

- Liston G. (1999). Interrelationships among snow distribution, snowmelt, and snow cover depletion: implications for atmospheric, hydrologic, and ecologic modelling. *J. Appl. Meteorol.*, **38**(10), 1474–1487.
- Mark O., Weesakul S., Apirumanekul C., Aroonnet S. and Djordjevic S. (2004). Potential and limitations of 1D modelling of urban flooding. *J. Hydrol.*, **299**(3–4), 284–299.
- Molteni F., Buizza R., Palmer T. and Petroliaigis T. (2006). The ECMWF ensemble prediction system: methodology and validation. *Quart. J. Roy. Meteorol. Soc.*, **122**(529), 73–119.
- NERC (1975). *Flood Studies Report* (in 5 volumes). Natural Environment Research Council, London.
- NRA (1993). *Guidelines for AMP(2). Periodic Review, Volume 2, Water Resources*. National Rivers Authority, London.
- Pomeroy R. and Parkhurst J. (1972). Self purification in sewers. In: Jenkins S. H. (ed.) *Proc 6th Joint Conf. on Advances in Water Pollution Research*, 18–23 June, Pergamon Press Oxford, Jerusalem.
- Saul A. and Ellis D. (1992). Sediment deposition in storage tanks. *Water Sci. Technol. WSTED 4*, **25**(8), 189–198.
- Vojinović Z., Bonillo B., Chitranjan K. and Price R. (2006). Modelling flow transitions at street junctions with 1D and 2D models. In: *7th International Conference on Hydroinformatics*, Acropolis, 4–8 September, Nice, France.
- Vojinović Z., Price R. K. and van den Broek W. (2005). Hydroplan-EU knowledge management framework for urban water asset management. In: *International Conference on Urban Drainage*, 21–26 August, Copenhagen, Denmark.
- Vojinović Z., Seyoum S. D., Mwalwaka J. M. and Price R. K. (2009). Effects of model schematization, geometry and parameter values on urban flood modelling. In: *8th International Conference on Urban Drainage Modelling*, 7–11 September, Tokyo, Japan.
- Wallingford (1981). *Design and Analysis of Urban Storm Drainage – The Wallingford Procedure*. Department of Environment, National Water Council, Standing Technical Committee Reports 28. Hydraulics Research Limited, Wallingford.
- Wallingford H. (1981). *The Wallingford Procedure for the Design of Urban Storm Drainage*. HR Wallingford, Wallingford.
- WaPUG (2002). *Code of Practice for the Hydraulic Modelling of Sewers*. CIWEM, London.
- WaPUG (2006). *Guide to the Quality Modelling of Sewer Systems*. WaPUG (CIWEM), London.
- White M., Johnson H. and Anderson G. (1997). *Control of Infiltration to Sewers*. R175: 72. London, CIRIA.
- WRc (1994). *The Sewerage Rehabilitation Manual*. WRc Engineering, Swindon, UK.
- WRc (1995). *Use of Detention Tanks for Flow Attenuation*. ER 171E. WRc Engineering, Swindon, UK.
- WRc (2004). *Sewers for Adoption*. WRc Engineering, Swindon, UK.
- WRc (2006). *Sewers for Adoption*, 6th edn. WRc Engineering, Swindon, UK.

## Chapter 11

# Wastewater Treatment

### 11.1 INTRODUCTION

It is estimated that more than 1.2 billion people have no access to safe drinking water and over 2.4 billion lack basic sanitation. The need for proper sanitation was made explicit in the United Nations Millennium Development Goals (UNESCO, 2009). Goal number 7 urges for the reduction by half of the population living without proper sanitation.

The protection of our clean water resources and the sustainability of our water systems as such, is one of the biggest challenges of the 21st century. Water is essential for most living organisms on this planet. However, the growth of the earth's population and industrialisation has increased the need for clean water enormously, putting a large pressure on our water resources. Apart from this, the environment is also exposed to a growing amount of (untreated) wastewater.

