluster systems placed somewhere in between. The application of a centralised or decentralised approach is often more a matter of historical development and specific local conditions rather than an intentional decision. In developed countries, centralisation is usually the norm in urban areas, while decentralised systems are used in scattered rural settlements. In developing countries, wastewater from houses, businesses, and industry remains untreated or is frequently treated on-site and discharged into the ground or nearby watercourses [8]. Centralised systems usually are publicly owned, serve large populations, and treat large volumes of wastewater with advanced treatment methods. In contrast, decentralised systems treat wastewater from individual houses, groups of houses, and small settlements.

Table 7

Advantages and disadvantages of decentralised and centralised wastewater management systems

<table>
<tr><th>DECENTRALISED</th><th>CENTRALISED</th></tr>
<tr><td colspan="2">ADVANTAGES</td></tr>
<tr><td>Costs: no sewerage system necessary, internal labour possible, no large long-term investment
Better adjustment to the individual level of pollution
Flexible (expandable) and adaptable
Minimal risk of epidemics (separation of pathogens, etc.)
Resource conservation (water and nutrients)
Independence
No concentrated point source input into the ground
Limited damage in case of failure (risk minimisation)</td><td>Convenience for the wastewater producer
Reduced health risk
Simple pollution control
Limited odours and flies
‘Economy of scale’ for treatment in densely populated areas</td></tr>
<tr><td colspan="2">DISADVANTAGES</td></tr>
<tr><td>Less efficiency of treatment (N, P)
Insufficient control
Difficulty in finding qualified personnel for operation and maintenance
Legal framework is more complex
Requirements regarding space and climate
Subjective feelings and acceptance of users
Need for education and correct usage
May affect groundwater</td><td>Mixing of wastewater of all origins and compositions means nutrients are not reused
Water-intensive design: using potable water as a carrier means effluent reaches the plant highly diluted
Need for energy-intensive pumping where there is insufficient slope
Cost ratio of sewer and treatment systems: 4:1 (80% of the costs are in the transport system)
Acute impact in case of malfunction/natural catastrophe</td></tr>
</table>

Decentralised systems increase public awareness and create the potential for water to be reused within the same watershed. They also encourage public participation in water quality protection. The initial investment and operational costs are relatively low, too. On the other hand, the monitoring of small wastewater treatment plants is not as rigorous or strict as it is at large plants. Also, the treatment technologies used at small plants are usually less effective in removing pollution loads, especially in regard to nitrogen and phosphorus. As a result, the combined effect of pollution discharge from a number of small wastewater treatment plants may be harmful for water environment. In centralised systems, the treatment of wastewater occurs far away from the point of origin, so such systems do not encourage public awareness. The need for wastewater transport and pumping increases the investment and operational costs of the system

[27]. The major advantages of a centralised system include flexibility, effectiveness, and controllability. Large wastewater treatment plants are less sensitive to variations in wastewater flow and composition, as the parameters of the technological processes can be adjusted to changing operational conditions. Such plants usually use highly-effective treatment technologies which guarantee high-quality effluent and prevent the pollution of natural water resources. Large plants are usually equipped with advanced monitoring and control systems and are supervised by highly qualified personnel, minimising the risk of incidental water resource contamination.

In many areas, centralised and decentralised systems can exist as complementary solutions. Both treatment approaches have advantages and disadvantages and a case-specific analysis should inform the decision between them. When planning a centralised system, the issue of investment costs should be analysed with special care, as about 80%–90% of the capital costs are associated with the construction of wastewater collection systems [8]. Planners of decentralised systems should be especially concerned with the magnitude of the operational costs that will be placed on the local community and the proper supervision over the management and operation of such plants.

2.2. OVERVIEW OF THE TECHNOLOGY

The required effluent quality can be achieved by applying a sequence of physical, chemical, and biological processes that reduce the concentration of individual pollutants in wastewater. The specific sequence of treatment processes depends on the wastewater treatment plant's size, the raw wastewater's composition, and the specific effluent discharge limits set for each plant. Regardless of the individual conditions, typical municipal wastewater treatment scheme includes mechanical (physical), biological, and chemical treatment processes.

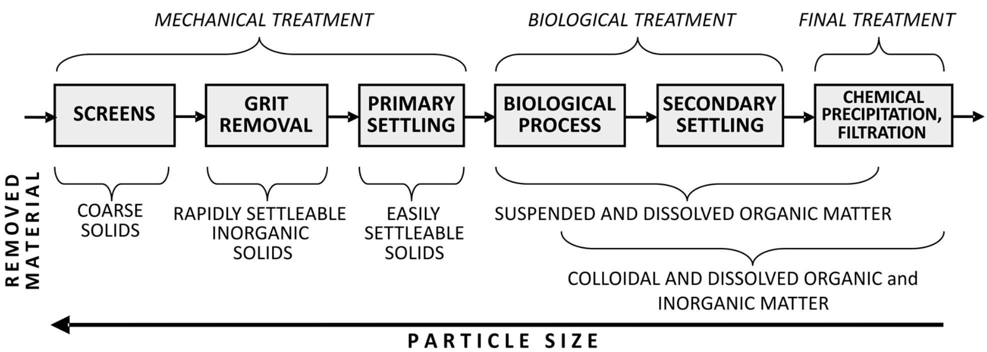

Fig. 5. Flowchart of typical municipal wastewater treatment processes and types of pollutants removed during treatment

2.2.1. MECHANICAL TREATMENT

Mechanical treatment removes only a relatively small mass of pollutants, as the majority of pollutants in raw wastewater are in soluble, colloidal, or suspended forms and are removed during the biological and chemical stages of treatment (see Fig. 5). The purpose of mechanical treatment is to remove from the wastewater large objects, sand and grit, and the easily-settleable fraction of total suspended solids, and therefore to protect further treatment processes from pump damage, clogging, or material being deposited in the biological reactors. This is accomplished by applying a sequence of simple physical processes, such as screening, sieving, and sedimentation.

Large objects—such as paper, bottles, pieces of wood, personal care materials, etc.—are removed in the process of screening through coarse and fine **screens** or sieves. As wastewater flows through the barrier, the objects are stopped at a screen or a sieve that are equipped with a system of automatic cleaning. Inorganic material, such as grit or sand, is removed from the wastewater in the process of sedimentation in **grit tanks**. Grit tanks are volume objects where the flow velocity is small enough to allow large inorganic particles to settle to the bottom. A special scraping and pumping system removes the settled particles from the bottom of the tank. This material is then dewatered and usually disposed of in a landfill. Once cleared of large objects and grit, the wastewater is pumped into **primary settling tanks**. The settlers are large-volume objects where easily-settleable fraction of the suspended solids is removed from the wastewater. As this fraction of suspended solids includes both inorganic and organic material, a reduction in the organic material within wastewater is also achieved. The efficiency of primary sedimentation depends on the hydraulic retention time in the primary settlers, usually varying between 0.5 and 1.2 hr. Primary sedimentation reduces the TSS concentration by about 60%–65%, BOD_5 by approx. 25%–30%, and the total N and P by 5%–10%.

2.2.2. BIOLOGICAL TREATMENT

Biological treatment methods utilise the natural capability of various microorganisms to carry on biochemical conversion of different substances. Microorganisms can be present in the form of a suspension (**activated sludge**) or **biofilm** (attached biomass). Hybrid systems are also used, where biomass is in the form of a biofilm growing on small, cylindrical carrier elements freely flowing in a biological reactor filled with wastewater (**Moving Bed Biofilm Reactors, MBBR**). Large wastewater treatment plants usually use activated sludge methods, while medium-sized and smaller plants may use both suspended or attached biomass systems.

The processes that occur in activated sludge and biofilms are of a biological, chemical, and physical nature and they are very complex. In simple terms, organic pollutants (represented by BOD or COD) are removed from wastewater by heterotrophic microorganisms that use them as a substrate. As microorganisms can use the substrate in soluble form only, particulate organic material must be hydrolysed before it can be consumed. Heterotrophic bacteria require access to oxygen to remove organic pollutants, so the biological reactor's contents must be aerated (activated sludge) or ventilated (biofilm). Enough nitrogen and phosphorus are removed from the wastewater to provide for the metabolic growth of the microbial cells. This process removes only 20%–30% of the N and P load from wastewater, which is usually insufficient to meet the required effluent standards.

Nitrogen can be effectively removed through the process of nitrification and denitrification. **Nitrification** is a process of autotrophic oxidation of ammoniac nitrogen to nitrites (NO_2) and nitrates (NO_3) by nitrifying bacteria, *Nitrosomonas* and *Nitrobacter*. The process occurs under aerobic conditions at sufficiently long **solid retention times (SRT)**. In the process of **denitrification**, nitrates are biochemically reduced to nitrogen gas (N_2) by heterotrophic bacteria under anoxic conditions and in the presence of a carbon substrate. Phosphorus can be effectively removed in the process of **biological excess phosphorus removal**. This process utilises the natural capability of some groups of heterotrophic bacteria to accumulate phosphorus in microbial cells 'in excess' of their metabolic growth needs. Phosphorus is subsequently removed from the reactor as a result of routine sludge wasting. At medium-sized and large wastewater treatment plants with activated sludge processes for organic compound oxidation (BOD/COD removal), biological nitrification, denitrification (N removal), and biological excess P removal are integrated into one activated sludge reactor divided into anaerobic, anoxic, and aerobic zones.

2.2.3. CHEMICAL TREATMENT

Various chemical processes can be used for municipal wastewater treatment. At mechanical/biological wastewater treatment plants with highly effective **biological nutrient removal (bio-CNP plants**), chemical methods are often designed to be used in conjunction with biological methods to remove phosphorus by chemical precipitation. Chemicals are usually added to the effluent from a biological reactor and the chemical sludge is collected in a secondary clarifier together with the biological sludge. Chemical precipitation is a complex process with various interacting chemical and physical processes. As a result, not only is phosphorus removed, but so is particulate and colloidal organic matter, which improves the quality of the final effluent.

Chemical treatment can also be used for enhanced removal of suspended solids in primary sedimentation (called **primary precipitation**). The use of primary precipitation may increase the removal efficiency of TSS to 80%–90%, of BOD_5 to 50%–80%, and of bacteria to 80%–90% [11]. Wastewater treatment plants with **biological nutrient removal (BNR)** technology should not use primary precipitation, as it may reduce the availability of carbon substrate for biological phosphorus removal and denitrification. At some plants chemical treatment can also be used to perform complete secondary treatment of wastewater in combination with different physical methods, including the removal of nitrogen and phosphorus. Chemical processes have also been developed for the removal of heavy metals and specific organic compounds.

2.2.4. SLUDGE PROCESSING AND DISPOSAL

During wastewater treatment, pollutants are separated from treated wastewater and concentrated in the form of sewage sludge. Pollutants separated in primary settlers form the primary sludge and those separated in the final clarifiers form the secondary sludge, which may sometimes also include chemical sludge from chemical precipitation. Sludge extracted from wastewater treatment processes must be properly treated and disposed of in order to prevent pollution of surface and groundwater.

Sewage sludge treatment has two major objectives: 1) the **reduction of sludge volume** and 2) the **reduction of organic matter content** (**stabilisation**). The sludge volume is reduced in two stages: **thickening** and **dewatering**. In gravitational or mechanical sludge thickening, the water content in the sludge is reduced from approx. 94% to 92% for primary sludge and from approx. 99% to 95% for secondary sludge. The thickened, mixed primary and secondary sludge—usually previously stabilised—is mechanically dewatered and the water content is further reduced to approx. 75%. All processes aimed at sludge volume reduction require prior conditioning with polymers in order to make the dewatering faster and more effective.

Sludge stabilisation reduces the content of organic matter and thus decreases pathogen content, offensive odours, and potential for sludge putrefaction. Sludge can be stabilised **aerobically** (smaller plants), **anaerobically** (large plants), **chemically** (with lime), or by **composting**. Each method has its advantages and limitations. One of the major advantages of anaerobic stabilisation is the potential for energy recovery from the sludge in the form of biogas with high content of methane. In all cases, sludge stabilisation must be designed according to the sludge quantities to be treated, integration of the stabilisation process into other treatment processes, and the specific objectives of the stabilisation process.

3. MECHANICAL TREATMENT OF WASTEWATER

Mechanical treatment of municipal wastewater usually comprises the following stages: screening, grit removal, and primary sedimentation as presented in Fig. 6. During each stage of treatment, by-products are produced in the form of screenings (all objects stopped by screens), grit and sand that is captured in grit tanks, and easily-settleable suspended solids retained in primary settlers. All these by-products must be adequately processed and disposed of.

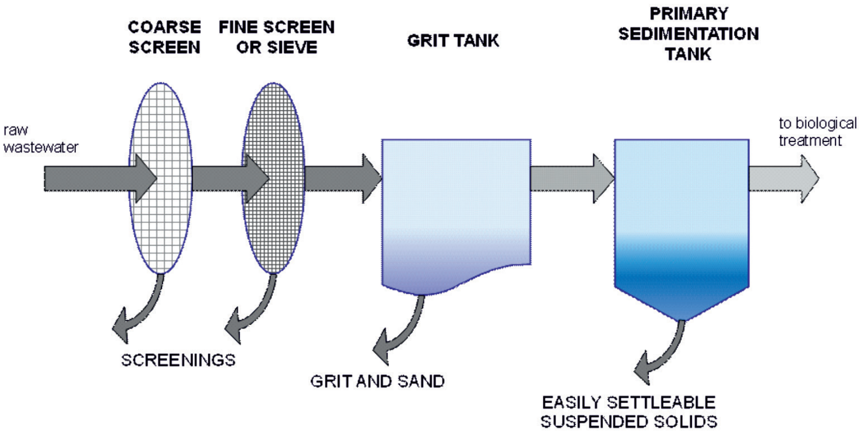

Fig. 6. Sequence of the processes of mechanical treatment of wastewater

3.1. SCREENING

Screening is usually the first process in wastewater treatment and its purpose is to remove large floating objects, such us rags (approx. 60%), paper (approx. 25%), and plastics (approx. 5%) that could disrupt further treatment processes. At the basis of this process is **drawling**.

3.1.1. TYPES OF SCREENS

Usually, two types of screens are used to treat municipal wastewater: coarse and fine screens. Coarse screens are usually installed at very beginning of the treatment train and are used to protect pumps, valves, pipelines, and other appurtenances from damage or clogging. Fine screens with very small openings (<6 mm), very fine screens (1.5–6 mm), and sieves are used in the preliminary treatment of wastewater. Apart from their opening size, the screens are classified according to their construction and the method used to clean them. The most common types include bar screens, step screens, belt screens, and rotary drum screens. The classification of screens is presented in Fig. 8. Although both coarse and fine screens can be cleaned manually or mechanically, in practice only the smallest plants manually clean static screens.

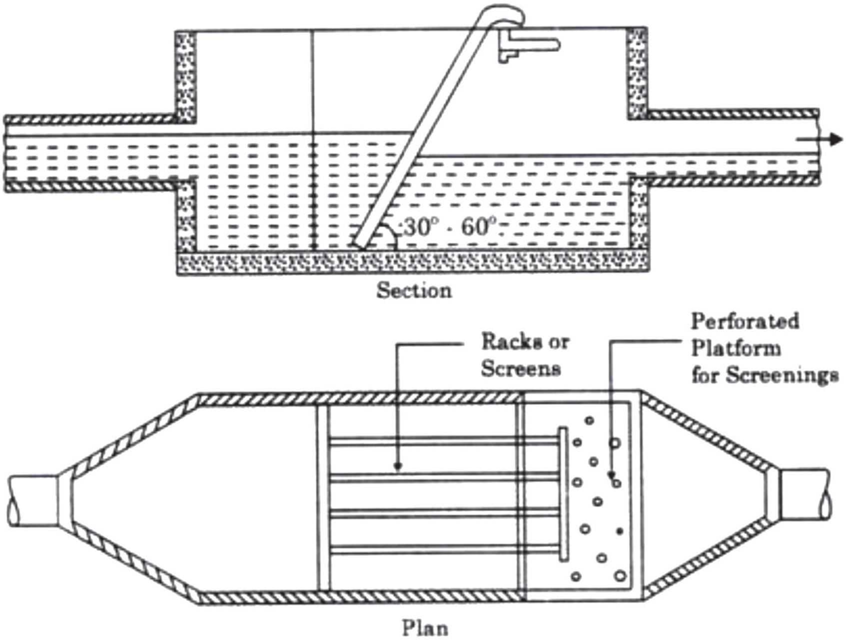

Fig. 7. Diagram of a manual bar screen installed in **a** channel

Basket screens are usually used for the preliminary treatment of wastewater in pump stations to protect the pumps from clogging and in small wastewater treatment plants as coarse screens. The wastewater flows through the screen in the shape of a basket, leaving the coarse, floating material inside it. The basket is raised at intervals by hand or mechanically, emptied, and then reinstalled.

Rake screens consist of a set of rods, in the form of a grate, on which the rake moves. The screenings are transported upwards by a hydraulically or electrically driven rake. Such screens are often used as coarse screens, as they generally have a wider ground clearance.

Step screens in the range of 3–8 mm are typically used. The name comes from their appearance, which is reminiscent of escalators. Theoretically, the principle behind their operation is similar: moving steps transport pollutants to the top of the grating. The grille is driven by eccentrics, which transfer movement to the lower shaft and to the steps through steel plates or ropes (depending on the manufacturer).

Continuous belt screens use a rotating endless sieve belt which is formed of identically shaped elements (hooks), located on a series of horizontally aligned members. The small suspended solid particles are lifted out of the water flow and conveyed to discharge at the top of the machine. In the discharge area, the inversion of the filter tape, thanks to the self-cleaning profile of the hooks, causes the screened material to fall into a special collection box.

Rotary drum screens can be internally or externally fed. In an **internally fed rotary drum screen**, the wastewater flows into the open end of the inclined screen basket and then through the screen. Floating and suspended materials are retained by the bars of the screen basket. Blinding of the screen surface generates an additional filtering effect so that solids smaller than the bar spacing can be retained. **Externally-fed rotary drum screens** feature the cylindrical drum sieve, which allows the water to pass through, while coarse particles are held back by the outer drum surface and are carried, in a rotating movement, to a scraper mechanism, where they are removed. The screen has a spray pipe installed into the drum in order to prevent the build-up of adhesive materials on the outer drum's surface. Spray cleaning takes place over the full lengths of the drum, ensuring continuous operation.

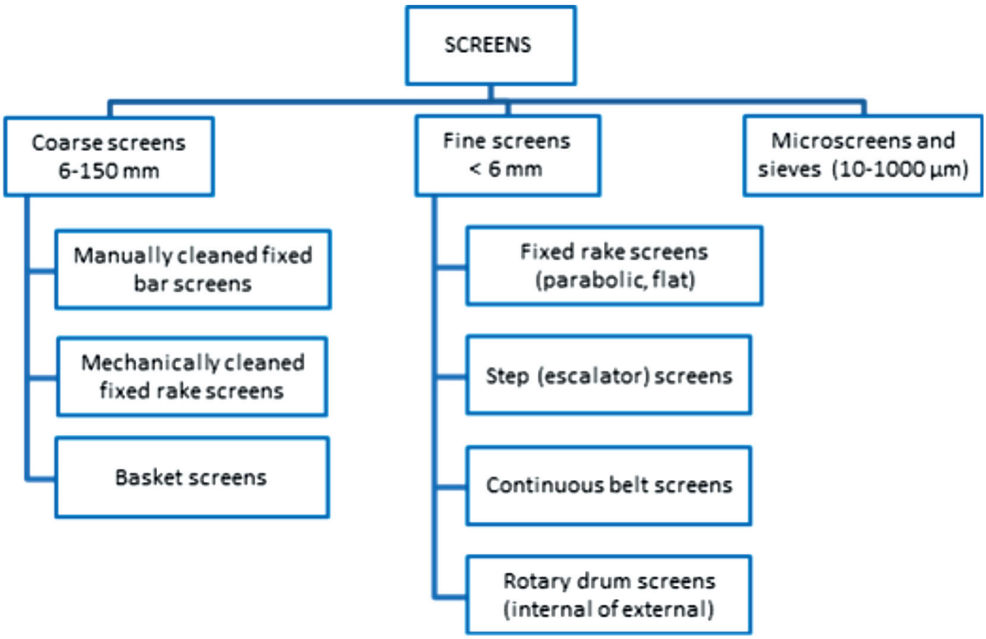

Fig. 8. Classification of the most common screens used in the preliminary treatment of wastewater

Sieves and **microscreens** have smaller openings than fine screens and therefore can more effectively separate suspended materials. There are several types of sieves: **rotary** or **static**, and those installed directly **inside the channel** or **outside the channel** in a separate container. They can be drums (static or rotary), disc filters, or rotating belts. The filter media come in pore sizes ranging from 10 μm to 1 mm and are made of woven polyester or stainless steel. One of the most common, the **rotary drum sieve**, consists of a slowly rotating drum that is equipped with small gaps. The drum is driven by the transmission using an electric motor. The treated wastewater is introduced into the drum and pushed out through the perforated casing. Sieved particles remain in the drum and—through the rotary movement of the drum and the internal screw—are moved to the end of the screen and then removed. Sieves are often designed all in one unit with the possibility of dewatering, compacting, and transporting the screenings into a container.

3.1.2. SCREENING VOLUME AND CHARACTERISTICS

The screenings collected on coarse and fine screens are different. Coarse screenings consist of debris such as rocks, branches, leaves, paper, plastic bottles, and rags. The quantity and characteristics of the collected screenings vary depending on the size of the bar openings, the type of sewerage system, the geographic location, and climatic conditions. Typical data on coarse screenings are presented in Table 8. Coarse screens installed in combined sewerage systems may produce volumes of screenings much larger than the amounts found in separate systems. Fine screenings consist of material retained on screens with openings smaller than 6 mm and include small rags, small plastics, grit, undecomposed food waste, faeces, sanitary products, etc. Compared to coarse screening, the specific weight is slightly lower and the moisture content is slightly higher, as shown in Table 8 [11].

Table 8

Typical characteristics of screenings removed from coarse and fine screens [11]

Screen type	Opening size (mm)	Specific weight (kg/m^3)	Moisture content (%)	Volume of screenings (dm^3/1000 m^3 of wastewater)	
				Range	Typical
Coarse bar screen	50	600–1000	50–80	4–11	6
Coarse bar screen	25	600–1000	50–80	15–37	22
Coarse bar screen	12.5	600–1000	50–80	37–74	50
Fine bar screen	12.5	900–1100	80–90	44–110	75
Rotary drum screen	6.25	900–1100	80–90	30–60	45

The total volume of screenings retained by the screens can also be expressed as a unit of 'production' of screenings per person per year and the number of inhabitants connected to the sewerage system. The value of this indicator is a function of the screen opening size as presented in Table 9.

Table 9

Unit volume of screenings retained by different screen opening sizes [2, 19]

Screen opening size (mm)	Volume of generated screenings (dm³/[person·year])
10	10
15–20	8
25–35	3
40–50	2.3
60–80	1.6
90–125	1.2

3.1.3. DESIGN CONSIDERATIONS

The design of screens requires a few things:

1) **Sizing of the channels and the screen width** – For most screen installations, at least two units are installed in case one of the units is out of service for maintenance. Each unit should be able to handle peak flows. If only one screen unit is installed, a bypass channel is obligatory. The screen channel is sized to prevent sedimentation and accumulation of grit and sand with a minimum design velocity of 0.4 m/s and a maximum of 0.9 m/s to prevent debris passing through the bar screen at peak flow.
2) **Calculations of headloss through screens** – The raking mechanism on the screens is usually operated based on the differential headloss (h_L) through the screens or by a time clock. Headloss in coarse screens typically should not exceed 150 mm. In coarse screens, the the headloss can be calculated with following formula, considering the approach velocity (v) and the velocity through the bars (V) [11]:

$$h_L = \frac{1}{C}\left(\frac{V^2 - v^2}{2g}\right)$$

where

h_L – headloss (m),
C – an empirical coefficient to account for turbulence and eddy losses (typically 0.7 for a clean screen and 0.6 for a clogged screen),
V – the velocity of flow through the openings of the bar screen (m/s),
v – the approach velocity in upstream channel (m/s), and
g – acceleration due to gravity (9.81 m/s²).

Clean water headloss through fine screens can be calculated with the formula below [11]. The values of C and A in this formula depend on the screen design parameters and must be determined experimentally:

$$h_L = \frac{1}{2g}\left(\frac{Q}{CA}\right)^2$$

where

h_L – headloss (m),
C – a coefficient of discharge for the screen (a typical value for a clean screen is 0.60),
Q – the flow through the screen (m^3/s),
A – the effective open area of the submerged screen (m^2), and
g – acceleration due to gravity ($9.81\ m/s^2$).

3) **Calculation of the volume of screenings captured on the screens** – The volume of screenings captured on the screen (V_{scr}) can be roughly estimated using the unit volume of screenings (a) presented in **Table 9**. For this purpose, the following formula can be used [19]:

$$V_{scr} = \frac{a \cdot n}{365 \cdot 1000}$$

where

V_{scr} – the volume of screenings captured on the screen (m^3),
a – the unit volume of screenings produced by one person per year (dm^3/[person·year]) (Table 9), and
n – the number of people connected to the sewerage system.

3.1.4. SCREENING PROCESSING AND DISPOSAL

According to Polish regulations, screenings captured by the coarse and fine screens at municipal wastewater treatment plants are considered waste (code 19 08 01) and must be properly processed and disposed of [15]. Screenings are burdensome to the environment because they quickly rot and emit unpleasant odours. After separation on the screens, screenings go to a screw conveyor, in which they are shredded, compressed, and dewatered, and then transferred to a storage container. This entire process is usually encapsulated to avoid the spread of unpleasant odours, and the polluted air is cleaned with filters.

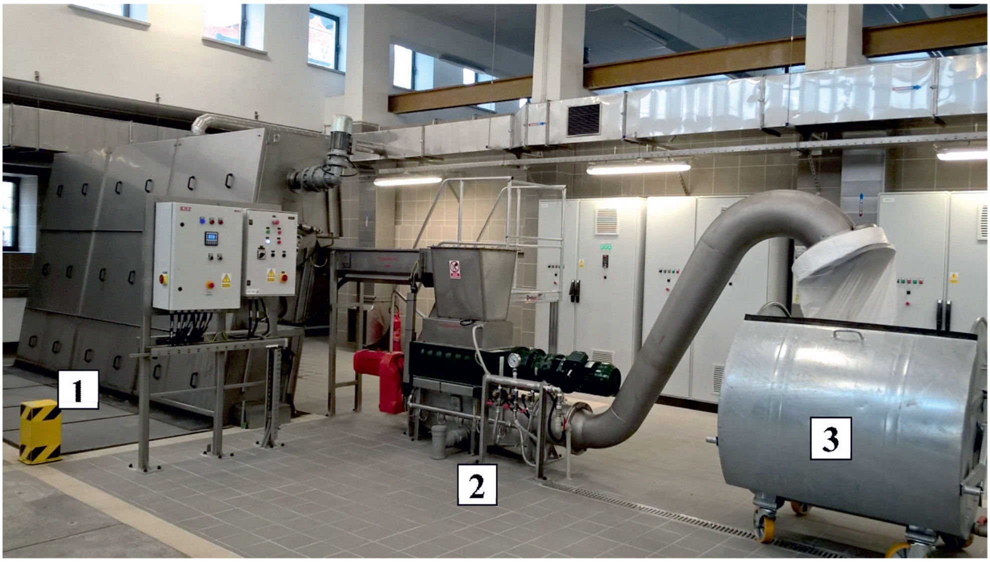

Fig. 9. A screening separation, processing, and disposal system by ENKO®: 1 – screens, 2 – screening press, 3 – container for screenings (ENKO S.A.)

3.2. GRIT REMOVAL

The removal of grit and sand carried with the wastewater is necessary to protect the pumps from mechanical damage and to prevent unnecessary abrasion and wear of mechanical equipment and the deposition of mineral particles in the sewage treatment plant's tanks, pipelines, and facilities. This process is carried out in devices called **grit tanks** using **sedimentation**, i.e. the falling of particles of a higher density (grit and sand) in an environment of lower density (wastewater). The grit tanks are designed to remove mineral particles with a grain size larger than 0.15 mm.

3.2.1. THEORY OF SEDIMENTATION

The description of the sedimentation process is different for **discrete** and **flocculent** particles. Discrete particles do not change in size, shape, or mass during settling, while flocculent particles tend to agglomerate during settling, and thus have dynamic characteristics. In grit tanks, particles settle separately following the theory of **discrete particle sedimentation**. According to this theory, a particle in a fluid of lower density accelerates until a limiting terminal velocity is reached under the equilibrium of **gravitational force** (left) and **friction drag forces** (right) [20]:

$$(\rho_s - \rho_w) g V = C_D A_c \rho_w \frac{v_s^2}{2}$$

where

ρ_s – the density of the particle,
ρ_w – the density of the fluid,
C_D – Newton's drag coefficient, which varies with Reynolds number (Re); in sedimentation $Re = v_s d/\mu$, where d is the particle diameter and μ is the kinematic viscosity of the fluid,
V – the volume of the particle,
A_c – the cross-sectional area of the particle, and
v_s – the settling velocity of the particle.

From the above equilibrium equation, the following equation for v_s can be obtained:

$$v_s = \sqrt{\frac{2gV(\rho_s - \rho_w)}{C_D \rho_w A_c}}$$

assuming that the density of the fluid $\rho_w = 1$ and, for spherical particles, that $V = \pi d^3/6$ and $A_c = \pi d^2/4$:

$$v_s = \sqrt{\frac{4gd(S_s - 1)}{3C_D}}$$

where

S_s – is the specific gravity of the particle[2].

For turbulent flow $Re > 10^3$ and then C_D tends to 0.4; thus

$$v_s = \sqrt{3.3gd(S_s - 1)}$$

For laminar flow $Re \leq 1$ and $C_D = 24/Re = 24\mu/v_s d$; thus

$$v_s = \frac{\breve{}\ {}^{2}(\ {}_{s} - 1)}{18\mu}$$

which is a form of Stokes' law.

Grit tanks are designed based on discrete particle sedimentation theory with the following assumptions, which bring the description close to the ideal sedimentation tank:

- there are quiescent conditions in the settling zone
- there is uniform flow across the settling zone
- solid concentration is uniform as flow enters the settling zone
- solids entering the sludge zone are not re-suspended [20].

[2] Specific gravity is the dimensionless ratio of the density of a substance (e.g. particle) to the density of a reference substance (e.g. water)

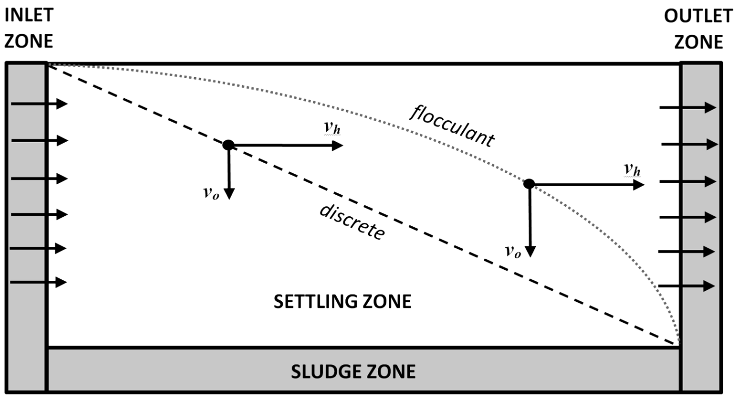

Fig. 10. Ideal rectangular sedimentation tank (settling of discrete and flocculent particles)

In a horizontal-flow, rectangular grit tank, a discrete particle settles with constant velocity (v_o), through a depth (h_o) in the retention time of the tank (t_o) (Fig. 10):

$$v_o = \frac{h_o}{t_o}$$

$$t_o = \frac{V}{Q} \frac{(\text{Volume of the tank})}{(\text{Wastewater flow rate})}$$

Thus

$$v_o = \frac{h_o \cdot Q}{V} = \frac{h_o \cdot Q}{A \cdot h_o} = \frac{Q}{A}$$

where *Q/A* is the hydraulic loading of the tank's surface. This equation shows that the removal of discrete particles is not dependent of the depth of the tank.

3.2.2. QUANTITY AND CHARACTERISTICS OF GRIT

The quantity of grit carried with wastewater depends primarily of the type of sewerage system. In combined systems, the amount of sand is noticeably larger than in sanitary systems. The quantity of sand is usually expressed in relation to a single inhabitant and falls with the range of 2–5 dm^3/(person·year). Another way to express the quantity of sand and grit is its volume per 1000 m^3 of wastewater. In separate sewerage systems and in combined systems during dry weather, this value falls in the range of 15–75, and on average 35 dm^3/1000 m^3 of wastewater. However, in combined sewerage systems during rain events, this value can easily reach as high as 150–200 dm^3/1000 m^3 of wastewater [2].

The composition of sand and grit trapped in grit tanks is not homogeneous. It includes sand, gravel, cinder, or other heavy solid materials that are 'heavier' than the organic biodegradable solids in the wastewater. Grit also includes eggshells, bone chips, seeds, coffee grounds, and large organic particles, such as food waste [22]. Organic matter in the trapped grit should not exceed 10%. The moisture content of grit ranges from 13% to 65%, with a volatile organic content of 1% to 56%, depending on specific conditions. Sand particles stopped in grit tanks have a minimum diameter of 0.15 mm. The specific density of clean sand particles is about 2.65 kg/dm^3 and for material with substantial organic content it is approx. 1.3 kg/dm^3. After being intercepted in a grit tank, the grit is pumped out and directed to sand separators in order to rinse out organic particles.

3.2.3. TYPES OF GRIT TANKS

Among the many types of grit removal systems, the following are widely used in practice: horizontal-flow grit tanks (rectangular and Dorra-type), aerated grit tanks, and vortex-type grit removal systems.

Rectangular horizontal-flow grit tanks – A rectangular horizontal grit tank is the simplest design for a grit tank, and such tanks have been used for very long time. They are designed as longitudinal tanks with a rectangular, trapezoidal, or parabolic cross-section. The cross-section is designed so that only sand—without any organic suspensions—is retained. Therefore, the basic design parameter is a constant flow velocity in the range of 0.25–0.35 m/s. This way the cross-sectional area of the grit tank (A) can be calculated as Q/v. The length of a grit tank (L) is calculated assuming a hydraulic retention time (τ) of approx. 1 minute:

$$L = v \cdot \tau = 0.30\frac{m}{s} \cdot 60s = 18m$$

The recommended relationship between the length and width of a horizontal-flow grit tank is as follows:

$$L = (15 \div 37)\frac{B}{1.25}\ [\mathrm{m}]$$

Grit is usually removed from rectangular horizontal-flow grit tanks by a conveyor with scrapers. Centrifugal pumps of an appropriate design are used to remove the grit. The pumps are stationed on a wheeled carrier that is placed lengthwise above the grit chamber.

Dorra-type horizontal-flow grit tanks – This style of grit tank has a square shape and a flat bottom, and its depth does not exceed 1–1.5 m. The wastewater flows in through special openings with adjustable baffles and flows out through the overflow located on the opposite side of the grit tank. The grit tank is equipped with a special rotary scraper which scoops the sand to a sump at the side of the tank. The sand then flows along with the rinse water into a steep flushing tank where a washing installation is installed. The washing can take place in a co-current or counter flow.

The Dorra-type grit tanks are sized based on the hydraulic loading of the tank's surface (O_h) which should not exceed $O_h \leq 50$ m³/(m²·h) [2]. Lately, these grit tanks are being replaced with aerated or vortex-type grit tanks.

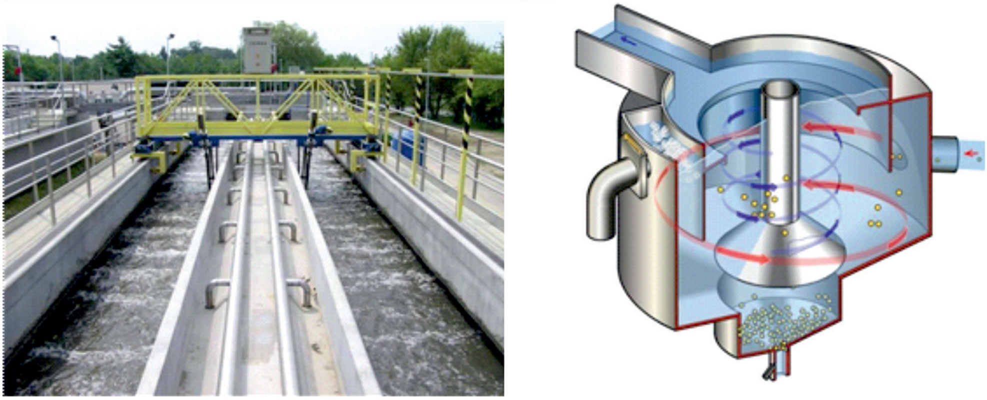

Fig. 11. Aerated horizontal-flow rectangular grit tank with travelling bridge at Przemyśl WWTP in Poland (left) and vortex grit tank (right) [10]

Aerated rectangular grit tanks – A specific feature of the aerated rectangular grit tank is a spiral flow path perpendicular to the flow of wastewater through the tank. This is due to air being introduced along one side of a rectangular tank. In such a tank, it is not the horizontal flow velocity but rather the velocity tangent to the spiral flow path of the particle that affects the efficiency of grit sedimentation. This velocity can be easily controlled with the amount of air introduced into the tank and the grit removal efficiency is very high—approaching 100%. There are two major types of aerated grit tanks: with and without grease removal. This function is performed by a side longitudinal chamber separated from the main tank by loosely hanging baffles.

Aerated rectangular grit tanks are sized for a detention time of 10–15 minutes at maximum dry weather flow (Q_{max}) and 3–4 minutes at rain flow. The cross-sectional area of the tank should be designed so that horizontal flow velocity does not exceed 0.20–0.25 m/s, and the peripheral velocity is no more than 0.3–0.4 m/s. Compressed air is supplied to the tank as medium or coarse bubbles through the 'reversed T' air outlet situated approx. 0.6–10.8 m above the tank's bottom. The amount of supplied air is 0.2–0.5 $m^3/(m \cdot min)$ (per each metre of the tank's length) [11].

Vortex-type grit tanks – These tanks are designed in the shape of a funnel, where wastewater enters the tank tangentially at the top of the unit. The energy of the wastewater flow generates a vortex. The outflow is located at the top of the unit, usually a bit higher than the inflow channel because the wastewater table rises as the distance from the centre of the device increases. Grit settles by gravity into a hopper, from which it is removed by a grit pump or a lift pump. Vortex-type grit tanks are considered effective and reliable and are used at many small and medium-sized plants.

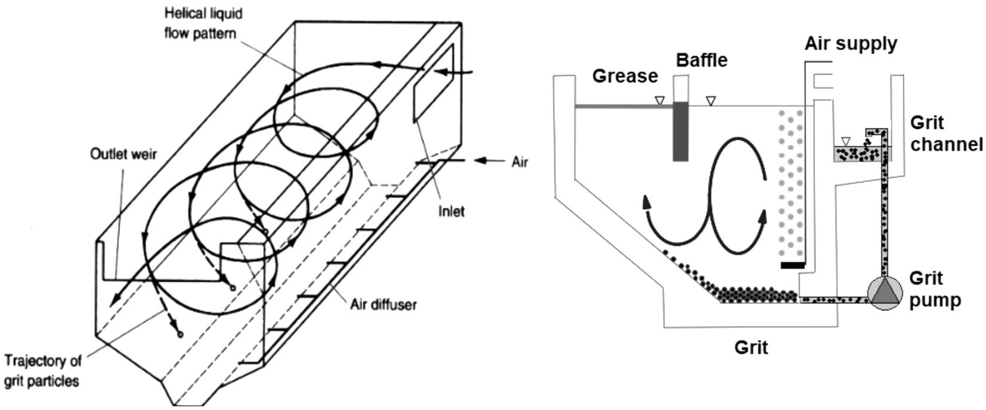

Fig. 12. Flow pattern in an aerated rectangular horizontal-flow grit tank [11] and cross-section of an aerated rectangular grit-tank with **a** degreaser

When designing vortex-type tanks, it should be assumed that the inflow velocity is in the range of 0.75–1.0 m/s and the outflow velocity is no more than 0.8 m/s. Theoretical detention time should be kept at approx. 25–45 s. The slope of the walls in the conical part of the tank should be set at 1:1 and the ratio of the tank's surface area to its depth should be 2:1. The surface area of the vortex-type grit tank F can be calculated with the following formula:

$$F = \alpha \frac{Q_{\max} \cdot 1000}{n \cdot v_o}$$

where
Q_{max} – the maximum wastewater flow (m^3/s),
α – a correctional factor due to deficient use of tank's surface (α = 2),
n – the number of grit tanks, and
v_o – the settling velocity for the smallest particles retained (mm/s).

3.2.4. GRIT WASHING AND CLASSIFICATION

Grit trapped in grit tanks must be removed, washed, and disposed of. In horizontal-flow grit tanks, the grit is pumped out directly from the longitudinal channel or from a hopper at the end of the grit tank where the grit is scraped. In both cases travelling-bridge installations are used, moving from one end of the tank to the other. Airlifts, screw conveyors, or jet pumps are used for grit pumping. Among the types of scrapers used to shift the grit settled to the bottom in rectangular grit tanks, the most popular in recent years have been the chain scrapers consisting of plastic chains and scraper flights made of plastic, galvanised steel, or aluminium. In vortex-type grit tanks, the grit settles in the hopper, from where it is pumped out in the form of a sand-pulp.

The sand-pulp removed from the grit tanks must be washed in order to remove any organic material and then classified (separated). Two principal types of grit classifiers are used. One type uses an inclined, submersed rake which vibrates to provide the mixing necessary to separate the organic material from the gravel, and lifts the washed grit to the discharge point above the level of the wastewater. The other type uses an inclined screw and moves the grit up the ramp [11]. Other types of grit separators can also be used e.g. hydro-cyclones. Sometimes the washing and classification of grit are integrated in the same unit. The washed and separated grit is disposed of in a landfill or—if it is of good quality—it is sometimes recycled and utilised at the wastewater treatment plant. In such situations, the grit should be lime-stabilised before recycling.

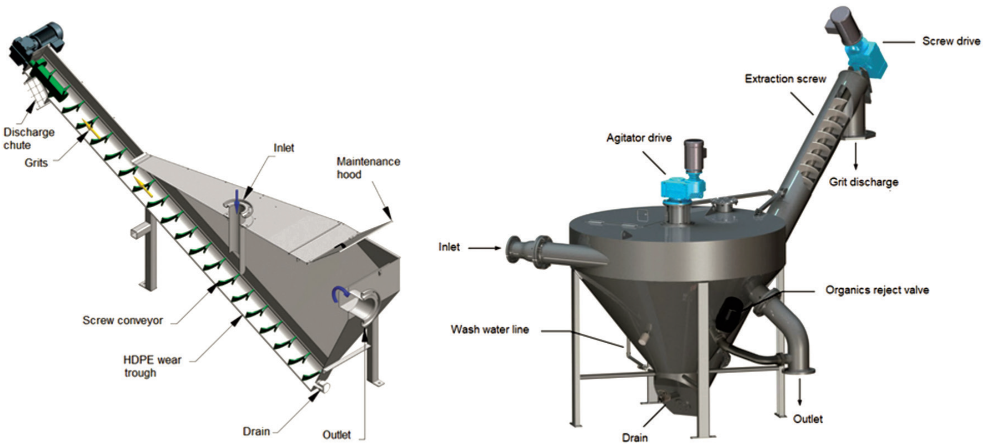

Fig. 13. A Kusters Water grit separator (left) and a grit washer integrated with a separator (right)

3.3. PRIMARY SEDIMENTATION

The process of primary sedimentation consists in separating solid particles from flowing wastewater by settlement in near quiescent conditions. This process takes place in **settling tanks**, which play a dual role: the removal of settleable solids, and the concentration (**thickening**)of the solids removed (**sludge**). The removal of easily-settleable solids in primary sedimentation is intended to reduce the pollution load in the biological reactor, and thus reducing the size of the reactors and the cost of the biological treatment.

3.3.1. FACTORS AFFECTING PRIMARY SEDIMENTATION

Wastewater directed to primary sedimentation has already had the majority of floating material and large mineral particles removed by screens and grit tanks. However, it still contains a lot of suspended organic material ranging from 0.05 to 10 mm in size. This material, if it occurs in concentrations not exceeding 500 g/m^3, exhibits

flocculation properties, which results in **increased vertical velocity** of settling with depth during sedimentation (see Fig. 10). Thus, the depth of the settling tank is of fundamental importance for the efficiency of the sedimentation process and should be large enough for the particles to agglomerate.

The theory of discrete particle sedimentation cannot be used to describe flocculent particle sedimentation, and the mathematical description of the process is very complex. This is why laboratory tests are usually performed with a **settling column**[3] to determine the flocculent particle settling velocity that is necessary to determine the settling tank's depth and length. In such tests, concentrations of suspended solids are measured at each sampling point. The results are presented graphically, aiding the determination of the depth and optimal hydraulic loading of the primary settling tank necessary for the effective removal of suspended solids.

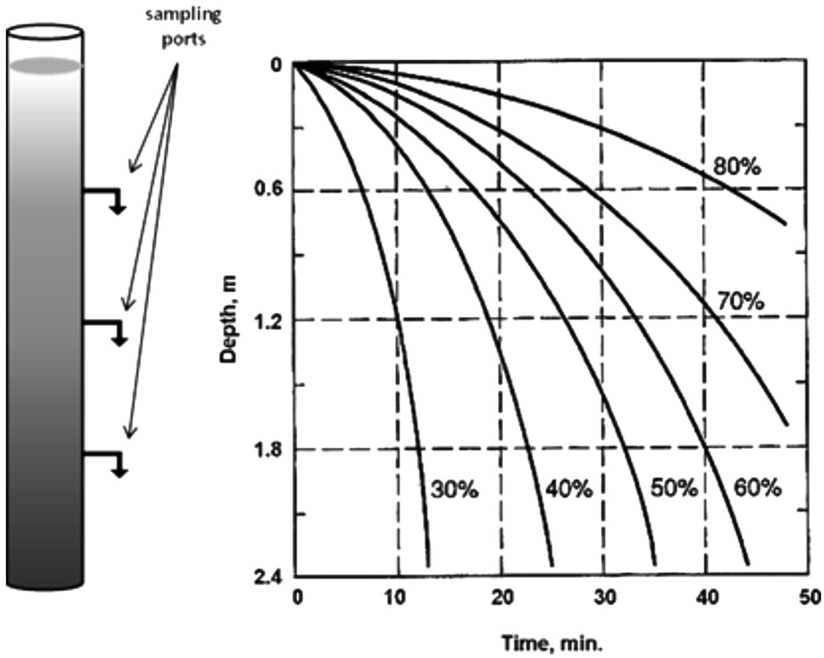

Fig. 14. Flocculent particles settling test in a column with isopercentage removal curves

Two important design parameters must be considered when designing primary settling tanks. One is the hydraulic retention time (detention time) and the other is the surface-loading rate. **Detention time** is mostly a function of the flocculating capabilities of the suspended solids. As actual detention time varies with the wastewater flow rate (storm flows) and the hydraulic flow pattern in the settling time, the theoretic detention time (τ_t) calculated as a ratio of a tank's volume (V) and the

[3] A settling column is a vertical tube 2–3 m high and approx. 12 cm in diameter with sampling ports at intervals of approx. 0.6 m.

wastewater flow rate (Q) ($\tau_t = V/Q$), or according to the literature must always be increased by a ratio of 1.5–2 [2]. The **surface-loading rate** (*SLR*) is a recommended parameter for designing primary settling tanks and can be calculated from the settling velocity (v_s) obtained during laboratory settling tests, by dividing it by a reduction coefficient of 1.25–1.75. The effect of detention time and surface-loading rate on the efficiency of primary sedimentation is shown in **Błąd!**.