The European Union recognised this problem for the first time in the 1970s and introduced several water directives in its environmental policy. Among them are the EU Urban Wastewater Directive (CEC, 1991), and the EU Water Framework Directive (CEC, 2000), which has among its goals 'water management based on river basins' and 'getting the citizens more involved'.

On the other side of the Atlantic, the US Environmental Protection Agency (EPA) adopted the National Pollution Discharge Elimination System (NPDES) as part of the Clean Water Act (CWA) in 1972 (USEPA, 2004). In their latest regulation, the EPA requires states to develop prioritised lists of polluted or threatened water bodies and to establish the maximum amount of pollutant that a water body can receive while still meeting water quality standards, the so-called total maximum daily load (TMDL).

#### 11.1.1 Short history of wastewater treatment

Sewage has for a long time been considered a potential health risk and nuisance in urban agglomerations. The Ancient Greeks (300BC to 500AD) used public latrines which drained into sewers conveying the sewage and

stormwater to a collection basin outside the city. The Romans took the system further and constructed the Cloaca Maxima. The system worked well, but collapsed along with the Roman Empire. The period between 450 and 1750 AD is therefore known as the 'Sanitary Dark Ages'. During this period the main form of waste disposal was simply to dispose of it on the streets using buckets. Around 1800, collection systems appeared in many cities since people did not want to put up with the smell and farmers welcomed this 'humanure'. The collected sewage did not undergo treatment. Instead, it was spread out over the land as fertiliser. However, water-logging became a major problem, as well as finding sufficient land due to the expansion of the cities. At that stage, the idea of using organisms within the treatment process started to grow; biofilms growing on rocks in the river bed were applied as biofilters. The activated sludge process was discovered in the UK by Ardern and Lockett in 1914. At that time, rivers were considered as part of the treatment process. But at some point, the increased load of sanitary waste due to city growth could no longer be accommodated by the rivers, which meant that higher requirements were set for the wastewater treatment plants to reach better removal efficiencies. Moreover, the toxic effect of ammonia on aquatic species introduced awareness of nitrification. This process did not always work well, especially in winter. Nitrate was not considered as being undesirable at that time. This viewpoint changed in the second half of the 20th century when eutrophication became a severe issue; it led to an explosive growth of algae through the fertilising effect of nitrate. As a consequence there was further understanding of the processes and denitrification was introduced with particular bacteria using nitrate as an electron acceptor. This led to the development of pre- and post denitrification systems and the introduction of recycled flows. As well as nitrogen, phosphorus also poses a eutrophication problem. Unlike nitrogen, however, phosphorus needs to be removed by converting it to a solid phase. This resulted in the development of chemical precipitation followed by tertiary filtration in the 1970s. Later, biological phosphorus removal was discovered by accident. However, it led to the established EBPR system we know today. The energy crisis of the 1970s associated with an increased demand for industrial wastewater treatment, shifted attention from aerobic to anaerobic treatment, which was especially important for concentrated and warmer industrial influents. After a century of urban development and expansion, the problems generated by rapid urbanisation became apparent. This raised the urgent need for the development of a whole range of new treatment processes. Also the demands of the increasing effluent triggered the need for upgrading wastewater treatment plants as well as the treatment of sludge effluents containing high loads of nitrogen and phosphorus. This led to the development of SHARON-ANAMMOX and the BABE-process. The control of plants became yet another point of attention, and this was a driver for development of mathematical models. More recently, aspects such as control of sludge properties, disinfection and micropollutants have received growing attention.

Today's municipal wastewater treatment (WWTP) in developed countries is commonly achieved through the activated sludge process. This process consists of

- mineralisation and conversion of the biodegradable components by a broad variety of micro-organisms
- nutrient removal (nitrification, denitrification, phosphorus removal)
- separation of the grown activated sludge from the purified water through either gravitational settling or membrane filtration

The efficiency of the entire process depends on the efficiency of the processes separately. The first two biological steps have found considerable attention in research, which has resulted in a sufficient understanding of the various biological processes and conversions that take place. This knowledge has been translated into mathematical models which are able to describe this degradation process adequately (Henze *et al.*, 1987; Henze *et al.*, 1995a; Henze *et al.*, 2000a and Henze *et al.*, 2000b). As for the second, separation step, the processes involved are not yet fully understood. However, models have been developed for these as well, but are not as advanced and conclusive compared to the biochemical models.