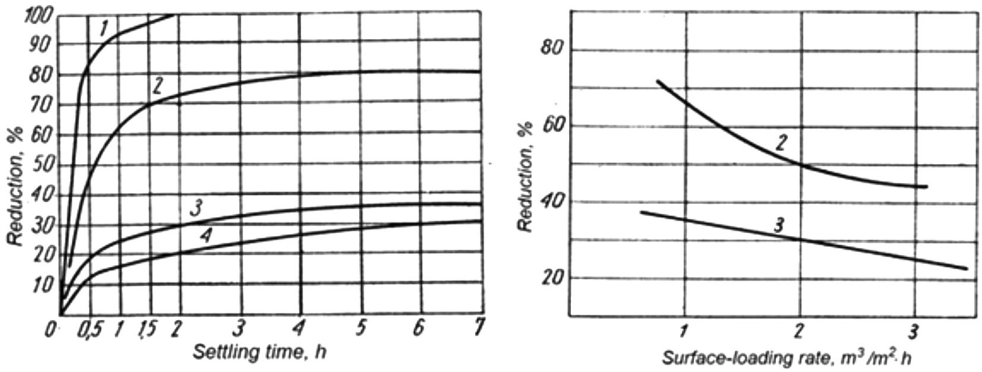

Fig. 15. Efficiency of sedimentation as a function of settling time and surface-loading rate [2]: 1 – easily-settleable solids; 2 – total suspended solids; 3 – BOD; 4 – COD ($KMnO_4$)

3.3.2. TYPES OF SEDIMENTATION TANKS

According to their technical design, most of the sedimentation tanks fall into one of the following categories: horizontal-flow **rectangular tanks**, horizontal-flow **radial tanks,** or **vertical-flow tanks** (Fig. 16).

3.3.2.1. Horizontal-flow rectangular settlers

This type of settler is a rectangular tank in the range of 30–60 m in length (L), 4–12 m in breadth (B), and with an active height (depth; H) of 1.5–3.0 m. The total height of the settler is about 2.5–4.0 m. The inlet to the tank is on the shorter side of the tank, and the flow moves along its longer side at a mean velocity of approx. 5 mm/s to the other short side of the tank, where a weir is located. The bottom of the tank is flat or slightly inclined in the opposite direction of the wastewater flow (1%–2%) in order to facilitate sludge scraping. The specific design recommendation regarding the ratios of the rectangular tank's dimensions vary from country to country and by the tank's application (e.g. primary or secondary sedimentation).

However, the following values can be assumed to be appropriate for **primary sedimentation** in most cases [2]:

$$\frac{L}{H} = \frac{9}{1} \div \frac{22}{1}; \ \frac{L}{B} = \frac{4}{1} \div \frac{7.33}{1}$$

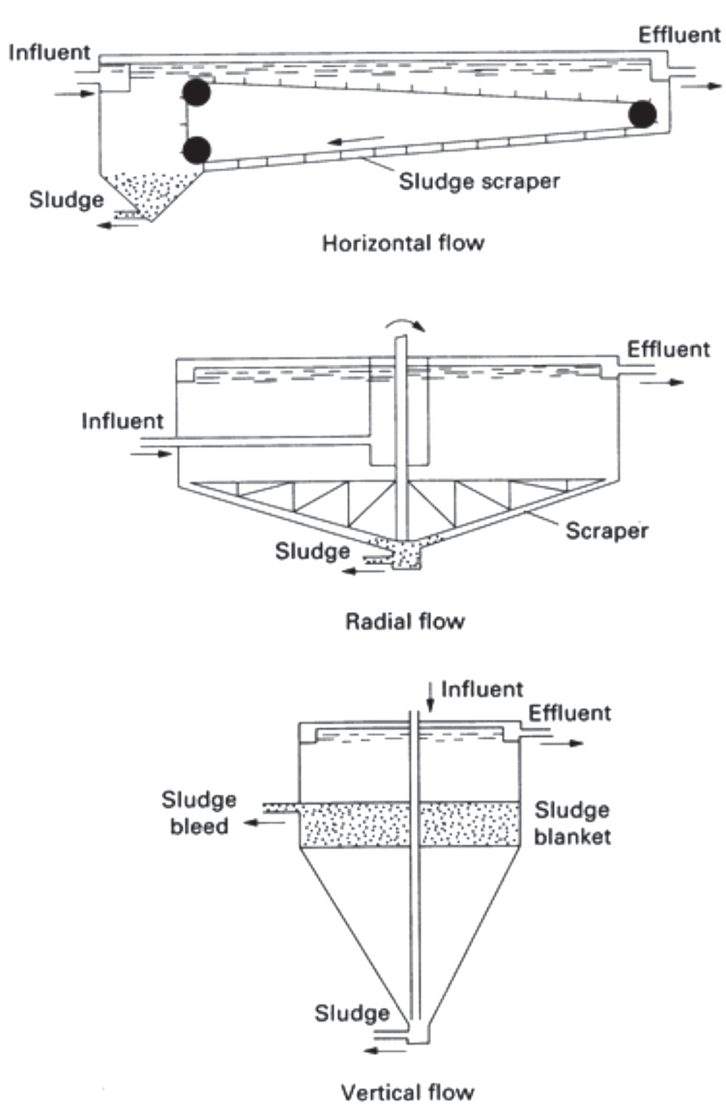

Fig. 16. Major types of sedimentation tanks [20]

From the hydraulic efficiency standpoint of the above values should be: $L/H \geq 15$ and $L/B \geq 5$.

The settler's **volume** is designed on the basis of hydraulic detention time (τ_d), which should be not shorter than the required sedimentation time (τ_s). In practice, the detention time is equal to the sedimentation time multiplied by a safety factor of 1.5–2. In such a case, the tank's volume (V) equals $Q \cdot \tau_d$. The settler's surface area is calculated from the assumed surface hydraulic load (q_A), which depends on the type of settler and its application (primary or secondary sedimentation; see Table 10).

Table 10

Approximate design values of surface hydraulic loading and detention time for different types of settlers [19]

Type of settler	Hydraulic surface loading *qA* (m3/m2·h)			Detention time τ*d*, h
	Horizontal flow		Vertical flow	
	rectangular	circular		
Primary before biofilters	≤1.3	0.8–1.3	≤1.3	1.5–2.3
Primary before activated sludge	≤4.0	2.5–4.0	≤3.0	0.5–1.0
Secondary after biofilters	≤1.5	1.0–1.5	≤1.5	1.5–2.3
Secondary after activated sludge	≤1.2	0.7–1.2	≤1.2	1.7–2.7
After coagulation	≤1.5	1.0–1.5	≤1.5	1.5–2.3

Inlet arrangements to the rectangular settling tank are designed to deflect and attempt to diffuse the incoming flow so that jets of high-velocity flow are avoided. Various technical concepts are used for this purpose, including channels and stilling walls with holes and baffles.

The **outlet** consists of a weir across the whole width of the tank. The length of the outlet weir should be designed in such way that hydraulic loading of the weir's length is ≤20 $m^3/(h{\cdot}m)$, and sometimes even several single– or double-sided weir channels must be installed in a settler. In front of the weir there is usually a scum trough for intercepting floating parts.

Fig. 17. Outlet from **a** rectangular settler with three double-sided weir channels and scum trough

Sludge deposited on the settler's floor must be periodically scraped into the sludge hopper near the settler's inlet. There are three common arrangements of **scrapers**. In one of them the floor is swept by scraper blades supported from a **travelling bridge** that spans the breadth of the settler. The bridge is carried on flanged wheels which run on rails on top of the side walls (Fig. 18A). Another type of scraper arrangement utilises a number of scraper blades attached to endless **chains** running on sprocket wheels (Fig. 18B).

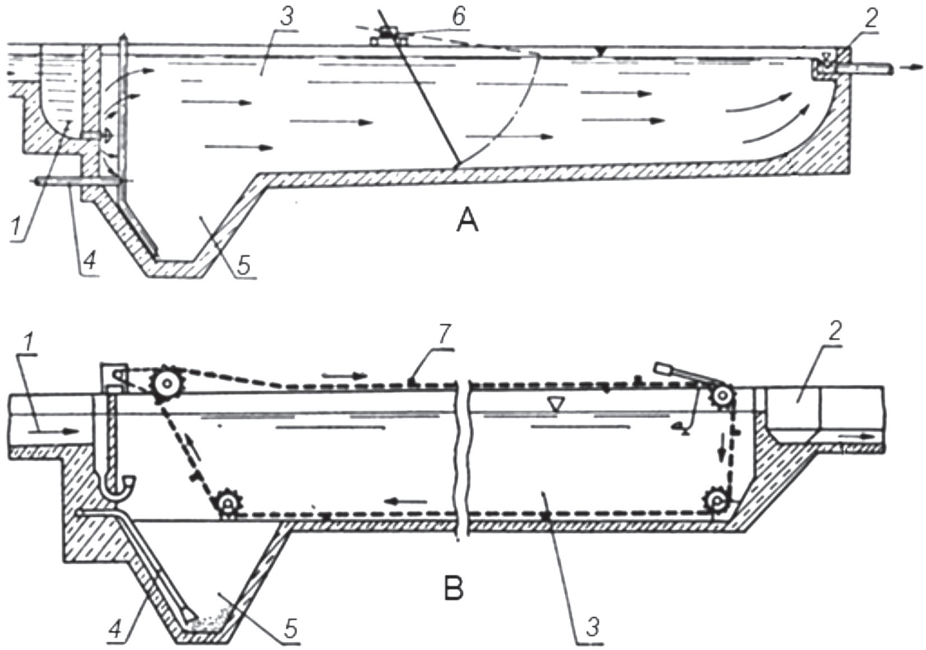

Fig. 18. Horizontal-flow rectangular setting tank with travelling bridge scraper (A) and chain scraper (B). 1 – inlet; 2 – outlet (weir); 3 – settling zone; 4 – sludge pipe; 5 – sludge hopper; 6 – travelling bridge with scraper; 7 – chain scraper

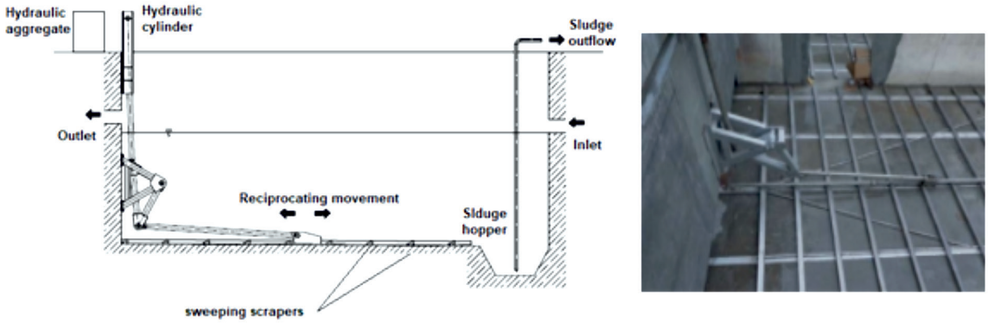

Fig. 19. Rectangular settler with reciprocating sludge scraper (modified from Dynamik Filtr)

The third type is a **reciprocating frame scraper** based on the forward and return motion of hydro-dynamically shaped beams. The frame with scrapers covers the whole floor of the tank. The concave side of each beam pushes the sediment towards the sludge hopper, whilst on the return movement the tapered part wedges under the sludge blanket. The movement slows down drastically when approaching the contamination release point and the turning motion is performed three times faster (Fig. 19).

3.3.2.2. Horizontal-flow radial settlers

Radial settling tanks are typically used for primary sedimentation at **medium-sized and large wastewater treatment plants**. They vary in size from 15 to over 40 m in diameter (D) and the heights are about the same as for rectangular tanks (H = 2.5–4.0 m) used for primary sedimentation.[4] The inlet to the tank is in the centre—the **inlet pipe** passes under the settler's floor and rises vertically in a covered **central distribution chamber** in the tank's centre. The central chamber has a diameter of approx. 10–20 cm. It has a number of holes with baffles in its vertical wall to force the **uniform horizontal radial flow** of wastewater with velocity decreasing outwards. The tank's floor is **horizontal** or **slightly inclined** towards the centre (2%–8%) to facilitate the scraping of sludge into a hopper located under the central distribution chamber.

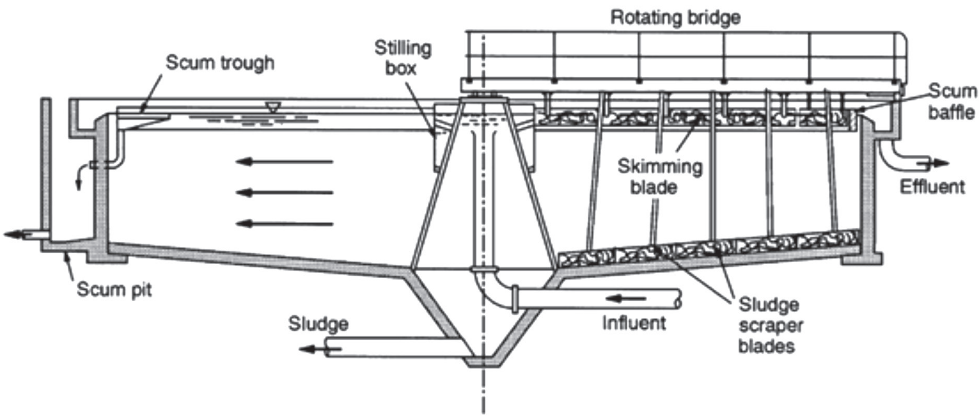

Fig. 20. Horizontal-flow radial sedimentation tank (adapted from [1])

The settler's **volume** is calculated on the basis of required detention time (τ_d) and hydraulic surface loading q_A (see **Table 10**). The active depth of a radial settler is derived from the simple dependence $H = V/A$, where A (the settler's surface area) is calculated from the settler's surface hydraulic load. Typically, the settler's active depth is in the range of 1.5–3.0 m. In the case of a settling tank with a sloping bottom, the active depth of the settler only applies to its cylindrical part, leaving the conical part

[4] Settlers used for secondary sedimentation are deeper than those used for primary sedimentation.

for collection of sludge. For settlers with a diameter in the range of 18–48 m used **for primary sedimentation**, it is recommended that the *D*/*H* ratio be

$$\frac{D}{H} = \frac{11}{1} \div \frac{16.8}{1}$$

The **outlet** in the circular settler is located at one point on its circumference in the form of a channel or pipe to which wastewater is supplied from the peripheral V-shaped weir. The **peripheral weir channel** is often slightly offset from the wall of the settling tank to allow the inflow of sewage from its two sides. Such a design allows lower linear hydraulic loading of the weir as compared to rectangular settlers. The length of the outlet weir should still be designed based on a hydraulic loading of the weir not exceeding 20 $m^3/(h{\cdot}m)$. In addition, before the weir channel there is a **scum baffle** to prevent access of floating parts to the channel.

Fig. 21. Empty horizontal-flow sedimentation tank in Gorzów Wielkopolski (Poland) with central distribution chamber (1), rotating bridge (2), sludge scraper (3), scum removal screw (4), peripheral weir channel (5), and scum baffle (6)

The floors of circular settlers are most commonly swept by blades supported from a **rotating radial bridge**. The bridge's wheel running on the outer wall often has rubber tyres or is flanged when running on a rail. Single spiral scraping blades have been used, but multiple blades in echelon at an angle to the direction of motion are more common [26]. Speeds of the radial bridge at the settler's circumference are typically about 0.05–0.1 m/s, which translates into 2–3 rotations per hour. The sludge scraped into the central hopper is removed by gravity and redirected for thickening. Sometimes suction scrapers are used instead of mechanical scrapers. The suction scrapers suck the sludge directly from the tank's floor, so there is no need for a sludge hopper. However, the suction scraper pipes becoming clogged is a very common problem.

3.3.2.3. Vertical-flow settlers

Vertical-flow settlers[5] are most commonly **square** in plan though **circular** ones are also used. They consist of two parts: an upper one in the shape of a **cuboid or cylinder** and a lower one in the shape of an **inverted pyramid or** a **cone**. Wastewater enters the tank through the **vertical feed pipe**, which terminates in an upward-facing bell-mouth. A cylindrical or cuboidal **central baffle** prevents flow from passing over the surface. To avoid ruffling of the settled sludge, there is also a **deflector** placed approx. 0.25–0.50 m below the lower end of a central baffle. The flowing wastewater emerges from beneath the central baffle and passes upward in the space between the baffle and the settler walls, leaving the tank through the v-shaped **weir** around the tank. Settled sludge is kept in the hopper of the settling tank and periodically drawn off under the **hydraulic pressure** of the tank's contents. Most vertical-flow settlers do not use any mechanical scrapers, instead relying on appropriately inclined walls in the conical part of the settler (at least 1–1.2 to 1).

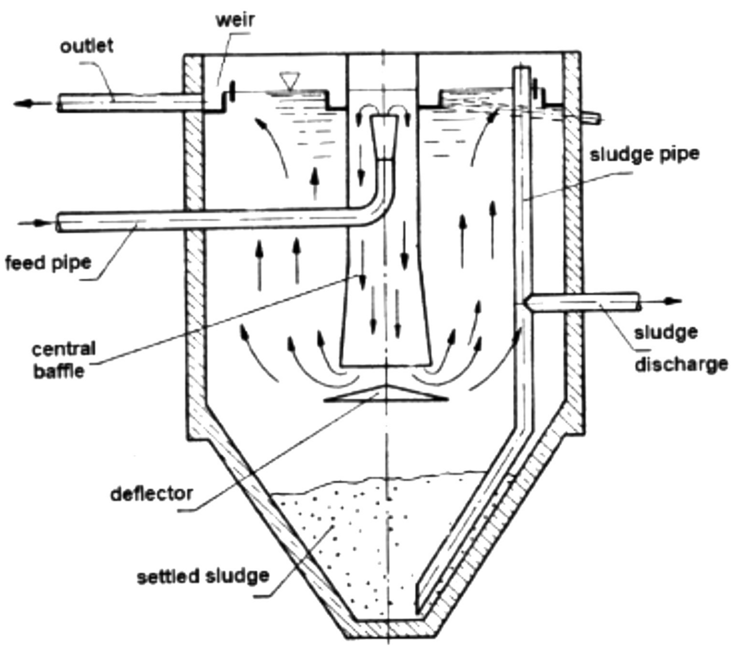

Fig. 22. Vertical-flow sedimentation tank

[5] These are sometimes also called '*upward-flow tanks*'.

The volume of the vertical flow is limited by its depth and the slope of the walls in the conical part. This is why settlers of this type are usually used in small wastewater treatment plants and container plants. The settler's surface area should be designed based on an assumed maximum surface hydraulic load that is appropriate for the settler's planned use (see Table 10). The length (or diameter) of its wall should not exceed 10 m, and the hydraulic loading of the weir's length should be ≤20 $m^3/(h \cdot m)$, as with other types of settlers. The length of the vertical feed pipe should be equal to the height of the settler's cuboidal (cylindrical) part—no less than 2.75 m—and its diameter should be sized for a flow velocity of ≤0.1 m/s. The input of the sludge drain tube should be at least 1.5–2.0 m below the wastewater's surface to ensure adequate hydraulic pressure for sludge discharge; the tube's diameter should be 20 cm.

3.3.2.4. Lamella clarifiers

The effectiveness of sedimentation can be enhanced with the installation of packets of inclined tubes or lamellas in settlers.[6] Tube and lamella packets use multiple tubular channels sloped at an angle of 45°–60° and adjacent to each other, which combine to form an increased effective settling area. Such packets applied in sedimentation tanks increase the settling capacity by reducing the vertical distance a floc particle must settle before agglomerating into larger particles. Lamellas can be operated in a counter-flow, parallel-flow, or cross-flow pattern as shown in Fig. 23.

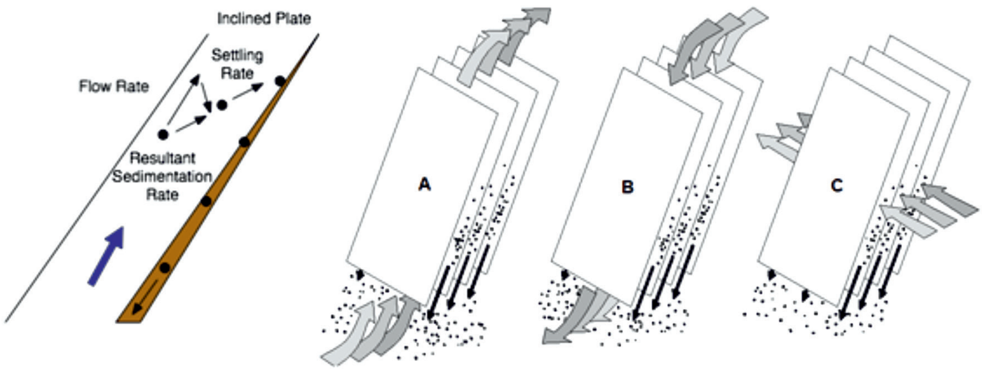

Fig. 23. Lamella clarification (left) and types of lamella settlers: counter-flow (A), parallel flow (B), and cross-flow (C) [25]

Settlers can be designed from the beginning as lamella clarifiers or existing settling tanks can be equipped with lamella or tubular packages in order to increase their capacity or efficiency of clarification. A settler with lamellas is usually compact, requiring only 65%–80% of the area of settlers operating without inclined plates. On the other hand, the settler can operate with overflow rates 2 to 4 times that of

[6] These are also called '*inclined plate settlers (IPS)*'.

traditional settlers. Lamellas installed in primary sedimentation tanks should be operated at a surface hydraulic load of <4 $m^3/(m^2 \cdot h)$ and a concentration of total suspended solids not exceeding 450 g/m^3.

Lamella clarifiers can be effectively used for separating flocculating particles, even in primary sedimentation, though not in all cases. This is because lamella clarifiers cannot be used for clarifying wastewater with a high variability of total suspended solids. They are most commonly used for separating flocculating iron and aluminium hydroxides after chemical precipitation. New constructions, which have been appearing for some time, allow more applications of lamella clarifiers.

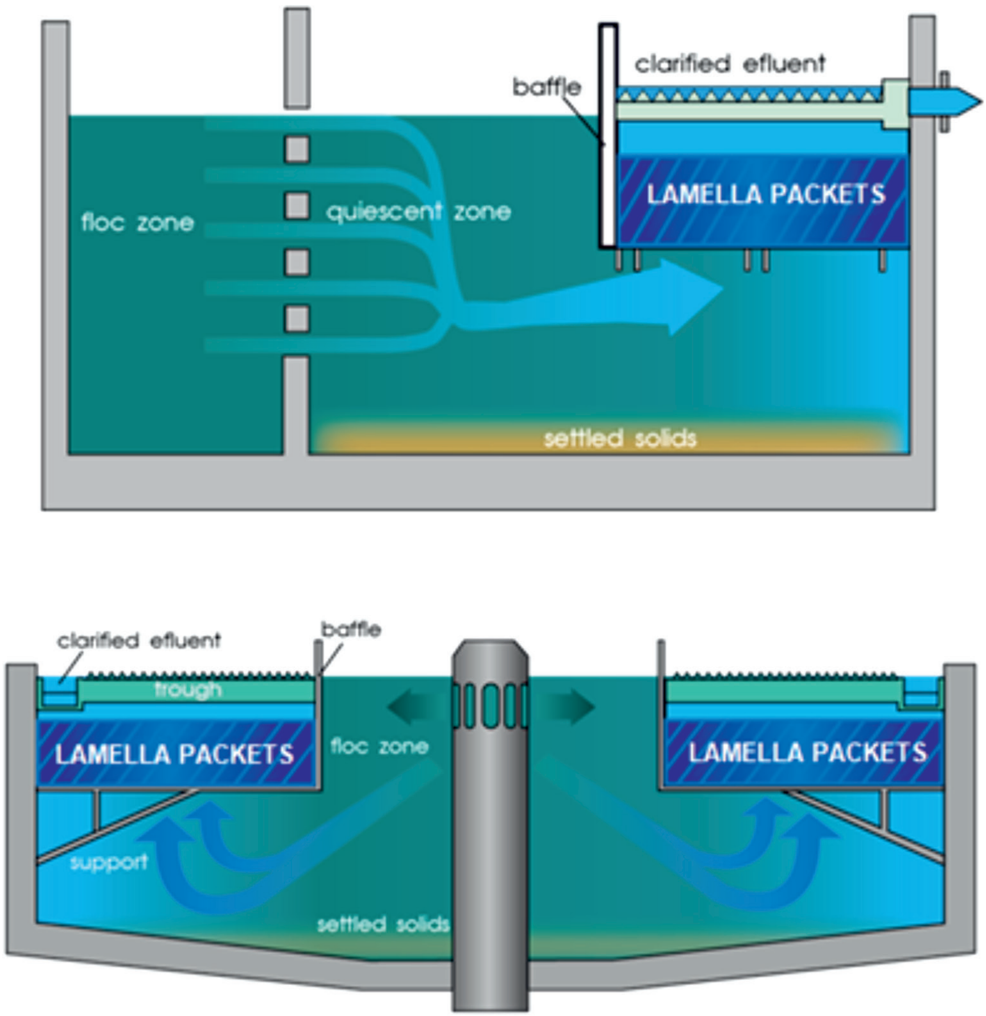

Fig. 24. Lamella packets installed in rectangular and radial sedimentation tanks (Eco-Potential Ltd.)