In some cases, technologies other than activated sludge are used, mainly to save costs and due their easier implementation. These are often beneficial for developing countries. Note that some of these cheap

alternatives are not feasible in densely populated areas due to lack of space. The need for space is even sometimes problematic when upgrading existing facilities which have space limitations, resulting in having to use expensive solutions like membrane bioreactors. Examples of alternative treatment solutions are waste stabilisation ponds and reed beds. These are not discussed here.

This chapter has two major parts. The first focuses on wastewater characterisation and the different treatment steps. The second part discusses the most commonly used mathematical models for the different processes and the description of the wastewater treatment plant hydraulics.

## 11.2 WASTEWATER CHARACTERISATION

Wastewater can be characterised by both its **quantity** and **quality**. The former refers to its flow rate, whereas the second is concerned with its composition. Wastewater can originate from municipal use, i.e. households, or from industry, or even a mixture of both if industry is allowed to discharge untreated wastewater directly into the sewers. All these resulting wastewaters have different specifications with respect to flow rate and composition, which have implications for their treatment. This chapter focuses mainly on municipal wastewater.

### 11.2.1 Wastewater quantity or flow

The flow of wastewater coming to a wastewater treatment plant is typically collected in a sewer system (see Chapter 10). The origins of the wastewater are households, industry, and rain (whether indirectly through infiltration to sewers or directly through rainfall-runoff to combined sewers). The actual flow coming to the treatment plant is also influenced by exfiltration taking place in the sewer.

A very important feature of wastewater flow or hydraulic load is that it is highly dynamic, which poses some serious challenges for the design and operation of wastewater treatment plants. First, a diurnal pattern can be recognised because of the typical water consumption pattern of people (Figure 11.1). A first peak can be observed in the morning when they wake up and take a shower or bath. However, the hydraulic load drops when they go off to work. After returning home, the flow increases again giving an evening peak. Overnight, the flow drops again. The occurrence of the peaks in time can be shifted slightly depending on the length and structure of the sewer system. Moreover, the maximum peak flow can be as high as 5–10 times the minimum flow.

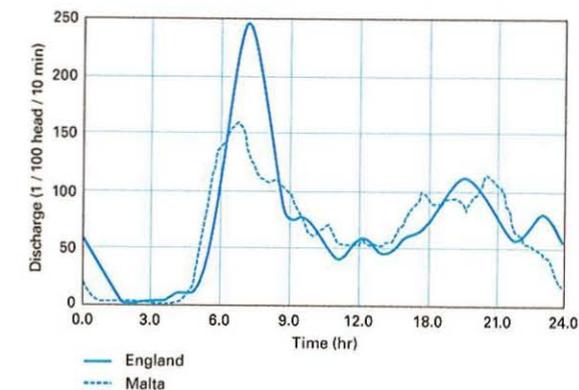


Figure 11.1 Examples of diurnal patterns of influent flows of different treatment plants (Henze *et al.*, 1995a)

Second, a differentiation is made between dry and wet weather flow (DWF vs WWF). This is strictly only applicable for combined sewer systems. WWF occurs when a rainfall event takes place in the catchment of the

sewer system. It results in peaks to the treatment plant as much as 6 to 10 times the average DWF (being the average of a diurnal dry weather day). This is illustrated in Figure 11.2.

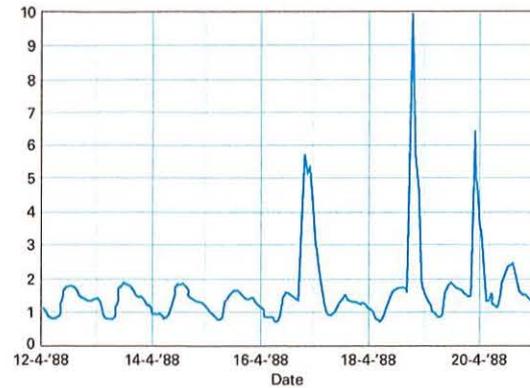


Figure 11.2 Examples of WWF diurnal patterns (Henze *et al.*, 1995a)

Sewer systems are designed in such a way that they can convey as much wastewater as possible to the treatment facility, but the maximum capacity is sometimes reached in combined sewers, leading to the need for combined sewer overflows (CSO). In order to limit discharges from CSOs to receiving waters, excess water is stored in retention tanks installed in the sewer system. The installation of such tanks increases the load to a treatment plant due to the need to empty the tanks as quickly as possible after a rain event to be ready for the next event. This may make a treatment plant subject to prolonged maximum hydraulic loads that would affect its performance. Finding the optimal compromise between the frequency of operation of combined sewer overflows and the treatment plant overload is the subject of considerable research (Vanrolleghem *et al.*, 2005).

In order to design a WWTP, it is important to have a good knowledge of the flow dynamics. Cumulative flow distribution diagrams (CFDD) can be used to summarise both diurnal and wet weather peak variations; see Figure 11.3. The CFDDs indicate the amount of time (expressed as % of days) that a certain flow rate is exceeded. They can be obtained from long-term historical measurements of sewer flows (for new plants) or from the WWTP influent flow (for plant extensions). As a rule of thumb, the 60percentile is used to represent the daily average flow, while the 85percentile serves as the maximum daily dry weather flow. The latter assumes that 15% of the days are actually wet weather days. These numbers can, of course, vary geographically.

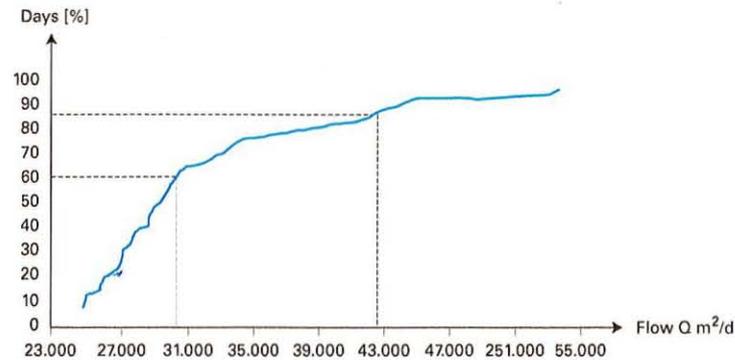


Figure 11.3 Example of a cumulative flow distribution diagram (Henze *et al.*, 1995a)

### 11.2.2 Wastewater constituents

An overview of different components that are likely to be present in wastewater along with their special interest and environmental effects is summarised in Table 11.1. This clearly illustrates why raw wastewater should not be discharged as such into receiving waters such as a river, lake or even coastal waters.

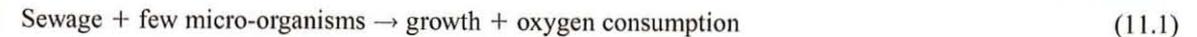
Table 11.1 Components in wastewater and their environmental effects (after Henze *et al.*, 2008)

Component	Of special interest	Environmental effect
Micro-organisms	Pathogenic bacteria, virus and worms eggs	Risk when bathing and eating shellfish
Biodegradable organic materials	Oxygen depletion in rivers, lakes and fjords	
Other organic materials	Detergents, pesticides, fat, oil and grease, colouring, solvents, phenol, cyanide	Toxic effect, aesthetic inconveniences, bio accumulation
Nutrients	Nitrogen, phosphorus, ammonia	Eutrophication, oxygen depletion, toxic effect
Metals	Hg, Pb, Cd, Cr, Cu, Ni	Toxic effect, bio accumulation
Other inorganic materials	Acids, e.g. hydrogen sulphide bases	Corrosion, toxic effect
Thermal effects	Hot water	Changing living conditions for flora and fauna
Odour (and taste)	Hydrogen sulphide	Aesthetic inconveniences, toxic effect
Radioactivity		Toxic effect, accumulation

In order to quantify the composition of wastewater, several determinants have been proposed. They can be grouped in terms of the type of pollution.