3.3.2.5. Imhoff tank

An **Imhoff tank** is a device for the primary sedimentation of wastewater that is often used in smaller wastewater treatment plants. Its characteristic feature is the physical separation of the **sedimentation par**t in the form of **horizontal-flow channels** from the **sludge chamber**, in which the **anaerobic digestion** of the retained sludge occurs. The Imhoff tanks may have a circular or rectangular shape with one or more flow channels.

The horizontal flow channels are designed in such a way that the wastewater will stay in them no longer than 1.5 hr. Hydraulic loading of the flow channel's surface should be <1.5 $m^3/(m^2 \cdot h)$. In cross-section, the flow channel is of a **complex shape**: in the upper part it is rectangular, and the lower part is an inverted triangle, terminated at the bottom with a slot 0.15–0.2 m in width, through which settling sludge slides into the sludge chamber. The slot at the bottom of the channel is protected by means of a **deflector** from the penetration of fermentation gasses from the sludge chamber.

The sludge chamber is designed so that the retained sludge can be **fermented** for about 150 days at a temperature of no less than 8–10°C. The sludge from the sludge chamber is removed gravitationally through a discharge pipe. Fermentation of the sludge reduces the content of organic matter by approx. 45%–50%, and the final water content is about 90% [19]. In smaller wastewater treatment plants, the biological sludge retained in secondary settling tanks and applied after the activated sludge process or biological filters is often directed to the sludge chamber of an Imhoff tank. In such situations, this chamber is used for anaerobic stabilization of all sludge produced in a plant.

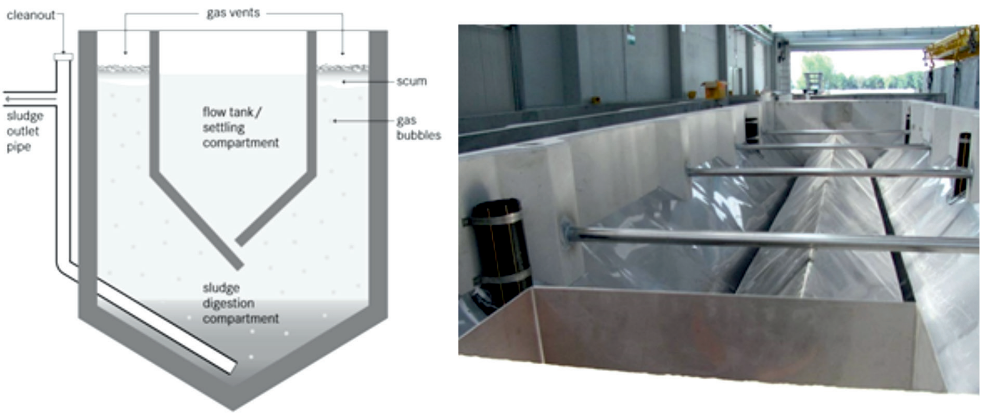

Fig. 25. Cross-section of an Imhoff tank (left) [21] and actual view of the flow channels in a rectangular tank (right) (GAZEBO)

3.3.3. CHARACTERISTICS OF PRIMARY SLUDGE

The volume and characteristics of the sludge produced during primary sedimentation must be known or estimated in order to properly design the sludge processing installation. The volume of primary sludge depends on:

- the characteristics of raw wastewater,
- the detention time in the primary settler and the effectiveness of sedimentation,
- the characteristics of settled solids (specific gravity, water content, etc.), and
- the period between operations for solids removal [11].

The sludge removed from primary sedimentation tanks has a very high water content (97.5%–98.5% by mass) and organic material [18]. The VSS content in primary sludge varies from 60% to 80%, with a typical value of 65% TS and organic acids from 200 to 2,000 g/m^3 (as CH_3COOH). It also contains nutrients such as nitrogen (2%–7% TS), phosphorus (0.4%–3% TS), and potassium (0.1%–0.7% TS). Primary sludge has a high potential for energy recovery—estimated to be 23,000–29,000 kJ/kg of TS—which can and should be used, e.g. for the production of biogas in an anaerobic digestion process [11].The sediments intercepted in primary sedimentation tanks sometimes show increased levels of heavy metals (such as cadmium, chromium, nickel, lead, and mercury) and various toxic substances which may create environmental hazards during processing and storage, thereby limiting the agricultural use of such sludge.

4. BIOLOGICAL TREATMENT PROCESSES

Historically, the primary objective of biological treatment was to reduce the oxygen demand (BOD and COD) of the treated wastewater so that it has no detrimental effect on the receiving waters. The increasingly more stringent effluent quality standards introduced in recent years—especially those regarding substances that stimulate eutrophication (nutrients)—have generated the need for nitrogen and phosphorus to be removed from wastewater [3]. The new technologies for biological wastewater treatment are a response to these challenges.

4.1. FUNDAMENTALS OF BIOLOGICAL PROCESSES

4.1.1. TYPES OF MICROBIAL METABOLISM

In their biochemical activities, microorganisms use different sources of carbon and different electron acceptors, and they generate different end products. Therefore, the following groups of bacteria can be distinguished: **aerobic autotrophes**, **aerobic heterotrophes**, **facultative heterotrophes**, and **anaerobic heterotrophes**. In order to grow and multiply, each bacterium must have access to sources of carbon (usually called a **substrate**), **energy**, and **nutrients** (nitrogen, phosphorus, sulphur, calcium, potassium, and magnesium). Each of the above-mentioned bacteria groups utilise these nutrients in their own way, as shown in Fig. 26.

Bacteria use the substrate as a food source by means of a series of complex reactions which may be catabolic (the substrate is broken down to release energy) or anabolic (energy is utilised to synthesise new cells). Biological reactions are controlled by enzymes, which are organic catalysts produced by living organisms. For example, enzymes catalyse reactions of **oxidation** (losing electrons), **reduction** (gaining electrons), **hydrolysis** (adding water), or **dehydrolysis** (losing water).

Bacterial growth occurs concurrently with the oxidation of organic or inorganic compounds. A part of the oxidised substrate is transformed into the growing bacterial biomass and the rest is used to maintain the metabolism of the bacterial cells. The mass of cells produced per unit of substrate removed is known as the bacterial **yield (Y)**. Yield is a stoichiometric parameter specific to both the type of microorganism (X) and the type of substrate utilised (S). This is why it is usually expressed in the form $Y_{x/s}$.

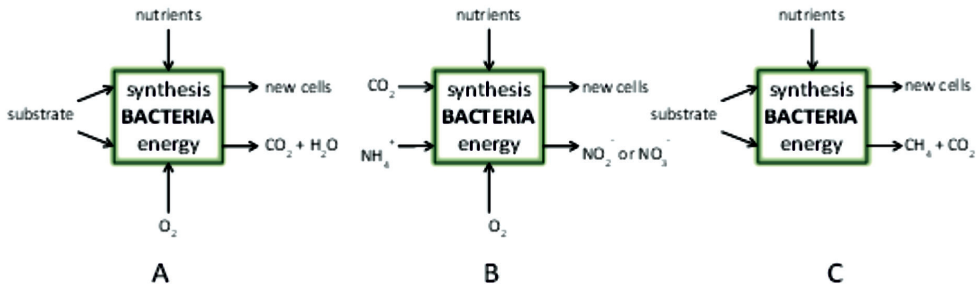

Fig. 26. Different types of microbial metabolism: A – aerobic heterotrophic; B – aerobic autotrophic; and C – anaerobic heterotrophic [11]

$$Y_{X/S} = \frac{\text{mass of biomass produced } (X)}{\text{mass of substrate utilized } (S)}$$

For wastewater containing a large number of organic compounds, the yield is usually calculated based on a measurable parameter that represents the overall consumption of organic material, such as COD. The biomass produced can also be expressed in COD units instead of VSS units, using the coefficient 1.42 g COD/g VSS, and in such case the yield can be presented in the following form (see also Table 11):

$$Y_{X/S} = \frac{\text{COD of biomass produced } (X)}{\text{COD of substrate utilized } (S)}$$

Table 11

Values of bacterial synthesis yield coefficients for different types of biological reactions (modified from [11])

Growth conditions	Oxidised compound (electron donor)	Oxidant (electron acceptor)	Synthesis yield (Y)	
			g VSS/g COD	g COD/g COD
Aerobic	organic compound	oxygen	0.40	0.57
Aerobic	ammonia	oxygen	0.12	0.17
Anoxic	organic compound	nitrate	0.30	0.43
Anaerobic	organic compound	organic compound	0.06	0.08
Anaerobic	acetate	CO_2	0.05	0.07

4.1.2. THE KINETICS OF BACTERIAL GROWTH

Almost all reactions occurring during biological wastewater treatment are catalysed by enzymes. According to the Michaelis-Menten equation, the **reaction rate of substrate utilisation** by bacteria depends on the **concentration of substrate**, which usually decreases during the reaction:

$$\frac{dS}{dt} = \frac{v_{\max} \cdot S}{S + K_{ms}}$$

where
v_{max} – maximum reaction rate (g/[m³·d]),
S – concentration of substrate (g/m³), and
K_{ms} – Michaelis-Menten constant (g/m³).

Biological degradation of the organic substrate is always related to the growth of microorganisms. Part of the substrate determined by a specific value of the yield coefficient ($Y_{x/s}$) is transformed into the biomass of the microorganisms. The growth rate of microorganisms is proportional to the concentration of substrate. This relationship can be expressed by **Monod's equation** (Fig. 27):

$$\mu = \frac{\mu_{\max} \cdot S}{S + K_s}$$

where
μ_{max} – maximum growth rate of microorganisms (1/d),
S – concentration of substrate (g/m³), and
K_s – half-saturation constant (g/m³).

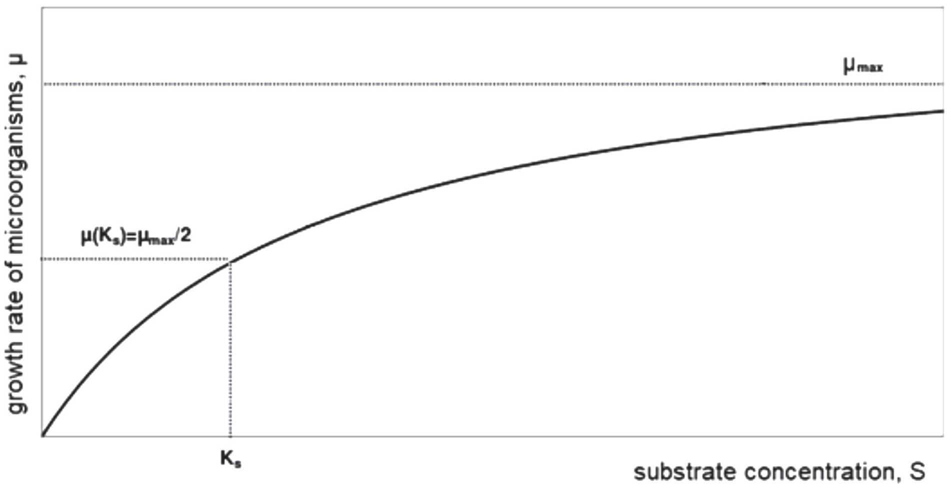

Fig. 27. The specific growth rate (**μ**) as a function of substrate concentration (S) according to Monod's equation

Monod's equation allows the calculation of the **growth rate of biomass** (*dX/dt*) in a biological reactor where the bacteria consume an amount of substrate (*S*) which is proportional to the bacteria biomass (*X*):

$$\frac{dX}{dt} = \mu X = \frac{\mu_{\text{ }} \cdot S}{S + K_s} X$$

Biochemical processes are strongly stimulated by **temperature**. The effect of temperature change within the range of 5–30°C on the maximum growth rate of microorganisms in a biological reactor can be expressed with the typical equation:

$$\mu_{\max}(t) = \mu_{\max}(20)\exp^{k(t-20)}$$

where

$\mu_{max}(t)$ – the maximum growth rate of microorganisms at temperature t (1/d),
$\mu_{max}(20)$ – the maximum growth rate of microorganisms at a temperature of 20°C (1/d),
t – temperature (°C), and
k – temperature constant (1/°C).

Considering that only part of the substrate (S) is used for the biomass increment ($Y_{x/s}$), the rate of substrate consumption (dS/dt) can be expressed by the following equation:

$$\frac{dS}{dt} = -\frac{\mu X}{Y_{x/s}} = -\frac{1}{Y_{x/s}} \frac{\mu_{\max} \cdot S}{S + K_s} X$$

where

$Y_{x/s}$ – the yield coefficient for the growth of biomass X on substrate S (gX/gS).

4.1.3. BIOLOGICAL NITROGEN REMOVAL

Biological removal of nitrogen from wastewater occurs during the metabolic dissimilation that is part of the processes of nitrification and denitrification. The final product of denitrification is nitrogen gas being released into the atmosphere. These processes require different conditions: nitrification needs aerobic conditions, long SRT, and low pollution loading, while denitrification requires a lack of oxygen and an organic substrate. Therefore, conducting these processes in a single-sludge system requires an understanding of their theoretical foundations.

4.1.3.1. Nitrification

The nitrification process is carried out by autotrophic organisms that use inorganic carbon in the form of CO_2 as a carbon source for biomass growth and energy from the oxidation of ammonium and nitrites. This reaction takes place in two stages:

1. Oxidation of ammonium nitrogen into nitrites by *Nitrosomonas* and *Nitrosococcus*:

$$55NH_4^+ + 76O_2 + 109HCO_3^- \rightarrow C_5H_7O_2N + 54NO_2^- + 57H_2O + 104H_2CO_3 +$$
$$+58 \div 84 \frac{\text{kcal}}{\text{mole}} N - NH_4$$

2. Oxidation of nitrites to nitrates by *Nitrobacter* and *Nitrocystis*:

$$400NO_2^- + NH_4^+ + 4H_2CO_3 + 195O_2 + HCO_3^- \rightarrow C_5H_7O_2N + 400NO_3^- +$$
$$+300H_2O + 15{,}4 \div 20.9 \frac{\text{kcal}}{\text{mole}}$$

Under normal conditions the first of these processes is slower and limiting for the entire nitrification process. The factors with the greatest impact on the nitrification process include substrate concentration, temperature, dissolved oxygen concentration, pH, SRT, and the presence of toxic substances. The autotrophic bacteria that carry out the nitrification have a slow maximum growth rate (μ_A)—within 0.3–0.8 d^{-1} at 20°C. Ammonia (K_{NH4}) and oxygen (K_{O2}) have similarly low half-saturation constants: 0.3–3.6 g $N\text{-}NH_4/m^3$ and 0.5–1.0 g O_2/m^3, respectively. Temperature in particular has an effect on nitrification, as the temperature coefficient (k) has a value in the range of 0.06–0.10$°C^{-1}$. The effect of ammonia (S_{NH4}) and oxygen (S_{O2}) concentrations on the nitrifying bacteria growth rate (μ_A) can be expressed with the multiple Monod equation:

$$\mu_A = \mu_{A,\max} \frac{S_{NH4}}{S_{NH4} + K_{NH4}} \frac{S_{O2}}{S_{O2} + K_{O2}}$$

where

$\mu_{A,max}$ – the maximum specific growth rate of nitrifying bacteria (1/d),
K_{NH4} – the half-saturation constant for ammonia (g $N\text{-}NH_4/m^3$), and
K_{O2} – the half-saturation constant for oxygen (g O_2/m^3).

4.1.3.2. Denitrification

In the denitrification process, heterotrophic microorganisms derive energy from the oxidation of organic compounds with nitrates as electron acceptors. In this process they reduce nitrates to nitrogen gas and other forms of nitrogen, such as N_2O, NO, and NO_2^-, which are undesirable and can be toxic. These by-products usually occur at very low concentrations and can only occur in higher amounts in the absence of organic material or highly variable process dynamics. The denitrification process takes place under anoxic conditions, when the redox potential is negative and nitrates are electron acceptors. An example reaction of denitrification using methanol (CH_3OH) as the carbon source occurs according to the stoichiometric equation shown below:

$$NO_3^- + \frac{5}{6}CH_3OH + \frac{1}{6}H_2CO_3 \rightarrow \frac{1}{2}N_2 + \frac{4}{3}H_2O + HCO_3^-$$

Many microorganisms in their metabolism have the ability to switch the final electron acceptor from oxygen to nitrates. However, in a system where both oxygen and nitrites are present, the bacteria will not lead to the denitrification process and oxygen respiration will occur. The kinetics of the denitrification process is consistent with Monod's equation, where the substrates are nitrates and organic compounds. The organic compounds contained in wastewater are a natural source of carbon (the internal substrate). A deficit of the substrate in the wastewater can lead to a reduction in the denitrification rate. This can be prevented by adding an external substrate. These external carbon sources include, but are not limited to, acetic acid, ethanol, methanol, glucose, methane, acetone, and others. Denitrification bacteria can use a broad range of substrates as an energy source, including those produced during acid fermentation of the primary sludge and then added to the reactor.

The effect of temperature on denitrification can be described with a typical equation and the value of the temperature constant (k) in the range of 0.06–0.12°C^{-1}. The optimal pH range is 7–9 but the effect of pH on the denitrification process is highly dependent on the duration of the process because microorganisms can gradually adapt to given conditions. At a pH of <7 the amount of N_2O produced increases.

The presence of oxygen, which inhibits the process, is essential for the course of the biological denitrification process. The oxygen concentration in the sludge flocs in the immediate vicinity of the bacterial cells is important, though it does not necessarily correspond to the measured concentration of dissolved oxygen. In many cases, this may lead to denitrification inside the activated sludge flocs located in the aerated chambers of the activated sludge. This phenomenon is called simultaneous denitrification, and it contributes an estimated 20% or even 100% of the total amount of influent nitrogen removed.

4.1.3.3. Biological phosphorus removal

Phosphorus is considered to be a limiting element in the process of eutrophication of water reservoirs. In typical municipal wastewater, phosphorus occurs in the form of:

- inorganic orthophosphates,
- inorganic polyphosphates, and
- organic phosphorus.

Polyphosphates are subject to hydrolysing into orthophosphates in wastewater, so the dominant form of phosphorus in wastewater undergoing biological treatment is orthophosphate, accounting for about 70% of the total phosphorus load. In biological wastewater treatment systems designed to remove carbon and nitrogen compounds, phosphorus is removed in quantities resulting from the metabolic demand of microorganisms determined by the stoichiometric composition of the bacterial cell. The phosphorus content in the dry mass of bacterial cells is about 2.3%, which in optimal conditions allows about 20% of the incoming phosphorus load to be removed. Increasing the effectiveness of the biological removal of phosphorus compounds is possible through the phenomenon of '**excess uptake of phosphates**' from the wastewater where certain groups of microorganisms store it in their cellular mass. The phenomenon itself was first observed already at the end of the 19th century, but intensive research in this field only began in the second half of the 20th century [12].

The process of excess phosphorus uptake is still not fully understood, but it can be presented in a simplified way. In the anaerobic zone, bacteria capable of excess phosphorus accumulation (**phosphorus-accumulating organisms [PAO]**) uptake carbon compounds in the form of short-chain volatile fatty acids (VFA) and store them in the form of poly-β-hydroxybutyrate (PHB). At the same time, the energy required for the uptake of carbon compounds derives from the decomposition of polyphosphates accumulated in biomass in the form of granules, releasing orthophosphates to the liquid phase. When these bacteria are under aerobic conditions in an environment where there is little readily-biodegradable substrate, they oxidise the accumulated PHB, releasing carbon compounds as well as energy in the form of adenosine triphosphate (ATP). The energy coming from ATP is used for the uptake of orthophosphates from the liquid phase, which are accumulated in the form of polyphosphates in granules in the cellular mass. The difference between the amount of phosphates taken up in the aerobic phase and released in the anaerobic phase determines the technological efficiency of the process. The cycle of the anaerobic and oxygen phase is shown in Fig. 28 and Fig. 29. This metabolic cycle makes PAOs perfectly adapted to compete with other microorganisms, which makes it possible for PAOs in treatment plants with highly effective biological phosphorus removal to account for up to 80% of the biomass.

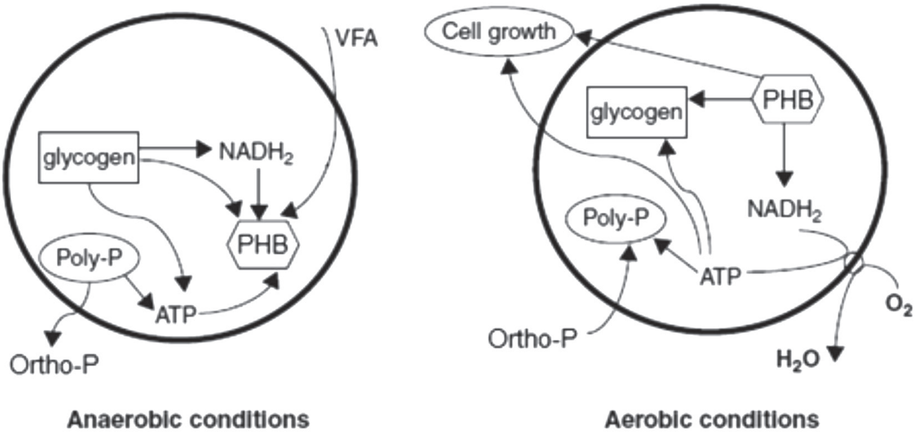

Fig. 28. Diagram of the anaerobic and aerobic metabolism during enhanced phosphorus removal

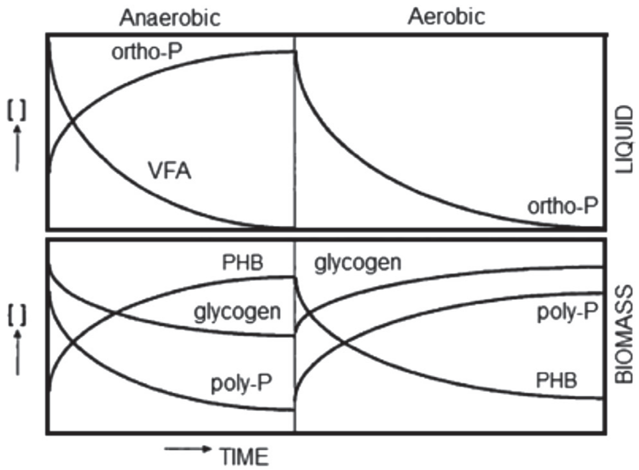

Fig. 29. Diagram representing the change in concentrations of ortho-P, poly-P, PHB, and VFA under the sequence of anaerobic/aerobic conditions during enhanced phosphorus removal

The reaction of PAOs to changes in environmental factors is similar to the reaction of denitrifying bacteria and aerobic heterotrophic bacteria. The influence of temperature on the growth of PAOs is much smaller than in the case of other groups of microorganisms. This is evidenced by the small value of the temperature constant (*k*): 0.01–0.02$^{o}C^{-1}$. The most important factors affecting excess phosphorus removal are **alternating anaerobic/aerobic conditions**, which has a selective function: creating preferential conditions for the growth of bacteria which accumulate phosphorus and **the absence of nitrates** in the anaerobic phase. The presence of nitrates negatively affects the metabolism of PAOs under anaerobic conditions, impairing the phosphorus accumulation mechanism. In addition, by creating favourable conditions for the denitrification process, readily-degradable carbon compounds (short-chain VFAs) are consumed by denitrifying bacteria. The lower concentration of VFAs available for phosphorus-accumulating bacteria results in a reduced phosphorus uptake in the aerobic phase.

4.2. ACTIVATED SLUDGE SYSTEMS

4.2.1. GENERAL CONCEPT OF AN ACTIVATED SLUDGE SYSTEM

In this method, mechanically treated wastewater is fed to a biological reactor filled with a suspension of microorganisms (called '**activated sludge**') that carry out the processes of decomposing pollutants contained in wastewater. The activated sludge is a living flocculent suspension composed mainly of heterotrophic bacteria. In addition, other organisms—such as protozoa, fungi, rotifers, diatoms, nematodes, and others—are also found in smaller numbers. The activated sludge is a heterotrophic biocenosis, in which the processes of organic matter decomposition greatly dominate in the processes of its synthesis.

The activated sludge has a loose, spongy structure, composed of small, flocculent creatures of various shapes. The main species of microbes that allow flocculation is *Zooglea ramigera*. However, other species of bacteria also have the ability to produce mucus, especially when wastewater contains large amounts of organic compounds.

In the basic version of the activated sludge process, the decomposition of pollutants occurs under aerobic conditions, and the sludge is aerated. The aeration of activated sludge has **two purposes**: firstly, it **provides oxygen** for the aerobic decomposition of pollutants provided with the wastewater, and secondly, it keeps the activated sludge in **suspension**, preventing it from settling to the bottom of the reactor. In more complex systems, the activated sludge reactor is divided into a series of zones with different oxic conditions, most often **anaerobic**, **anoxic**, and **aerobic**.[7] This configuration of the reactor allows not only the oxidation of organic material and biological nitrification, but also biological denitrification and excess phosphorus removal. Such reactors are called '**multi-phase biological reactors**' and in different configurations are now commonly used for the highly effective removal of carbon, nitrogen, and phosphorus compounds from municipal wastewater. In zones that are not aerated (anaerobic and anoxic), adequate **mechanical mixing** of the activated sludge must be ensured.

After pollutants are removed in the biological reactor, the treated wastewater must be separated from the activated sludge microorganisms. A **sedimentation** or **membrane filtration** process can be used for this purpose. Sedimentation is the most common separation method used today, but membrane processes are increasingly popular because of the many advantages they offer. In this situation, we are talking about '**biological membrane reactors (MBRs)**'.[8]

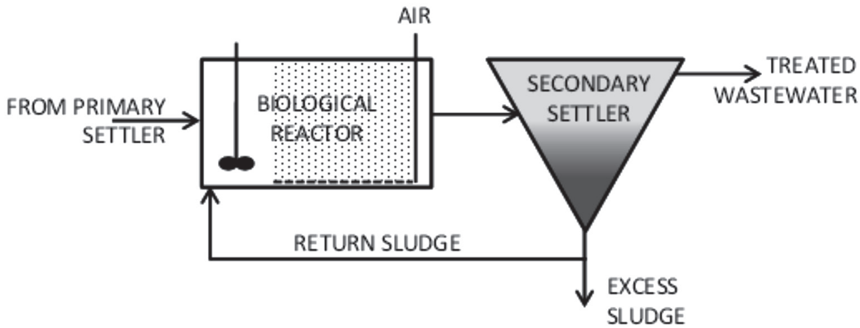

Fig. 30. General diagram of the activated sludge process

[7] Anaerobic conditions mean there is a total lack of oxygen. Anoxic conditions mean that a certain amount of oxygen is acceptable (<0.3–0.5 g/m^3), usually being a product of the reduction reaction. Anoxic conditions are typical in the biological denitrification process.

[8] MBRs are described in more detail in a separate subchapter.

Separation of the activated sludge biomass from wastewater in the sedimentation process is called '**secondary sedimentation**' and takes place in **secondary settling tanks**. These settling tanks are constructed similarly to primary settling tanks. However, the sedimentation process occurring within is more complex due to the high concentration of strongly flocculating particles of the activated sludge. Also, the detention times are longer than in primary sedimentation. These devices are described in more detail in the next chapter. The vast majority of the activated sludge (about 99% by volume) separated from treated wastewater in a secondary settling tank is recycled to the biological reactor as a **return sludge** in order to maintain a high concentration of biomass in the process. The remainder (about 1%), called **excess sludge**, is discharged from the settler for sludge processing, including thickening, stabilizing, and dewatering. The total dry mass of the excess sludge is proportional to the total mass of impurities decomposed by the bacteria in the activated sludge reactor. The basic components of an activated sludge system are shown in Fig. 30.

4.2.2. PARAMETERS OF THE ACTIVATED SLUDGE PROCESS

4.2.2.1. Mixed Liquor Suspended Solids (MLSS)

MLSS is the most fundamental parameter regarding activated sludge systems. The 'mixed liquor' is a mixture of wastewater and the biomass of activated sludge microorganisms. MLSS provides information on the concentration of suspended solids in the mixed liquor and is usually expressed in milligrams per litre (mg/l) or grams per cubic meter (g/m^3). Typical values are in the range of 2,000–5,000 mg/l. In highly effective multi-stage biological reactors, MLSS is usually maintained in the range of 4,500–5,000 mg/l in order to ensure a high rate of the biological processes. MLSS is usually monitored online with special probes and controlled by changing the amount of excess sludge discharged from the system. A similar parameter is Mixed Liquor Volatile Suspended Solids (**MLVSS**). While MLSS represents the TSS in the mixed liquor, the MLVSS represents the organic fraction of biomass (VSS), which is indicative of the population of bacteria in the activated sludge process. An increase in the ratio of MLVSS to MLSS indicates an increase in the bacterial population, whereas a decrease represents a decrease in the bacterial population.

4.2.2.2. Food-to-Mass ratio (F/M)

The **F/M ratio** is an important control parameter of activated sludge commonly used in practice. It shows the ratio of food (expressed as mass of BOD_5) entering the activated sludge system to the total mass of microorganisms (expressed as the mass of VSS) in the system. It is usually expressed in g BOD_5/(g MLVSS·d) or equivalent units.

$$\frac{F}{M} = \frac{BOD_5 \cdot Q}{MLVSS \cdot V}\left[\frac{gBOD_5}{gMLVSS \cdot d}\right]$$

where
BOD_5 – the value of BOD_5 entering the activated sludge system,
Q – the wastewater flow rate,
$MLVSS$ – the value of MLVSS in biological reactor, and
V – the volume of activated sludge system.

F/M values can range from 0.05 to even 5 g BOD_5/(g MLVSS·d), but in typical activated sludge systems it does not exceed 0.6. The F/M value determines which processes take place in the activated sludge. Nitrification and the aerobic stabilization of activated sludge can be expected at **low F/M** values (approx. 0.05–0.15 g BOD_5/[g MLVSS·d]). Excessively **high F/M** values may cause the appearance of filamentous bacteria, resulting in deterioration of the sludge settling properties and its bulking.

4.2.2.3. Solids Retention Time (SRT)

The **Solids Retention Time**, sometimes also called '**sludge age**', is another crucial parameter of the activated sludge system. The SRT is the time after which all the activated sludge in the system will be replaced with the newly growing microorganisms. This parameter is expressed in units of time, usually in days. It is calculated according to the following formula:

$$\mathrm{SRT} = \frac{MLSS \cdot V}{QX_e + Q_s X_s}\left[d\right]$$

where
$MLSS$ – the value of MLSS in the biological reactor,
V – the volume of the activated sludge system,
Q – the wastewater flow rate,
X_e – the TSS in effluent from the secondary settler,
Q_s – the volume of excess sludge wasted per day, and
X_s – the TSS of the excess sludge.

There is a relationship between SRT and sludge growth. This relationship shows that as the SRT increases, the sludge growth per unit of BOD_5 utilised decreases. At the same time, the BOD_5 removal efficiency decreases with shorter SRTs and higher F/M ratios (Fig. 31).

The **minimum required SRT** value for each microorganism group can be calculated according to the following formula as a function of microorganism growth rate (μ):

$$SRT_{\min} = \frac{1}{\mu}$$

Each microorganism group has a different expected minimum sludge age. Autotrophs, being the slowest growing bacteria group, require longer SRTs than most heterotrophs. When designing an activated sludge system, the sludge age (as well as the reactor volume) should be adapted to the microorganisms with the lowest growth rate (usually nitrifying autotrophs).

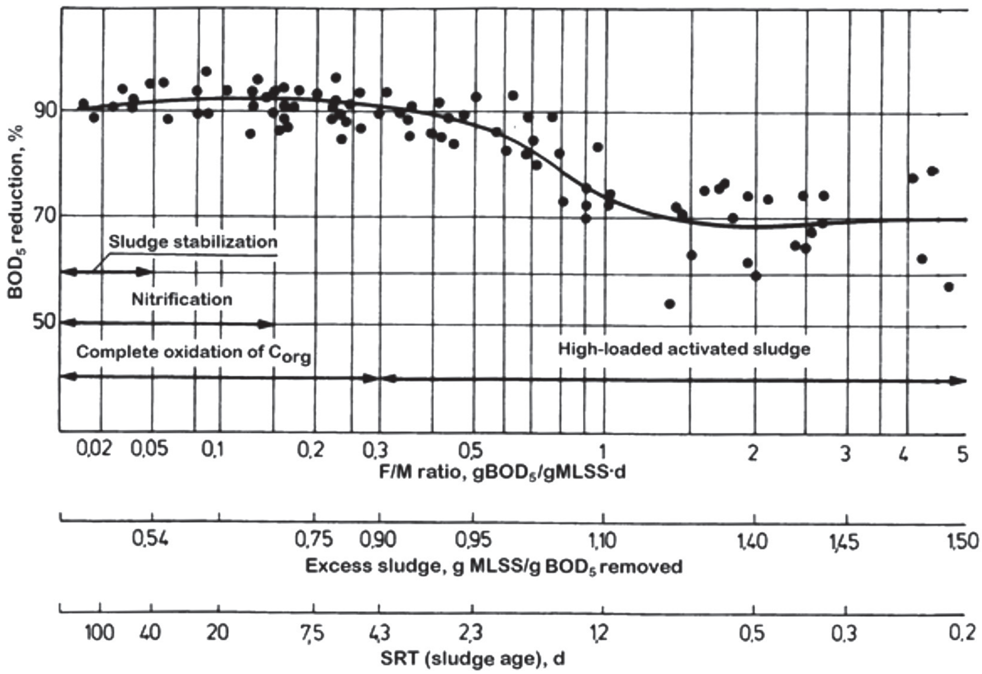

Fig. 31. The efficiency of BOD_5 removal in activated sludge systems is a function of the F/M ratio, excess sludge mass, and sludge age [4]

4.2.2.4. Sludge Volume Index (SVI)

The **Sludge Volume Index** is a parameter that describes the sedimentation properties of activated sludge. It expresses the volume (in cm^3) occupied by the mass of 1 g of sludge after 30 minutes of sedimentation in an Imhoff funnel.

$$SVI = \frac{\text{volume of sludge after 30 min. of sedimentation}}{\text{MLSS}}\left[\frac{cm^3}{g}\right]$$

The SVI ranges from about 40 to 300 ml/g, with values in the range of 80–120 ml/g indicating good sedimentation properties of the sludge. When SVI exceeds 150 ml/g, it means the sludge has a tendency towards bulking and accumulating on the surface of the secondary settling tank. Sludge bulking is caused by filamentous bacteria, which develop mainly in sludge, which is excessively loaded with BOD and is hypoxic.

4.2.3. TYPICAL CONFIGURATIONS OF MULTI-STAGE REACTORS

The practical application of technology for highly effectively removing nutrients in single-sludge systems consists in the integration of oxidation of organic compounds, nitrification, denitrification, and biological excess phosphorus removal in a system with one secondary settling tank. The first attempts to integrate the nutrient removal processes, combining the oxidation of carbon compounds and biological nitrification with the process of biological denitrification in aerobic/anoxic systems, were made in the early 1960s by Ludzack and Ettinger.

The better understanding of the phenomenon of excess phosphorus removal meant that the removal of carbon and nitrogen also began to be integrated with the removal of phosphorus. The beginnings of highly effective technology for the integrated removal of carbon, nitrogen, and phosphorus compounds from wastewater in activated sludge systems date back to 1974, when Barnard observed a relationship between the presence of nitrates in effluent and the removal of phosphorus. This prompted him to introduce an additional anaerobic zone in systems for removing carbon and nitrogen compounds.

Since then, many systems have been developed for the integrated biological removal of nutrients from wastewater. A characteristic feature of these systems is the presence of three zones—anaerobic, anoxic, and aerobic—which, with appropriate sludge recirculation systems, enable all of these processes to be carried out in a single-sludge system. The following is a concise description of the most commonly used configurations of reactors for the biological integrated nutrient removal from wastewater **Błąd!**.

4.2.3.1. Modified Ludzack–Ettinger system

The modified Ludzack–Ettinger system is a simple system with pre-denitrification, used in systems where the biological removal of phosphorus from wastewater is not required. The efficiency of denitrification depends to a large extent on the intensity of nitrate recirculation from the aerobic to the anoxic zone. This relationship is presented in **Błąd!** .

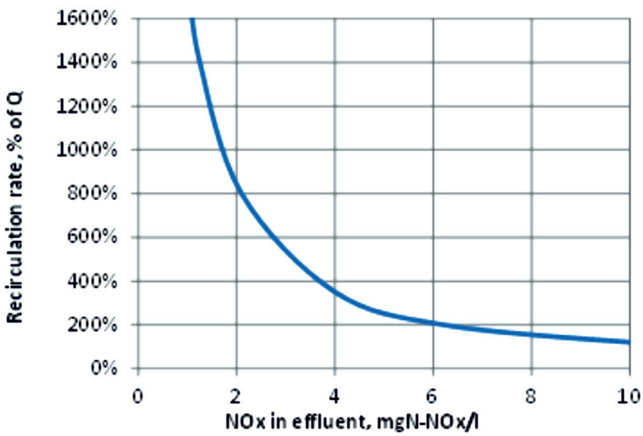

Fig. 32. The effect of internal recirculation on NOx in the effluent of systems with pre-denitrification

4.2.3.2. 4-stage Bardenpho

The Ludzack–Ettinger system was modified with the addition of a second anoxic zone in order to denitrify the nitrates escaping from the aerobic zone and the final, small aerobic zone. This aerobic zone was designed to nitrify the ammonium nitrogen formed from the decay of biomass cells. The system, known as the 4-stage Bardenpho, is characterised by highly efficient nitrogen removal from wastewater, but requires a reactor with a relatively large total volume.

4.2.3.3. 5-stage Bardenpho

Barnard developed a system in 1975 to expand the 4-stage Bardenpho system for the excess phosphorus removal process. This system makes it possible to achieve very low concentrations of nitrogen and phosphorus in treated wastewater. It is possible to operate the system by dosing the external substrate before the second anoxic zone, which allows its volume to be reduced. Barnard states that when removing 2–4 g $N\text{-}NO_3/m^3$ in the second anoxic zone, using an external substrate in the form of methanol is economically viable.

4.2.3.4. A_2/O (3-stage Bardenpho)

This system, also known as the 3-stage Bardenpho system, was proposed by Barnard in 1983 as a simplified version of the 5-stage Bardenpho. The lack of a second anoxic zone means that the concentration of nitrates in effluent can be relatively high. Only some of the nitrates are reduced in the return sludge recirculated to the anaerobic zone, and their presence can negatively affect the process of excess biological phosphorus removal. In this system, the phenomenon of simultaneous nitrification/denitrification taking place in the aerobic zone plays a large role.

4.2.3.5. Johannesburg

The system is sometimes also called the 'modified Bardenpho'. In this system, the problem of nitrates in the anaerobic zone was solved by introducing an additional anoxic zone (pre-denitrification) for the denitrification of nitrates contained in the return sludge. The sludge concentration in the pre-denitrification zone is twice the concentration in other zones. The organic compounds needed for denitrification in the pre-denitrification zone are supplied with a portion of the incoming wastewater (10%–20% of Q).

4.2.3.6. UCT

The idea of the UCT process was to prevent nitrate from entering the anaerobic zone. The nitrate-containing sludge is recirculated to the initial part of the anoxic zone, and then from the end of this zone to the anaerobic zone. The process should be carried out so that the total denitrification of nitrates takes place over the length of the anoxic zone.

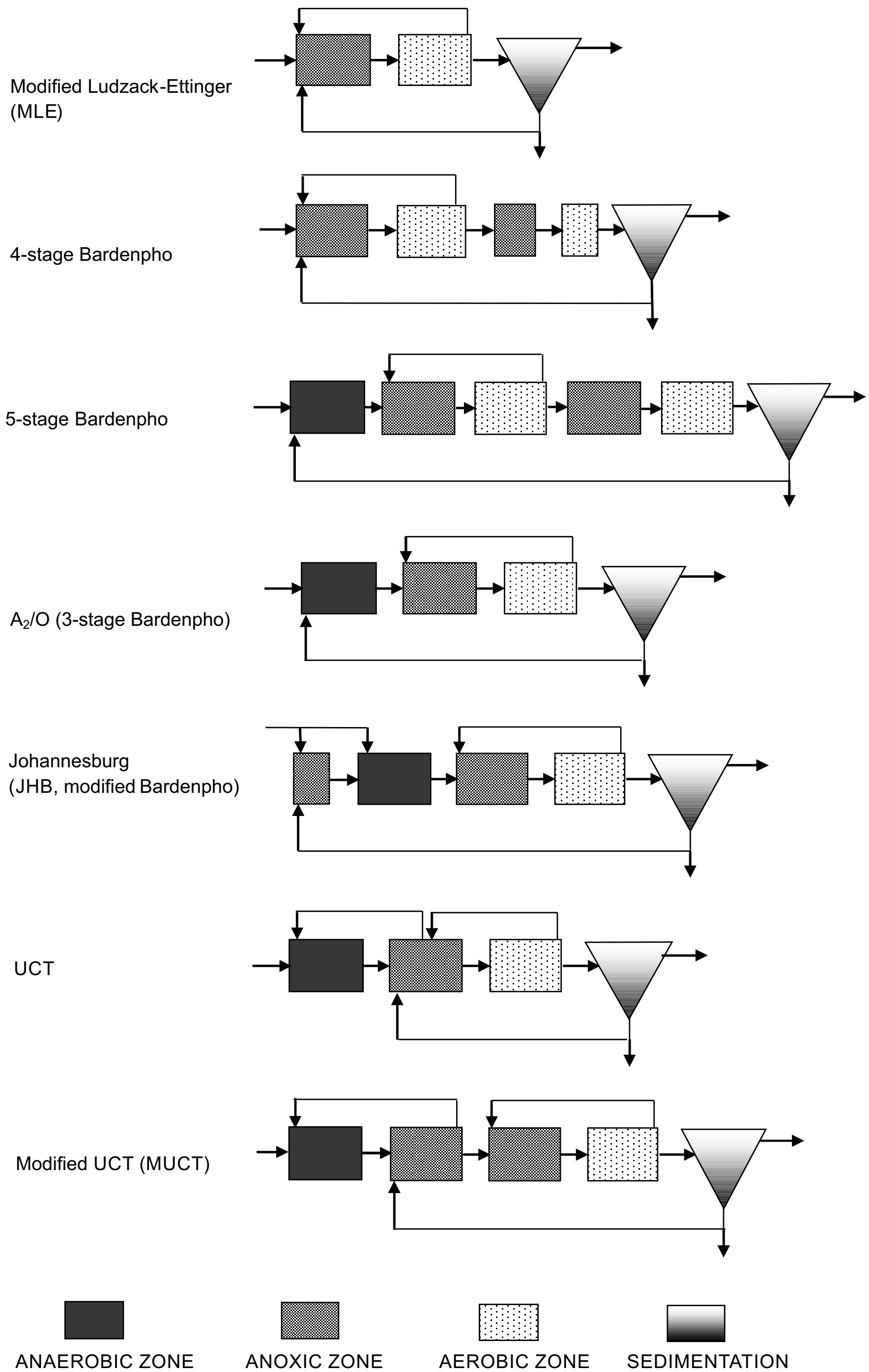

Fig. 33. Typical configurations of multi-stage activated sludge reactors for the integrated nutrient removal from wastewater

4.2.3.7. Modified UCT (MUCT)

Operational difficulties related to maintaining the required concentration of nitrates in the final part of the anoxic zone led to the development of a modified version of the UCT process. The change consisted in introducing an additional anoxic zone after the anaerobic one, in which a reduction of nitrates contained in the return sludge would take place with the use of carbon compounds remaining after the anaerobic process. This system protects the anaerobic zone against nitrates contained in the recycled sludge, but is difficult to operate.

4.2.4. SEQUENCING BATCH REACTORS

The **sequencing batch reactor (SBR)** is a tank equipped with aeration and mixing systems, and a decanter used to drain clarified wastewater. All processes that usually take place in the biological flow-through reactor and the secondary settling tank are conducted here **in one device** in the appropriate **time sequence**. In a wastewater treatment plant using this technology, 2–4 SBR reactors are usually used to ensure the continuity of the plant's operation and the appropriate length of the reactor's operational cycle. The reactors sequentially receive incoming wastewater and carry out the entire treatment cycle. After the end of the operational cycle, the wastewater is discharged from the reactor to the **equalising tank**, and then to the receiver.

The SBR cycle consists of a number of different **phases**, during which subsequent wastewater treatment processes take place, as with the flow-through reactor: the oxidation of organic compounds, nitrification, denitrification, excess phosphorus removal, as well as sedimentation and excess sludge discharge. A typical SBR cycle consists of the following phases:

- **Filling** – Raw wastewater flows into the tank and mixes with the activated sludge held in the tank. Aeration is on or off ('static filling') during the entire filling phase or its part depending on the planned strategy for wastewater treatment.
- **Reaction** – The influent is off and the mixed liquor of sludge and wastewater is aerated for a specified time. The structure of the reaction phase can be more complex, involving, for example, a sequence of aeration and mixing to achieve nitrification and denitrification.
- **Sedimentation** – During this phase, the reactor acts as a secondary settling tank and the treated wastewater is separated from the biomass of the activated sludge.
- **Decanting** – During decanting, treated wastewater is discharged to the equalising tank, from where it will flow evenly into the receiver.
- **Idle** – The reactor has completed the treatment cycle and is waiting for the next cycle to start.

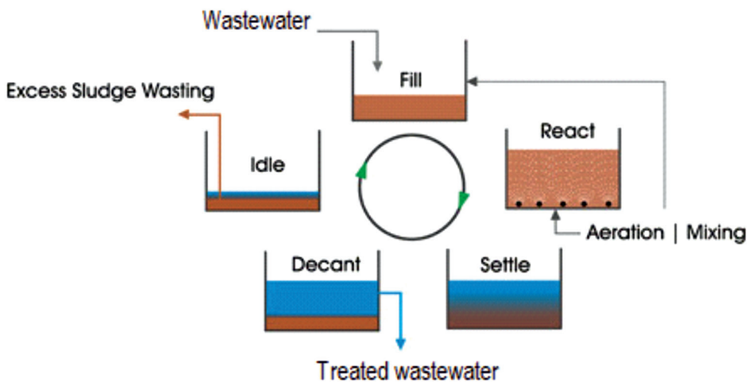

Fig. 34. Diagram of the SBR cycle

For the processes of nitrification, denitrification, and the removal of excess phosphorus to take place in an SBR reactor, an appropriate sequence of anaerobic, anoxic, and aerobic conditions must be applied. **Nitrogen removal** is carried out by performing preliminary denitrification of the nitrates remaining in the sludge after the previous cycle—during the process of feeding wastewater into the reactor (filling and mixing)—and then nitrification of the ammonia during aeration of the activated sludge (reaction). The combined **removal of nitrogen and phosphorus** requires the introduction of an **anaerobic phase** as well. In this situation, the reactor is filled under anoxic and anaerobic conditions, with the latter being extended to the reaction phase. The anaerobic phase is followed by aeration of the sludge, followed by another sequence of the anoxic and aerobic phases. The SBR cycle for N and P removal may appear as in Fig. 35.

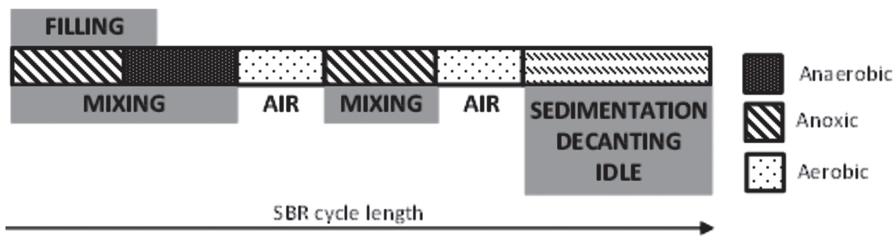

Fig. 35. The SBR cycle configuration for combined N and P removal [16]

The main technological parameters for the design and operation of SBRs are as follows:

- **Total cycle length (t_c)** – This is the sum of the duration of all phases of the cycle, usually 4, 6, or 8 hours, though other cycle lengths can be used.
- **Filling time (t_f)** – This is the time it takes to fill the reactor with wastewater. Under '**slow filling**' this time must be adapted to the total length of the cycle (t_c) and the number of SBRs operated in the plant (n) according to the following rule: $t_f = t_c/n$.

If a retention tank is used in the treatment plant before the SBR reactors, the filling time can be shortened accordingly—this is called '**quick filling**'. The filling time (slow or quick) affects the course of the pollution removal in the reactor.

- **Filling time coefficient (f_{tf})** – This is the ratio of filling time (t_f) to the duration of the entire cycle (t_c).
- **Volume of wastewater and activated sludge** remaining in the reactor after completion of the previous cycle (**V_o**)
- **Volume of wastewater discharged** from the reactor at the end of the cycle (**ΔV_w**) – This value is subject to fluctuations, which—with the simultaneous variability of concentration in raw sewage—leads to the variability of the load of impurities removed in the reactor in subsequent cycles.
- **Exchange coefficient (f_e)** – This is the ratio of the volume of wastewater discharged from the reactor in one cycle (ΔV_w) to the total active volume of the reactor ($V_{\max}$): $f_e = (\Delta V_w)/V_{\max}$. Values of 0.2–0.5 are used, depending on the operational strategy. Leaving the sludge and wastewater reserve in the SBR reactor after the end of the cycle replaces the recirculation in the flow-through systems. When comparing the SBR with the flow-through system, the SBR exchange coefficient f_e = 0.5–0.2 corresponds to the recirculation rate α = 1–4 in the flow-through system,according to the following formula:

$$f_e = \frac{1}{1+\alpha}$$

4.2.5. MEMBRANE BIOLOGICAL REACTORS

Membrane biological reactors (MBR) are devices in which the processes of biological wastewater treatment occur in the same way as in ordinary activated sludge reactors, but the process of biomass separation from treated wastewater occurs in the results of membrane filtration. In this way, **secondary settling tanks are eliminated** from the wastewater treatment system, and their role is taken over by the membranes.

A **membrane** used for wastewater treatment is a **semipermeable barrier** that can be made of various materials which allow various substances to pass through it, while stopping all others. The driving force that causes filtration through the membrane is usually the pressure of the filtered liquid exerted on one side of the membrane. The effectiveness of membrane filtration depends on many factors, the most important of which is the pore size of the membrane, the pressure applied, any fouling of the membrane surface, and the properties of the filtered wastewater. In the case of municipal wastewater treatment, microfiltration membranes (MF) with pore sizes of >50 nm are most often used; less frequently, ultrafiltration membranes (UF) with pore sizes of 2–50 nm are used. Larger membrane pore sizes mean lower filtration pressure is needed, and thus less energy is consumed. On the other hand, larger particles pass through larger pores, which worsens the overall filtration effect.

However, membrane filtration even at the MF level stops practically all suspensions present in the mixed liquid of the activated sludge, including virtually all bacteria, with relatively low filtration pressures (<0.3 mH_2O). This filtration efficiency is sufficient for most municipal sewage treatment plants at relatively low operating costs.

Membranes can have different shapes and forms. The typical forms used in internal systems include **flat-sheet** and **hollow-fibre** membranes. External systems use **spiral** and **pipe** membranes. Regardless of the form, the membranes are always consolidated into packages called **modules**. The reason for this is to facilitate the operation and replacement of membranes and to reduce the piping and control systems required. **Membrane modules** can be placed inside the reactor ('internal' or 'immersed' system) or outside the reactor ('external' or 'side-stream' system).

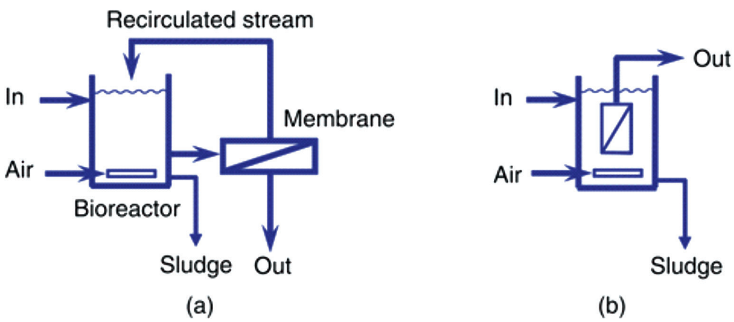

Fig. 36. External (a) and internal (b) configurations of MBRs [6]

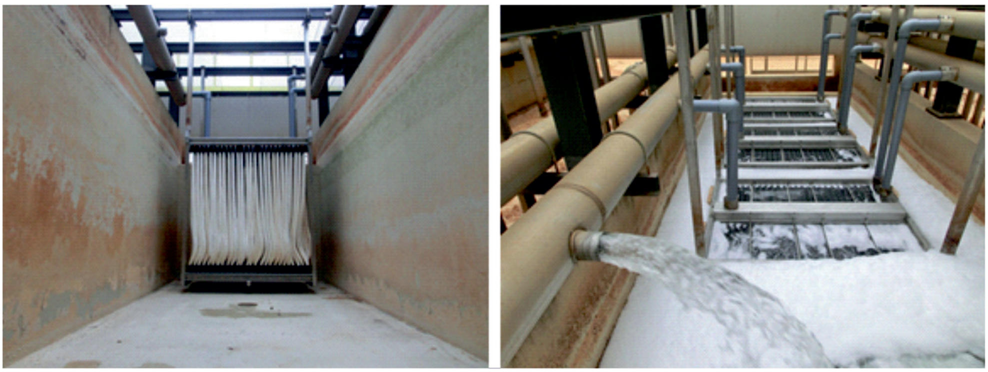

Fig. 37. Hollow-fibre membrane module installed in an empty MBR (left) and an MBR with membrane modules (right) (RIELLI)

MBRs allow for **more intense** wastewater treatment processes. The unit processes carried out in such reactors are the same as in an ordinary activated sludge reactor (oxidation of organic compounds, nitrification, denitrification, and excess phosphorus removal), but they take place under different operating conditions (Table 12). A typical MLSS in MBR is about 2–3 times greater than that of a conventional activated sludge system (10–15 vs 5 kg/m^3). Also, the typical SRT is much longer in an MBR, which in turn increases sludge mineralization (lower values of the VSS/MLSS ratio) and much lower unit sludge growth. Thus, the overall mass of excess sludge removed from an MBR system is approx. 30%–40% lower than that of a comparable conventional activated sludge reactor.

Table 12

Comparison of preferred operational parameters of a conventional activated sludge (CAS) reactor and an MBR [6, 28]

Parameter	Unit	CAS	MBR
MLSS	kg/m^3	3.5–5	8–12
MLVSS	kg/m^3	3–4	6–10
SRT	d	10–15	15–45
HRT	h	4–12	4–20
F/M (BOD_5 loading rate)	g BOD_5/(g MLSS·d)	0.16–0.24	0.04–0.12
F/M (BOD_5 loading rate)	g BOD_5/(g MLVSS·d)	0.2–0.3	0.05–0.15
Oxygen Uptake Rate (OUR)	mg O_2/(dm^3·h)	6–12	2–5
Specific OUR (SOUR)	mg O_2/(g MLVSS·h)	20–40	15–50
Observed sludge yield	g VSS/g COD	0.3–0.5	0.26–0.33
Flux (internal system)	L/(m^2·h)	–	10–30
Flux (external system)	L/(m^2·h)	–	30–40
Aeration intensity (per membrane area)	m^3/(m^2·h)	–	0.24–1.28
Effluent BOD_5	g/m^3		< 3
Effluent TSS	g/m^3		< 1
Effluent TN	gN/m^3	8–15	< 3
Effluent TP	gP/m^3	0.8–2	< 0.05

4.3. ATTACHED GROWTH SYSTEMS

4.3.1. THEORETICAL BASIS

Unlike in activated sludge systems, in attached growth systems (**biofilters**, **biological filters**, **and trickling filters**), microorganisms do not float freely in the mixed liquor of sludge and wastewater, but they grow on the surface of the material filling the biofilter. The rate of biochemical processes taking place in such a system depends on the total mass of microorganisms and the contact area of the biological film (**biofilm**) with wastewater and the air. The mechanism that determines the rate of penetration of individual components (pollutants) into the biofilm is **diffusion**. In the outer layer of the biofilm with a thickness of approx. 2–3 mm, **aerobic** conditions prevail, while **anaerobic** conditions exist in the inner layer.

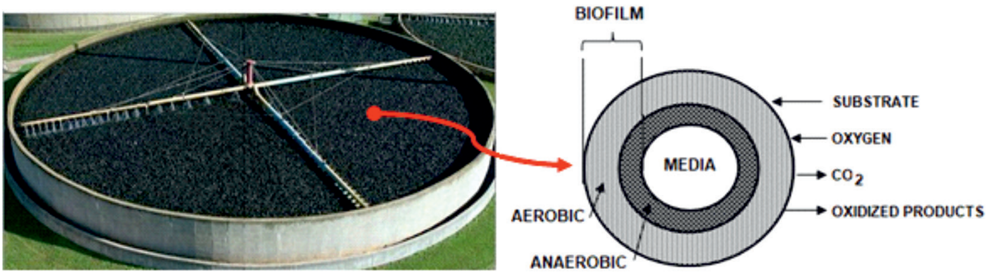

Fig. 38. Cross-section of the microbial biofilm in an attached growth system

Diffusion of pollutants to the internal layer is limited and most of the wastewater treatment processes occur under aerobic conditions in the outer layer. As the biofilm increases in thickness, the substrate in the wastewater is oxidised before it can penetrate the inner layer of the biofilm. Bacteria in the inner layer enter an endogenous respiration state and lose their ability to cling to the media surface. The wastewater flow then washes the biofilm off the filling media, and a new biofilm layer starts to grow. This loose of the biofilm layer is called '**sloughing**' and it is primarily a function of the organic and hydraulic load on the biofilters [11].

Treatment systems using biofilters usually require the use of **secondary settling tanks**, just like activated sludge reactors, in order to separate treated wastewater from sloughed biofilm. The settled biomass is not recirculated back to the biofilter, but removed as **waste sludge**. Treated wastewater effluent from the secondary settling tanks is usually **recirculated** before the biofilter in order to maintain the required hydraulic load to enable sloughing of the biofilm.

Attached growth processes are commonly used in small– and medium-capacity wastewater treatment plants. The have a number of **advantages** over activated sludge processes, including a smaller energy demand, simple operation and maintenance, good sludge thickening properties, and effective secondary sedimentation with no sludge bulking problems. The **disadvantages** may include poorer effluent quality in terms of BOD, TSS, nitrogen, and phosphorus, and a greater sensitivity to low temperatures. However, these drawbacks can be eliminated by using multi-stage biofilters or hybrid systems combining attached biomass with activated sludge systems.

4.3.2. TRICKLING FILTERS

4.3.2.1. Classification and applications

Modern **trickling filters** can be classified into several groups: for the oxidation of carbon compounds (high rate), the combination of oxidation of carbon compounds and nitrification (intermediate rate), and separate nitrification (low rate) (Table 13). Anaerobic trickling filters can also be used to carry out separate denitrification in multi-stage systems.

Table 13

Classification of trickling filters (modified from [11, 24])

Design parameter	**High rate**	**Intermediate rate**	**Low rate**
Filling media	random media, rock media, cross-flow media, vertical flow media	random media, rock media, cross-flow media	random media, cross-flow media
Technological effect	cBOD removal only	partial nitrification	full nitrification
Hydraulic loading ($m^3/[day·m^2]$)	10–40 (rock) 10–75	4–10 (rock) 13.7–88	1–4 (rock) 35.2–88
Organic loading (kg $BOD_5/[m^3·d]$)	0.4–2.4 (rock) 0.6–3.2	0.24–0.48	n/a
Nitrogen loading (kg NH_3-N/$[m^2·d]$)	n/a	0.2–1.0	0.5–2.4
Effluent quality	15–30 g/m^3 BOD_5 and TSS	<10 g/m^3 BOD_5 <3 g/m^3 NH_4-N	0.5–3 g/m^3 NH_4-N
Depth (m)	1.8–2.4 (rock) ≤12.2	1.8–2.4 (rock) ≤12.2	1.8–2.4 (rock) ≤12.2
Energy demand (kW/1000 m^3)	6–10	2–8	2–4

4.3.2.2. Design

The trickling filter consists of a housing made of reinforced concrete or steel, filling media (natural or plastic), influent, a wastewater distribution system, and effluent. Wastewater is supplied to the trickling filter through a **vertical pipe** placed in the centre of the filter and connected by means of a **rotary central head** with several horizontal **distribution pipes**. The sprinkling pipes rotate thanks to the reaction force of the wastewater flowing out through the horizontal holes. The rotational speed of the sprinkler arms is usually in the range of 0.5–2 per minute. Wastewater is distributed on the surface of the filter and flows downwards. During contact of the wastewater with the biofilm, the bacteria present in the biofilm uptake the substrate from wastewater and oxidise them. The **contact time** between wastewater and biofilm, i.e., the hydraulic loading of the filter's surface and the **pollution loading**, largely determine the effectiveness of wastewater treatment. The wastewater flowing downwards meets the counter-stream of air flowing from the bottom upwards. The air is supplied to the filter through special **windows** located just above the trickling filter's bottom. The treated wastewater flows down to the specially formed bottom of the filter, and is then discharged through the drainage pipe to the **secondary settling tank**.

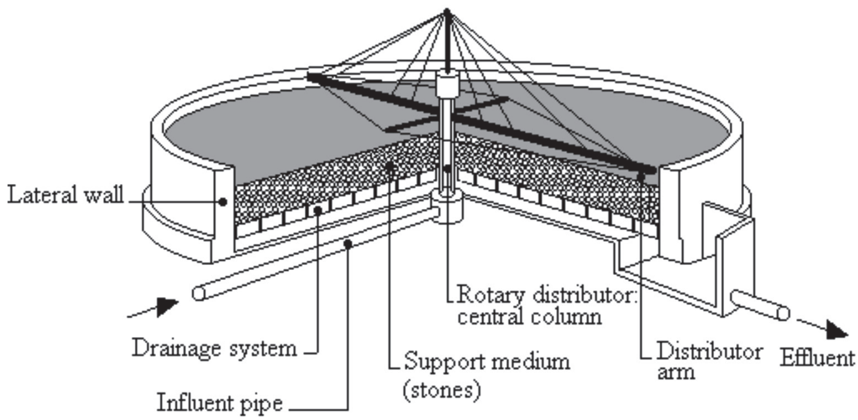

Fig. 39. Construction of a typical circular trickling filter [5]

The type and properties of media used as a filling material in the trickling filter are of great importance for its efficiency. This material should have a **large specific surface area** (area per unit of volume) and adequate mechanical strength. A large specific surface ensures a large contact area between the wastewater and the biofilm and enhances the overall process rate. In the past, **river rocks** or **crushed stones** (75–100 mm in diameter) were used as filling material. Currently, **plastic materials** with a large specific surface and a low specific weight are used for this purpose. The **low specific mass** of the filling material allows for the construction of taller filters, thus increasing the contact time of the wastewater with the biofilm as well as the efficiency of the treatment.

The plastic materials used as filling media in trickling filters can have different forms: **random filling media** or large, **flow-oriented rectangular modules**. Random filling media can have different shapes which are often protected as patented designs. The modular filling has the appearance of rectangular honeycomb modules stacked on top of each other. Each layer of modules is turned at right angles to the previous layer to further improve wastewater distribution. The most commonly used modular media types are **cross-flow media (XF)**, which are made of sheets formed with corrugations alternating at 60° to horizontal and vertical flow media. It has been found that 60°-cross flow media are the most efficient media for secondary treatment and nitrification applications, and that **vertical flow media (VF)** are more suitable for high-load applications. Examples of the different types of random and modular fillings are shown in Fig. 40.

The **height** of the biofilter depends on the filling material used and the BOD_5 value of the influent. The more concentrated wastewater requires a higher trickling filter bed. Biofilters with heavy natural filling cannot be as tall as those using light plastic fillings. The heights are usually in the range of 1.8 to 2.4 m for natural fillings, but can even reach up to 12.2 m for modular flow-oriented fillings.

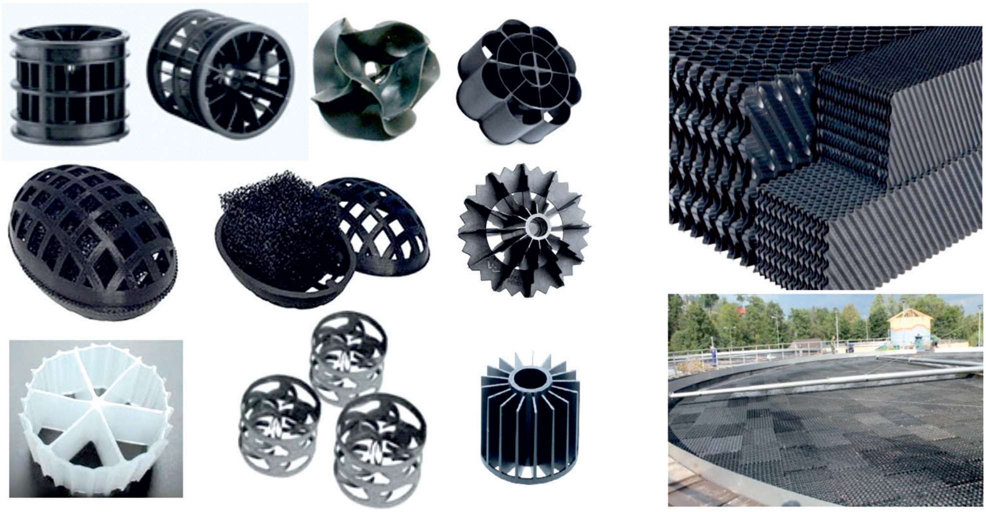

Fig. 40. Examples of random (left) and modular (right) packing media applied in trickling filters

4.3.2.3. Technological parameters

The **hydraulic loading** of the biofilter's surface is the primary operational parameter. It is usually expressed in ($m^3/[m^2 \cdot d]$) or equivalent units. The value of this parameter determines the wastewater flow velocity through the bed, and thus the contact time between the biofilm and the treated wastewater. In addition, it provides adequate shear force, which removes fragments of excessively thick biofilm growing on the bed's packing. The recommended values of this parameter depend on the type of trickling filter (low or high loads) and on the type of filling media (natural or plastic) (see Table 13).

BOD_5 loading of the trickling filter is another important parameter. It expresses the load of pollutants in the form of BOD_5 per unit volume of the filter's filling [kg $BOD_5/m^3 \cdot$day]. For each type of biofilter, there is an optimum BOD_5 loading range that ensures aerobic conditions are maintained. If the load is too large, anaerobic conditions may first occur locally, then gradually encompassing the entire bed, causing a decrease in the treatment efficiency. Typical values for different types of trickling filters are presented in Table 13. It is usually assumed that in wastewater treated on trickling filters the BOD_5 value should not exceed 100–150 g/m^3.

Wastewater recirculation is always required during the operation of trickling filters. This ensures the required hydraulic loading of the filter's surface and an adequate concentration of pollutants in the influent, which prevents anaerobic conditions from developing in the filter's bed. The recirculation rate (R) is expressed as the ratio of the flow rate of recycled wastewater (Q_r) to flow rate of raw wastewater (Q_w): $R = Q_r/Q_w$. The required recirculation rate is set individually and based on the composition of incoming wastewater and the specific design of the trickling filter. By increasing the recirculation rate, lower biofilters—but with a larger surface area—can be used in order to achieve similar technological effects.

4.3.2.4. Typical configurations

Biological filters can be used as **single**- or **multi-stage systems**, depending on the required technological effects. **Single-stage** systems are used for the simple treatment of wastewater with a low concentration of organic compounds and for partial nitrification (see Fig. 41A).

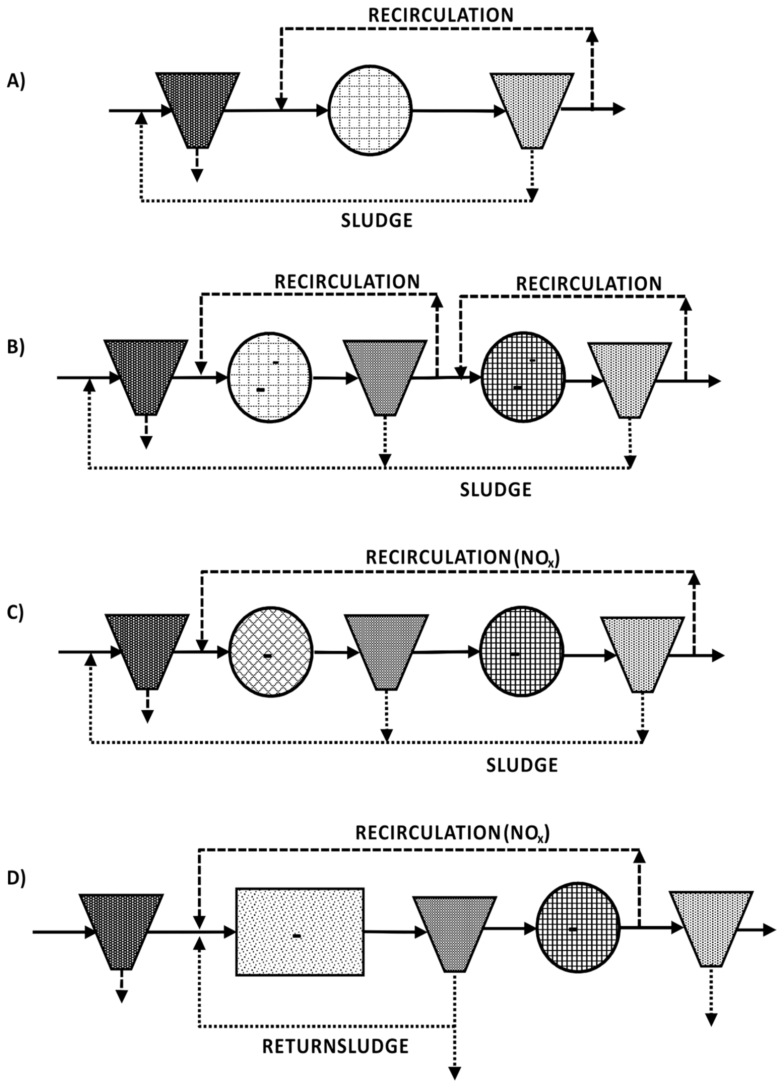

Fig. 41. Examples of different configurations of biological filters: (A) a single-stage layout with a primary settler and a trickling filter; (B) a two-stage layout for BOD_5 removal (1st stage) and full nitrification (2nd stage); (C) a two-stage layout for BOD_5 removal and pre-denitrification (anaerobic 1st stage) and full nitrification (aerobic 2nd stage); (D) a hybrid two-stage layout with an anoxic activated sludge reactor for BOD_5 removal and pre-denitrification (1st stage) and aerobic biofilter for full nitrification (2nd stage)

Two-stage systems are often used for the treatment of **highly concentrated wastewater**. They are usually separated by means of an **intermediate clarifier** to remove solids generated in the first biofilters. In this arrangement, the first biofilter removes most of the carbon compounds ($cBOD_5$), while the **nitrification** process takes place in the second biofilter, which is possible due to its low $cBOD_5$ load (see Fig. 41B).

The removal of nitrogen from wastewater through biological **nitrification and denitrification** also requires a two-stage layout consisting of aerobic and anaerobic biofilters. In such a system, the **pre-denitrification** layout (the anaerobic biofilter is placed upstream of the aerobic one) is most often used and the nitrate-rich effluent from the aerobic bed is recycled before the anaerobic bed for denitrification (see Fig. 41C). The layouts with **post-denitrification** (the aerobic biofilter is placed upstream of the anaerobic one) are less frequently used because they require the dosing of an external carbon source to an anaerobic bed for denitrification to occur.

Yet another possible solution is a **hybrid layout** combining an activated sludge reactor with a biological filter. A hybrid system uses an activated sludge reactor in the first stage to remove organic compounds and for denitrification along with a low-loaded biological filter for nitrification in the second stage. The nitrate-rich outflow from the nitrifying biofilter is recirculated to the inflow of the biological reactor for denitrification (see Fig. 41D).

4.3.3. ROTATING BIOLOGICAL CONTACTORS

Rotating Biological Contactors (RBC) are a modification of the typical attached growth systems that consist of a series of closely spaced circular disks of polystyrene or PVC that are approx. 40% submerged in wastewater and rotated through it. The **cylindrical plastic discs** have a diameter of approx. 3.5 m and are attached to a **horizontal shaft** of approx. 7.5 m in length [11]. The disks **rotate slowly** at a speed of about 1–1.6 revolutions per minute, driven by an electric motor. The **biofilm grows on the rotating discs**, being intermittently in contact with the treated wastewater while immersed and with atmospheric air while ascending.

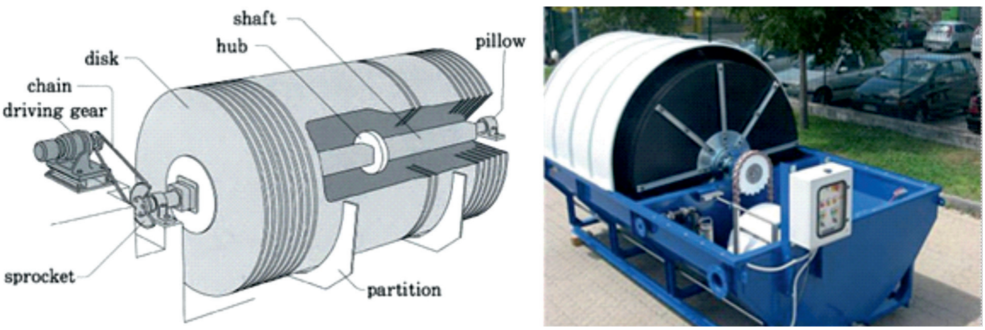

Fig. 42. Diagram and photo of the Rotating Biological Contactor (RBC) (photo: Aqua-Aerobic Systems, Inc.)

The wastewater **treatment method** in an RBC is similar to wastewater treatment in trickling filters. Both devices use a large area of biofilm growing on the filter media and the rely on the contact of the biomass with an organic substrate (wastewater) and atmospheric oxygen. In an RBC, as in biofilters, the treatment efficiency and the possibility of nitrification depend to a large extent on the BOD_5 load. Typical **operational parameters** specific to the application of RBCs are presented in Table 14.

Table 14

Typical RBC parameters for different applications [11]

Parameter	Unit	Treatment level		
		BOD removal	**BOD removal and nitrification**	**Separate nitrification**
Hydraulic loading	$m^3/day{\cdot}m^2$	0.08–0.16	0.03–0.08	0.04–0.10
Organic loading	$gBOD_5/(m^2{\cdot}d)$	8–20	5–16	1–2
Max. 1^{st} stage organic loading	$gBOD_5/(m^2{\cdot}d)$	24–30	24–30	
NH_3 loading	kg NH_3-N/$(m^2{\cdot}d)$		0.75–1.5	
Hydraulic retention time	h	0.7–1.5	1.5–4	1.2–3
Effluent BOD_5	$gBOD_5/m^3$	15–30	7–15	7–15
Effluent NH_3-N	g NH_3-N/m^3		<2	1–2

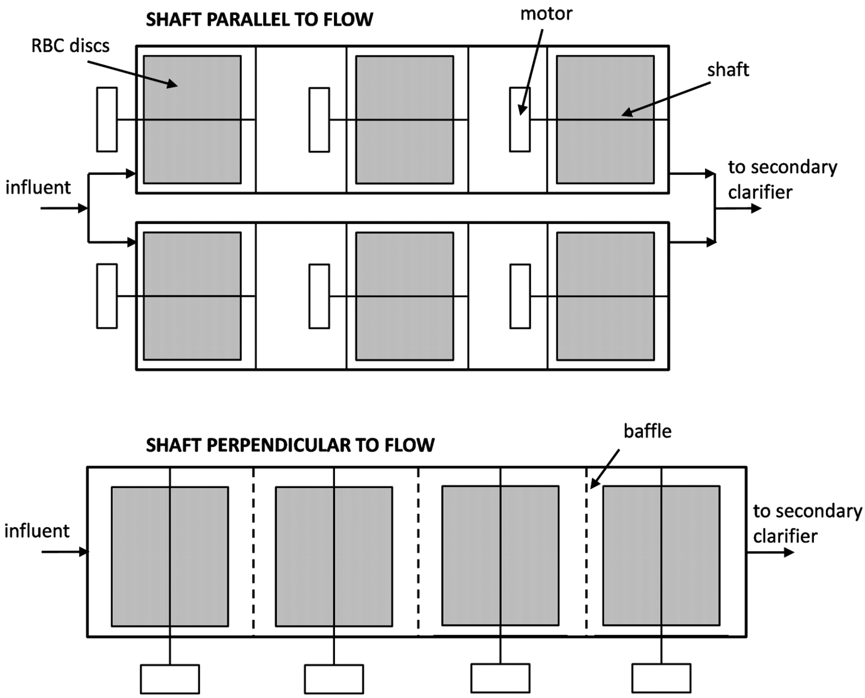

Fig. 43. Different arrangements of RBC units: parallel and perpendicular to the direction of flow (based on [11])

Due to the sensitivity of the processes occurring in an RBC to the influent organic load, they are usually **staged**, which allows for differential loading of successive filters. The RBC process typically consists of a number of individual units operated **in series** or a larger unit with sections separated by baffles. Usually, 2–4 units are sufficient for BOD removal and 6 or more are required for nitrification. Typical staging arrangements are presented in Fig. 43. In small plants, the RBC drive shafts are **oriented parallel** to the direction of wastewater flow, while in larger plants they are **perpendicular**, with several stages in a series forming a process train [11]. The greatest advantages of an RBC are its simplicity, reliability, and low energy consumption for high technological efficiency. They are also quite resistant to a reduced inflow of pollutants, even in the long-term, which means they are suitable for treating wastewater from periodically used sites such as holiday homes, hotels, spas, etc.

4.4. SIMPLIFIED AND NATURAL TREATMENT SYSTEMS

Natural wastewater treatment systems use **simple biochemical processes** occurring spontaneously in the environment, which lead to the consumption of substances contained in wastewater. These processes occur with **low intensity**, which makes it necessary to use objects with **large areas and volumes**. This makes it difficult to control and monitor such processes, and their negative impact on the environment is often difficult to assess. For this reason, these methods are usually used to treat small volumes of wastewater or as a final post-treatment step after applying the conventional intensive treatment processes. On the other hand, natural treatment processes are characterised by **low energy consumption**, which is a significant advantage they hold over conventional systems, especially with activated sludge. The most common natural methods include:

- septic tanks with drainage fields,
- constructed wetlands, and
- lagoons.

4.4.1. SEPTIC TANKS WITH DRAINAGE FIELDS

This type of system consists of a **septic tank**, in which wastewater is pre-treated, and a **drainage field**, which distributes wastewater to the ground, where it is biologically treated and disposed of. A septic tank is an **anaerobic tank** where suspended solids present in the wastewater settle to the bottom and are subject to bacterial digestion. If the hydraulic retention time (HRT) is long enough (approx. 10 days), some of the dissolved pollutants in the wastewater are also anaerobically decomposed. This causes the wastewater to be partially treated biologically and decreases the overall pollution load in the effluent from the septic tank. Wastewater treated this way is sent to **drainage pipes** to soak into the surrounding soil, where further natural treatment processes can take place.

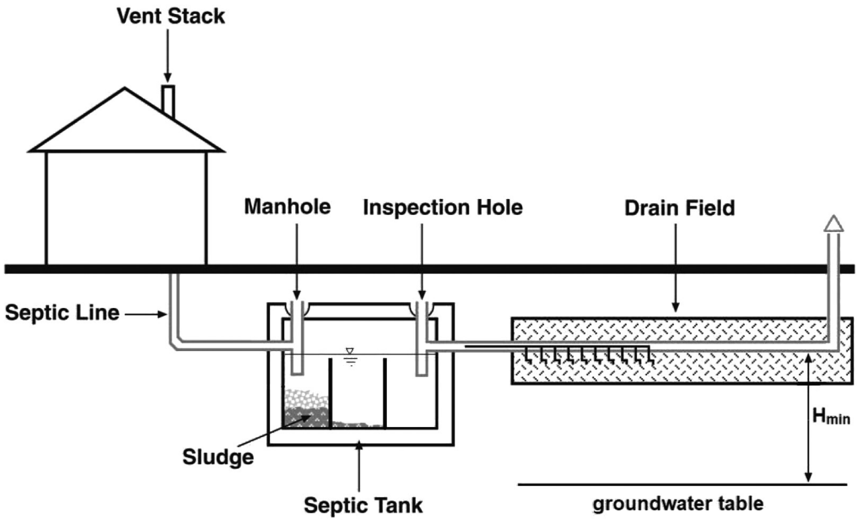

Fig. 44. Diagram of a treatment system with a septic tank and drainage field

In order for such a treatment to be implemented, the following general conditions must be fulfilled:

– The **soil properties** must enable the percolation of wastewater. Therefore, such a drainage system cannot be used in impervious soils such as clays or silts.
– The **underground water table** must be at the appropriate depth below the drain field (usually >2 m), so that the percolating wastewater has time to be cleaned before reaching the underground water.
– The **drainage pipes** must be of an appropriate length for the volume of treated wastewater and must be arranged horizontally in a suitable backfill.
– The **septic tank** should have adequate volume (an HRT of >2.5 days) and be divided into at least two separate chambers with adequate ventilation.
– The **sludge** from the tank should be removed at least once a year and taken to a wastewater treatment plant.

In addition, the **minimum distances** of the drainage field to the water wells and of the septic tanks to neighbouring buildings must be maintained. In Poland, the drainage field systems can be used to treat and discharge **no more than 5 m^3/d** of wastewater.

4.4.2. CONSTRUCTED WETLANDS

Wastewater treatment in **constructed wetlands** is based on the decomposition of pollutants contained in wastewater through microorganisms living in the soil, aided by the root system of **macrophytes**. Wastewater undergoing treatment flows through a reservoir in which macrophytes have been planted. As the wastewater flows through the wetland, it is treated by the processes of sedimentation, filtration, oxidation, reduction, adsorption, and precipitation. Plant roots **uptake** nutrients from the wastewater, and some pollutants are retained and **adsorbed** on grains of bed filling the reservoir. This method of wastewater treatment enables the efficient removal of organic pollutants, but the removal of N and P is variable and depends to a large extent on local conditions. The **efficiency** of constructed wetlands **decreases over time**, particularly with respect to the removal of **phosphorus**. This is due to the fact that phosphorus is largely removed by adsorption on the bed grains, and over time the sorption capacity of the soil is gradually exhausted (after approx. 7–10 years).

Both **vascular plants** (taller plants) and **non-vascular plants** (algae) are important in constructed wetlands. Constructed wetlands are usually planted with emergent vegetation (non-woody plants that grow with their roots in the substrate and their stems and leaves emerging from the water surface). Common emergents used in constructed wetlands include bulrushes, cattails, reeds, and a number of broad-leaved species [23].

Constructed wetlands can be categorised as either **free-water systems (FWS)** or **subsurface flow systems (SFS)**. An **FWS wetland** consists of a shallow basin, soil or other medium to support the roots of vegetation, and a water control structure that maintains a shallow depth of water (Fig. 45, 46). The water surface is above the bed. An **SFS wetland** consists of a sealed basin with a porous substrate of rock or gravel. SFS systems are known by several names, including vegetated submerged beds, the root zone method, microbial rock reed filters, and plant-rock filter systems [23]. Constructed wetlands can also differ in the direction of wastewater flow: it can be **horizonta**l or **vertical**. In most of the systems in the United States, the flow path is horizontal, although some European systems use vertical flow paths.**FWS wetlands** have areas of open water and are similar in appearance to natural marshes. They are the nearly exclusive choice for the treatment of urban, agricultural, and industrial storm waters, because of their ability to deal with pulse flows and changing water levels. In **horizontal SFS wetlands**, the water is kept below the surface of the bed and flows horizontally from the inlet to the outlet. In general, horizontal SFS wetlands are used for smaller flow rates than FWS wetlands, probably because of cost and space considerations. These systems are capable of operating under colder conditions than FWS systems, because of the ability to insulate the top. A key operational consideration is the propensity for the media to get clogged. **Vertical SFS wetlands** exist in several variations. The most common type, used most often in Europe, employ surface flooding (pulse loading) of the bed in a single-pass configuration. Very concentrated wastewaters can be treated in vertical SFS systems [7]. Constructed wetlands must always be preceded by at least primary sedimentation to remove suspended solids and avoid clogging of the wetland's bed.

Constructed wetlands can be used as secondary wastewater treatment or for post-treatment of wastewater after intensive biological treatment processes (activated sludge or trickling filters). Its main advantage is the extremely low energy demand (<0.1 kWh/m^3 of treated wastewater). However, due to the low rate of the biochemical and physical processes occurring in such a system, this technology is only applicable for the treatment of small volumes of wastewater generated in rural areas or small towns (i.e. <2,000 p.e.). Another important limitation in the use of constructed wetlands is climatic conditions, because in winter the efficiency of the processes taking place there is much lower.

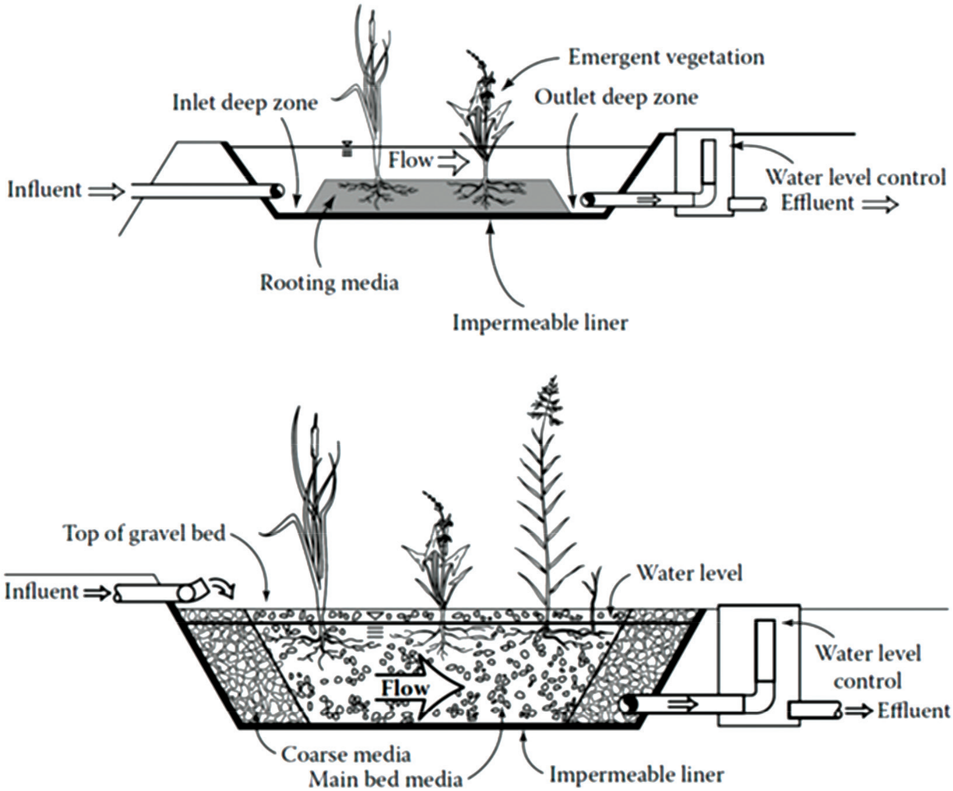

Fig. 45. Two major types of constructed wetlands: a Free Water Surface (FWS) wetland (upper) and a Subsurface Flow System (SFS) wetland (lower)[7]

Fig. 46. Examples of free-water system (FWS) (left) and subsurface flow system (SFS) (right) wetlands

4.4.3. WASTEWATER LAGOONS

Wastewater lagoons (ponds) are a simple and easy method of treating small volumes of wastewater in natural conditions. These are relatively shallow reservoirs in which wastewater is kept for a period of 3–50 days, during which the pollutants contained in wastewater are naturally decomposed. The processes occurring in lagoons resemble natural self-purification processes that occur in the water environment. The rates of pollutant transformations are low; thus the use of lagoons requires the availability of large areas of land. The treatment processes can be stimulated by additional aeration and mixing.

Lagoons can be **anaerobic**, **facultative**, **or aerobic**, depending on the processes we want to carry out. Aerobic and facultative lagoons can be **aerated** or not. Non-aerated aerobic lagoons are usually shallower (0.3–1.5 m) than anaerobic (2.5–5.0 m) and facultative (1–2 m) ones, which ensures penetration of atmospheric air throughout its depth. Aerated lagoons can be as deep as 2–6 m. Usually, lagoons are combined into systems that allow the processes of BOD oxidation, nitrification, and denitrification to be carried out in the following ways:

- anaerobic → facultative → aerobic
- anaerobic → aerated → aerobic
- aerated → aerobic with macrophytes (e.g. *Lemma minor*)

Like other natural methods, wastewater lagoons are used to treat smaller volumes of wastewater and are usually built away from human settlements because of their potential to become a nuisance (birds, insects, mosquitoes, etc.). It is quite common to use lagoons for final stabilization of wastewater after the processes of intensive biological treatment in small communities.

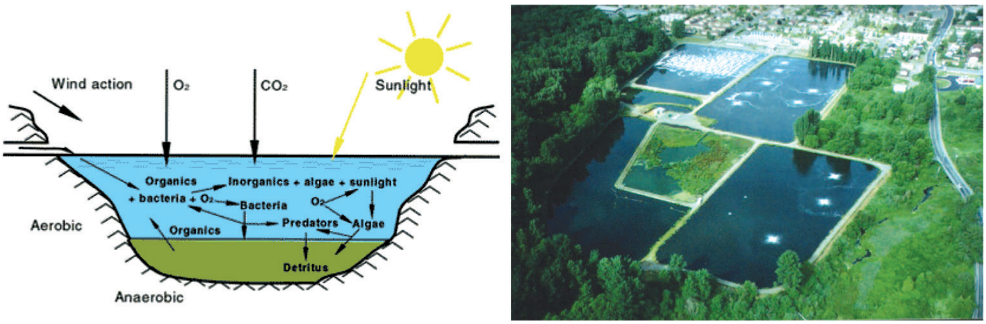

Fig. 47. Processes occurring in a facultative lagoon (left) and an aerial view of aerated and facultative lagoons (right) (Bioscience, Inc.)

4.5. SECONDARY SEDIMENTATION

Secondary sedimentation plays a very important role in the biological treatment of wastewater, especially when using activated sludge. In biological wastewater treatment processes, the contaminants contained in wastewater are transformed to a large extent into the growing biomass of microorganisms. This biomass must be **effectively separated** from the treated wastewater—for this purpose the secondary sedimentation process is most frequently used.[9] This process is slightly **different from primary sedimentation**, mainly due to the **higher concentration** of the suspended solids and the **distinctive characteristics** of the settling activated sludge flocs. During secondary sedimentation, two processes are to be taken into account: settling of activated sludge flocs occurring to a greater extent under **zone sedimentation**[10] conditions, and **thickening** of the settled sludge. During zone sedimentation the particles adhere to each other and settle along with neighbouring particles at a velocity that is dependent on the local concentration of the suspended solids.

The basic parameter that determines the course of this two-stage secondary sedimentation process is the **loading of the settling tank's surface with the suspended solids** expressed in kg TSS/m^2·d. This parameter can be determined as a function of the sedimentation properties of the sludge and of the recirculation rate of the return sludge by means of a solid flux analysis or state point analysis[11] and by experiments performed with the sludge in question. In a simplified design, the typical loading values available in the literature can also be used with some caution (Table 15). Another important parameter is the hydraulic load of the settler's surface, which is also significantly smaller than for primary settlers (see Table 10).

Table 15

Typical design parameters for settlers downstream of the activated sludge process (based on [11])

Type of treatment	Hydraulic surface load (m^3/[m^2·h])		Surface load of solids (kg TSS/[m^2·h])		Depth (m)
	average	**peak flow**	**average**	**peak flow**	
After activated sludge process	0.67–1.17	1.67–2.67	4–6	8	3.5–6
After activated sludge with BNR	0.67–1.17	1.67–2.67	5–8	9	3.5–6
After extended activated sludge process	0.33–0.67	1–1.33	1–5	7	3.5–6
After excess biological P removal:					3.5–6
– effluent P conc. = 2 gP/m^3	1–1.33				
– effluent P conc. = 1 gP/m^3	0.67–1				
– effluent P conc. = 0.2–0.5 gP/m^3	0.5–0.83				

[9] Membrane filtration is another increasingly popular method for separating solids from treated wastewater. The applications of Membrane Biological Reactors (MBR) are discussed in Section 4.2.5.

[10] Also called 'hindered sedimentation'.

[11] A description of these methods would be beyond the scope of this course, due to their complexity.

Fig. 46. Example of horizontal-flow circular secondary sedimentation tanks (FRIEDRICH KRUMME GmbH)

In practice, horizontal-flow circular and rectangular tanks are usually used for secondary clarification in large wastewater treatment plants and vertical-flow tanks are used in smaller ones. Although secondary settling tanks have a structure similar to primary settlers, they differ in their size and physical proportions, which results from the strongly flocculating properties of settling particles. For example, the physical proportions of the **horizontal-flow rectangular settlers** used for **secondary clarification** between length (L), breadth (B), and depth (H) should be [2]

$$\frac{L}{H} = \frac{9.6}{1} \div \frac{27.4}{1}; \ \frac{L}{B} = \frac{5.33}{1} \div \frac{27.4}{1}$$

and for **horizontal-flow circular settlers**, the proportions between diameter (D) and depth (H) should be

$$\frac{D}{H} = \frac{9}{1} \div \frac{12}{1}$$

Typically, the length of the rectangular tanks does not exceed 90 m and the breadth of an individual one is no more than 4 m. Circular tanks are constructed with a diameter ranging from 3 to 60 m, although they usually do not exceed 40–45 m [11].

The **detention time** in secondary settlers is about twice the detention time for primary settlers and ranges from approximately 2 hr to as long as 6 hr, depending on the sludge settling properties. The increased detention times directly determine the large volumes of the tanks. The **sludge hopper** in secondary settling tanks should be designed in such a way that it accommodates large volumes of sludge retained in the tank. In horizontal-flow tanks (rectangular and circular), the hopper should accommodate the activated sludge for approx. 3 hr. Sludge coming from biological filters can be kept in the settler's hopper much longer, because there is much less of it and it is not recirculated. In vertical settlers used in activated sludge systems, the sludge retention time in the settler's hopper should be about 0.5 hr, and in the case of settlers used downstream of biofilters, no more than 6 hr. The sludge hopper can be smaller when the sludge is removed from it continuously, or in short intervals [2].

5. SLUDGE TREATMENT AND UTILISATION

5.1. SLUDGE TREATMENT OBJECTIVES

The processing of sewage sludge generated during mechanical and biological treatment of wastewater is **closely integrated** with wastewater treatment processes. On the one hand, the volume and composition of sewage sludge are dependent on the wastewater treatment technology applied, and on the other hand, supernatants and process waters generated during thickening and dewatering of sewage sludge are recycled to the inflow to the treatment plant, **increasing the influent pollution load**. In addition, proper processing of sewage sludge allows the recovery of a significant amount of **energy** contained in wastewater in the form of biogas, which improves the overall energy balance of the treatment plant and reduces the emissions of pollutants. Therefore, the sludge processing system must be adapted to the wastewater treatment processes applied in the plant.

The sewage sludge treatment system in a municipal wastewater treatment plant has **two main objectives**: to **reduce the volume** of treated sludge and to **stabilise** them. The first of these goals is carried out in the **thickening** of sludge and the **dewatering** and **drying** of stabilised sediments. The relationship between sludge's volume (V_s), total mass (m_s), and water content (u) (or dry solids content) is as follows:

$$V_s = \frac{m_s}{\rho_s \left(100 - u\right)}$$

where
V_s – sludge volume (m^3),
m_s – sludge mass (kg),
ρ_s – sludge density (kg/m^3), and
u – water content (%).

During the thickening process the total mass of sludge remains almost constant and its water content decreases (e.g. from u_1 to u_2). Thus, according to the principle of the conservation of mass, its total volume is reduced (e.g. from V_{s1} to V_{s2}), which can be shown with the following formula:

$$V_{s1}\left(100 - u_1\right) = V_{s2}\left(100 - u_2\right)$$

The second goal is achieved by **aerobic digestion**, **anaerobic fermentation**, **composting,** or **chemical stabilisation** of the sludge. All these processes can be carried out using various methods and in different devices, depending on the volume and properties of the sludge being processed, and the technical capabilities of the treatment plant. Typical masses and properties of different types of raw (unprocessed) sewage sludge are presented in Table 16.

Table 16

Typical characteristics of different types of raw sewage sludge [9]

Type of sludge	Unit mass (g DS/[person·d])	Water content (%)	Unit volume (dm^3/[person·d])
Primary	54	91–95	1.08–5.4
Secondary after activated sludge	16–36	98.8–99.2	1.06–7.2
Secondary after biofilters	20–40	96–99	0.5–4.0
Secondary after RBC	20–40	97–99	0.67–4.0

5.2. SLUDGE PROCESSING SCHEMES

5.2.1. LARGE WASTEWATER TREATMENT PLANTS

At typical medium-sized and large wastewater treatment plants with primary sedimentation and activated sludge reactors (or biofilters), **gravitational thickeners** are used to thicken the primary sludge. The secondary sludge is thickened mechanically with the use of **decanting centrifuges** or **belt thickeners**. Additionally, polymers are added in the amount of 3–7 kg/1,000 kg of dry solids (Table 17). The thickened primary and secondary sludge are **mixed together** in a retention tank and directed to the process of **digestion**.

Table 17

Solids in different types of sewage sludge after processing [11]

Type of sludge	Solids concentration (% dry of solids)		Polymer dose (g/kg of dry solids)
	range	typical	
Primary from primary settling tank	4–10	6	1–4
Mixed sludge primary and secondary (activated sludge)	2–6	4	2–8
Mixed sludge primary and secondary (trickling filters)	4–10	5	2–8
Primary after gravitational thickening	5–10	8	-
Mixed after gravitational thickening	2–9	4.5	-
Secondary from secondary settling tank	0.5–1.5	0.8	4–10
Secondary after mechanical thickening	4–8	5	3–7
Mixed after anaerobic digestion	3–6	5	–
Mixed after aerobic digestion	4–8	6	–

Dewatered, mixed sludge after anaerobic digestion (centrifuge)	25–35	28	2–5
Dewatered, mixed sludge after anaerobic digestion (belt press)	15–30	22	3–8
Dewatered, mixed sludge after aerobic digestion (centrifuge)	12–15	14	-
Dewatered, mixed sludge after aerobic digestion (belt press)	15–23	18	2–8

In medium-sized and large plants (>10,000 p.e.) **anaerobic fermentation** is most often used for this purpose. Anaerobic digestion of the mixed sludge is carried on in **anaerobic digestion chambers** under **mesophilic** (30–38°C) or **thermophilic** (50–57°C) conditions. The anaerobic digestion can be a **single**- or **two-stage process**. The two-stage digestion process is carried out in two digestion chambers coupled in a series (separately, they are acid and methane fermentation stages), but the advantages of doing so are not obvious and it significantly reduces the capacity of the digestion system. In practice, **single-stage anaerobic digestion** is most often used; under mesophilic conditions it takes about **20–25 days** and under thermophilic conditions **14–16 days**. This time allows about **60%–65% of the volatile solids** present in the sludge to be degraded and transformed into biogas.

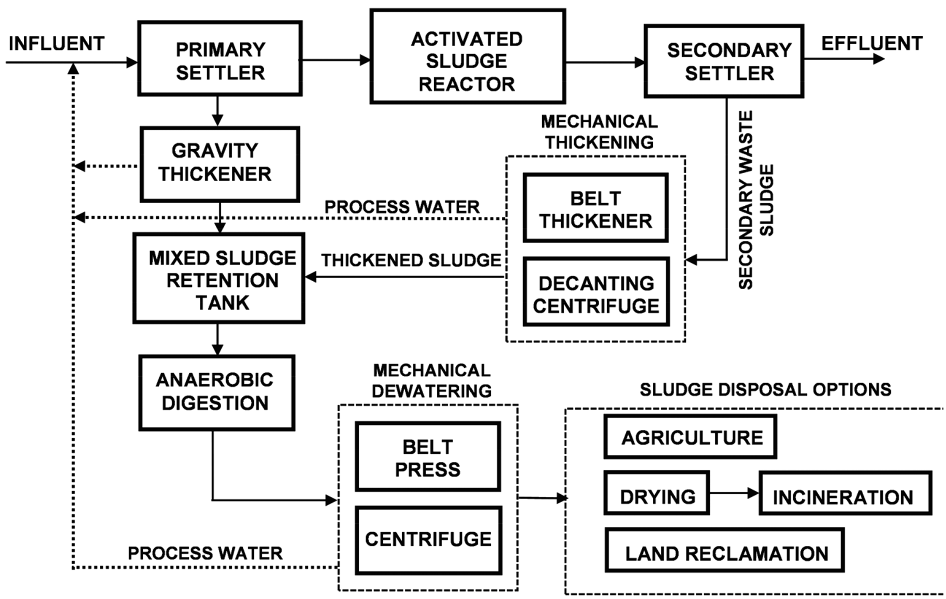

Fig. 49. Diagram of a typical sludge processing scheme at a large wastewater treatment plant with biological nutrient removal (BNR)

The content of anaerobic digestion chambers must be **continuously mixed** in order to increase the digestion process rate. There are several methods for mixing systems that utilise mechanical stirring, mechanical pumping of sludge, and biogas injection. They all have advantages and limitations. **Mechanical mixers** perform well but are susceptible to bearing and impeller wear and mechanical failures. The systems with **biogas injections** mix efficiently, but they accelerate the corrosion of pipes and can cause excessive foaming of the chamber contents. Systems with **pump mixing** are also susceptible to mechanical damage of the rotors and bearings, but are readily used due to the high mixing efficiency.

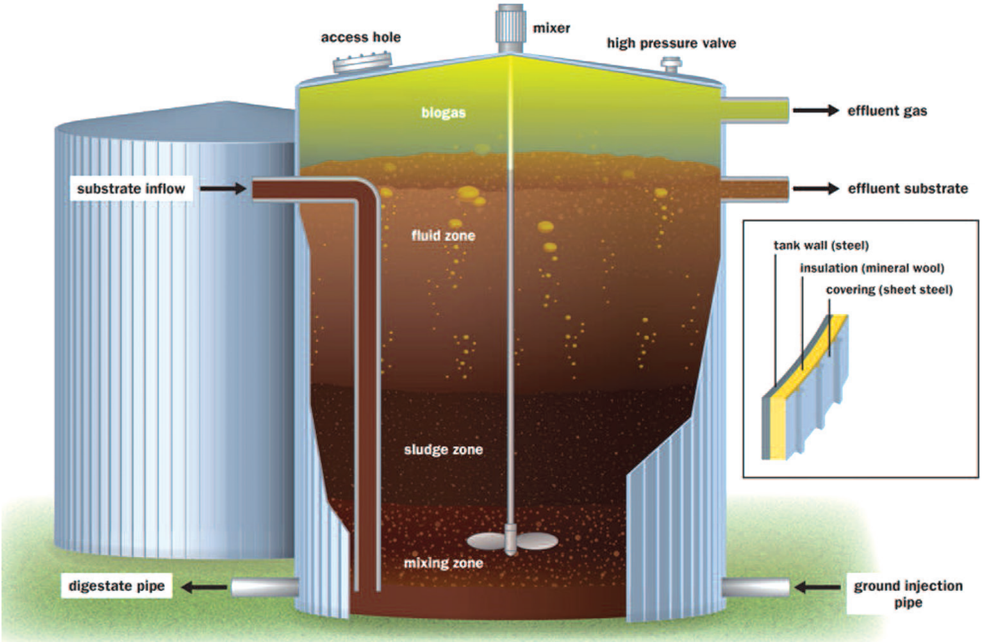

Fig. 50. Scheme of an anaerobic digestion tank (Triad Recycling and Energy)

Biogas produced during methane fermentation of sewage sludge accumulates in the upper part of the chamber, from where it is discharged for **desulphurisation** and is collected in the **gas tank**. Biogas contains about 65%–70% methane by volume, 25%–30% CO_2, and small amounts of N_2, H_2, H_2S, water vapour, and other gasses. Biogas yield depends on the reduction of volatile solids occurring in the digestion chamber. Usually, about 0.75–1.12 m^3/kg of volatile solids are degraded. Biogas production can also be estimated on a per capita basis. In municipal wastewater treatment plants the production yield is about 28 m^3 per 1,000 people per day [11]. Biogas is used as a fuel for heat and electricity production in **combined heat and power (CHP)** co-generation systems. Its heating value is about 22,400 kJ/m^3 (natural gas has 37,300 kJ/m^3). The hot water is used to heat the treatment plant itself and the sludge is pumped into the fermentation chamber. The electricity generated in CHP units is used to power the pump motors and is sold to the public energy grid.

A well-digested sludge contains little organic matter and is relatively susceptible to dewatering. The digested sludge is dewatered using **belt presses** or **centrifuges** after adding an adequate amount of polyelectrolyte (Table 17). With the correct choice of polyelectrolyte and its dosage, the centrifuges can reduce the water content in the sludge to 25%–35% dry solids. A slightly lower dry solid content in dewatered sludge can be obtained with the use of belt presses. Such a value allows for both the combustion of the sludge after pre-drying and agricultural use.

5.2.2. SMALL WASTEWATER TREATMENT PLANTS

In small wastewater treatment plants (<2,000 p.e.), the sewage sludge processing scheme is much simpler. However, in each case the sewage sludge must be thickened, stabilised, and then dewatered and disposed of. In small facilities, **sludge thickening**, both primary and secondary, is often carried out by **prolonged holding** of the sludge in the sludge hoppers of the settling tanks. Another option is the use of small devices for mechanically thickening the sludge, such as **disc thickeners**, **screw thickeners,** or **filter belts**.

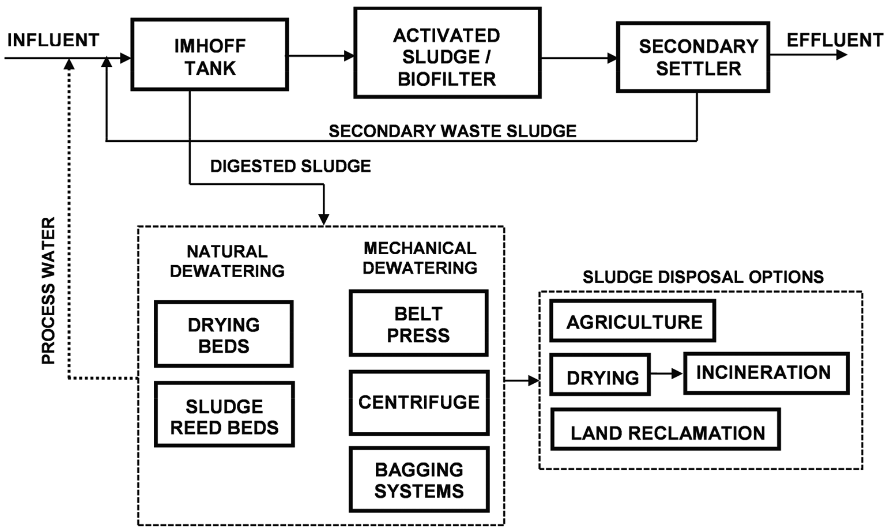

Fig. 51. Diagrams of a typical sludge processing scheme at a small wastewater treatment plant ith an Imhoff tank

Small wastewater treatment plants usually do not use anaerobic digestion to stabilise sludge as is done at large plants.[12] This is due to the relatively small mass of sewage sludge generated at smaller plants. Instead, the sludge is most often

[12] The only exceptions are wastewater treatment plants using Imhoff settling tanks for the primary treatment of wastewater. In such plants, both the primary sludge and the excess secondary sludge can be subjected to methane fermentation in the sludge chamber of an Imhoff tank. In a similar way, adequately large septic tanks can be used. See Section 3.3.2.5 for more details.

aerobically stabilised in biological reactors (**simultaneous stabilisation**) or in a separate aerobic stabilisation tank (**separate stabilisation**). Aerobic digestion can be applied to both secondary sludge and mixed (primary and secondary) sludge. During aerobic digestion, the sludge is intensively aerated for a period of time long enough for the **self-oxidation** of a majority of the microbial cells forming an activated sludge (75%–80%). This time depends on temperature, usually between 8 and 15 days. Simultaneous aerobic digestion is carried out in the common activated sludge reactor, in which the sludge (SRT) is held longer, about 25–30 days. After this time the amount of VSS is reduced by 38%–50%. If a separate aerobic digestion system is used, the thickened sludge is usually aerated in a separate aeration tank for a period of about 10–15 days. The process requires 0.8–0.9 kgO_2/(kg VSS·h), which translates into 0.9–1.8 m^3/hr of air needed per each m^3 of tank volume. Aerobic digestion is an **energy-intensive process** and, unlike for anaerobic digestion, **it is not possible to recover energy** in the form of biogas from the processed sewage sludge. A variation of the aerobic digestion process is **autothermal thermophilic aerobic digestion** (ATAD),[13] but it also requires intense aeration. Therefore, in many very small wastewater treatment plants, **chemical stabilisation**[14] of sewage sludge or **composting**[15] are used as cost-effective alternative solutions.

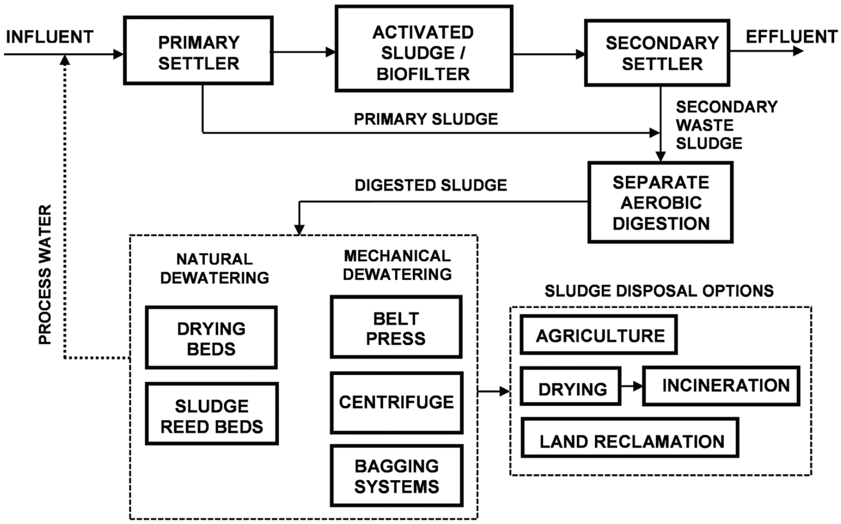

Fig. 52. Diagram of a typical sludge processing scheme at a small wastewater treatment plant with aerobic stabilisation

[13] The process is similar to aerobic digestion except that higher amounts of oxygen are added to accelerate the conversion of organic matter. The process operates at 40–80°C autothermally, in an insulated tank (Metcalf & Eddy, 2004).

[14] For example, by adding lime to sludge in a quantity sufficient to raise the pH to 12 or more. The process also inactivates viruses, bacteria, and other microorganisms. A typical dose is about 1.5 kg CaO/kg of dry solids.

[15] The process is usually carried out under aerobic conditions and approximately 20%–30% of VSSs are converted into CO_2 and water.

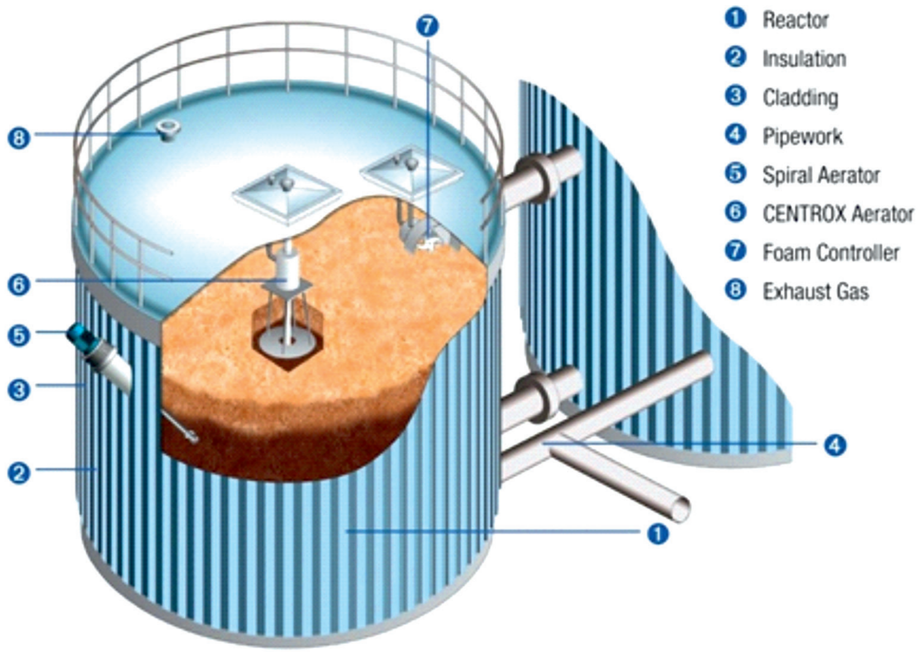

Fig. 53. Diagram of the Autothermal Thermophilic Aerobic Digestion (ATAD) system developed by FUCHS Enprotec GmbH

The stabilised sludge can be **dewatered mechanically** or dried in **sludge drying beds**. Mechanical sludge dewatering is carried out using methods and equipment similar to those used in large wastewater treatment plants: **centrifuges**, **belt-filter presses**, and different types of **bagging systems**[16] [13]. However, small amounts of stabilised sewage sludge are usually effectively dewatered on drying beds. After drying, the solids are removed and can be used in agriculture as a soil conditioner. **Drying beds** are made of several layers of slag or crushed stone with a total height of 25 cm and covered with a 10-cm layer of sand or with concrete plates. They are 5–20 m in width with central drains. Individual beds are enclosed with concrete walls of an appropriate height. They are filled with a stabilised semi-liquid sludge to a height of about 20 cm. Approximately every two months, dry sludge is removed and the beds are re-filled with new sludge. After drying, the sludge contains about 45% dry solids. In recent years the drying beds have been coupled with **solar systems** for sludge heating, thus enhancing the sludge drying process (solar drying beds). Sometimes, drying beds utilise different types of plants to enhance sludge drying. Such beds are called **sludge drying reed beds (SDRB)**.

[16] The bagging sludge dewatering system uses disposable and/or reusable bags made of a water-repellent material which are mounted on a stainless steel frame designed to provide even sludge distribution. The bags are filled with stabilised sludge under pressure. Once the cycle is over, the full bags are removed, sealed, and stored in the open air for further dehydration.

Fig. 54. Sludge drying beds at a small wastewater treatment plant (Ethics Infinity Pvt. Ltd.)

5.3. SLUDGE DISPOSAL AND UTILISATION

Sewage sludge after dewatering (and drying, optionally) is still a valuable material that can be used in a variety of ways. The method and principles of using sewage sludge are regulated by the relevant legal regulations. In the case of large wastewater treatment plants, it is recommended that the sewage sludge be **incinerated**. The stabilised sludge contains small amounts of volatile organic compounds and it is necessary to add an external fuel (e.g. oil, natural gas, or biogas) to initiate and maintain the incineration process. Non-stabilised and raw sludge has a much higher heating value—e.g. the heating value of raw sewage sludge is about 25,000 kJ/kg of dry solids, and for anaerobically digested sludge it is only 12,000 kJ/kg of dry solids. Moreover, the sludge subjected to incineration must be dried first because the evaporation of each kg of water requires an input of 4–5 MJ of energy [11]. In order to reduce the total costs of sewage sludge incineration, it is sometimes co-incinerated with municipal solid waste.

Another method for disposing of sewage sludge to use it in **agriculture** or for **land reclamation**. However, in order to qualify for agricultural and non-agricultural land application, the sludge must be hygienised and must meet standards for allowable pollutant concentration set in the appropriate regulations. At the EU level, this issue is regulated by the *Directive of 12 June 1986 on the protection of the environment, and in particular of the soil, when sewage sludge is used in agriculture (86/278/*

EEC). It states that sewage sludge shall be treated before being used in agriculture and shall be used in such a way that the nutrient needs of the plants is taken into consideration and that the quality of the soil and of the surface and ground water is not diminished. Moreover, it sets the limits for heavy metal concentration in sludge for use in agriculture (Table 18). All EU member countries must follow these restrictions and must implement them in their respective national regulations.

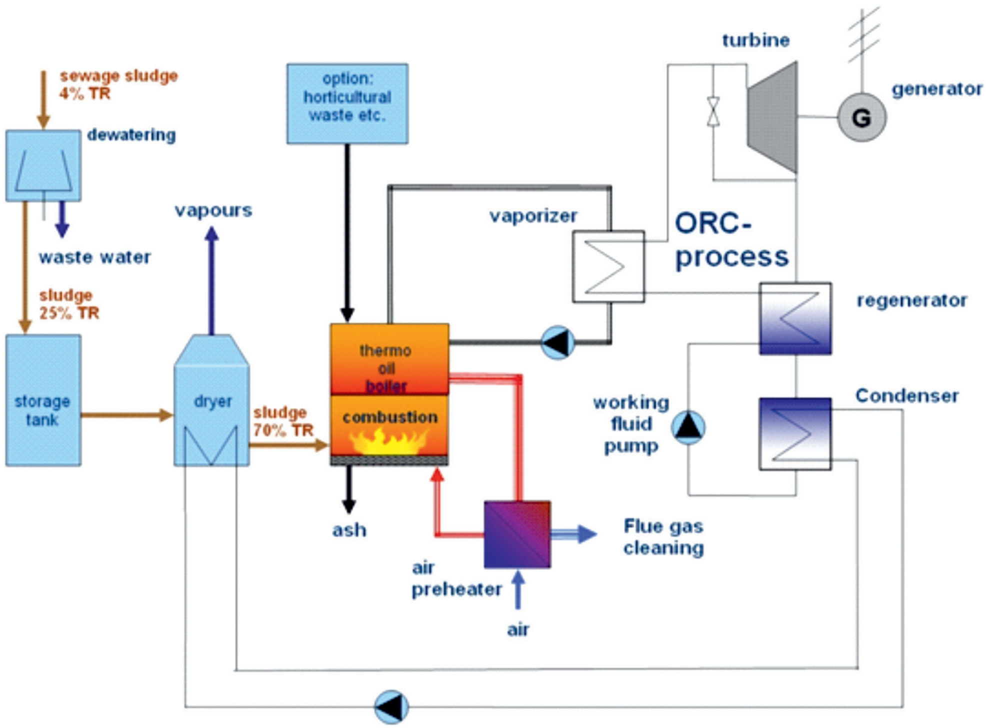

Fig. 55. Example sewage sludge incineration system developed by HUBER SE

Table 18

Limits for heavy metal concentrations in soil and sludge for use in agriculture and 10-year average load according to *Council Directive 86/278/EEC of 12 June 1986 on the protection of the environment, and in particular of the soil, when sewage sludge is used in agriculture*

Parameter	Limit, in soil (mg/kg dry solids)	Limit, in sludge (mg/kg dry solids)	10-year average load (kg/[ha·year])
Cadmium	1–3	20–40	0.15
Copper	50–140	1000–1750	12
Nickel	30–75	300–400	3
Lead	50–300	750–1200	15
Zinc	150–300	2500–4000	30
Mercury	1–1.5	16–25	0.1

6. CONCLUDING COMMENTS

The protection of freshwater resources should integrate a number of actions for increasing public education and environmental awareness, adjusting the legal and regulatory framework, and creating a technical and technological base for water protection. Inadequately treated effluents from municipal wastewater treatment plants may significantly contribute to chemical and microbiological pollution of freshwater resources and restrict its further uses. This cannot be constrained only to the question of the technological efficiency of treatment processes, but is part of a broader problem of the appropriate design and operation of a wastewater management system in the area. The selection of a centralised or decentralised wastewater management strategy must be made in the context of site-specific geographical, social, and economic conditions. Also, wastewater treatment technology should be chosen in consideration of these circumstances, and should be based on genuine technological knowledge and operational experience from similar installations. Even then, it should be remembered that while both wastewater management strategies have advantages and disadvantages, it is not the level of centralization or even a specific treatment technology that determines the overall efficiency of the system. It is more important is how well the system responds to local conditions, and the needs and expectations of the communities in the area.

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