• **Organic pollution:**

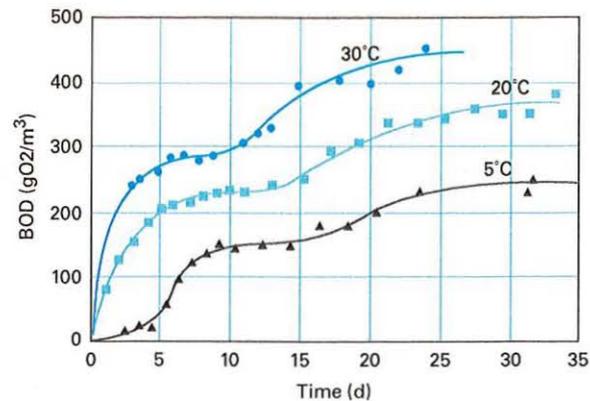
**Biochemical Oxygen Demand (BOD)** represents the amount of oxygen required by a small amount of micro-organisms to degrade a wastewater sample's organic pollution in a small (litre-scale) batch experiment equipped with a Dissolved Oxygen (DO) sensor according to:



BOD is measured over time, resulting in a cumulative curve that is highly temperature dependent; see Figure 11.4. BOD values can be determined after different time instants. An often used value is BOD<sub>5</sub>, which is the reading of the consumed DO after 5 days. 5 days was chosen to speed up the measurement, assuming that most of the readily biodegradable organic pollution is degraded. BOD<sub>7</sub> uses 7 days, which was regarded as the hydraulic residence time of most rivers to their mouth. If the total BOD (BOD<sub>t</sub>) is to be determined, then typically BOD<sub>28</sub> is used. In Figure 11.4, there is an intermediate plateau after which a further increase occurs (beyond the dashed lines) due to nitrification, which is brought about by slower growing organisms and, hence, occurs at a later time. This is often referred to as NBOD (nitrogenous BOD) as opposed to CBOD (carbonaceous BOD).

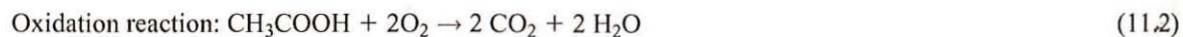
**Chemical Oxygen Demand (COD)** represents the amount of oxygen required to completely chemically oxidise the organic pollution in a wastewater sample into carbon dioxide, water, ammonium and sulphate.

The chemical reaction is conducted at a high temperature and in a very acid environment. As not all organic pollution is biodegradable (some part is inert), the COD of a wastewater is always somewhat higher than the BODt. A typical value observed for municipal wastewaters is 1.33.



**Figure 11.4** Illustration of typical BOD curves monitored at different temperatures. The oxygen consumption beyond the dashed lines represents the NBOD. Temperature significantly slows down the biodegradation processes

Unlike BOD, which always needs to be measured, COD can be calculated for specific components based on the elemental composition of the sewage components and the oxidation reaction. The former enables the computation of the molar mass of the component, whereas the latter yields the amount of oxygen molecules needed to perform the complete oxidation. Normalising to 1 g of a component yields the mass of oxygen that this 1 g of organic pollution represents. For acetate:



60 g of acetate ( $= 2 \times 12 + 2 \times 16 + 4 \times 1$ ) requires 2 molecules of oxygen or 64 g of oxygen. Hence, 1 g of acetate represents 64/60 g or 1.07 g of COD.

**Exercise: compute the amount of COD represented by ethanol ( $\text{CH}_3\text{CH}_2\text{OH}$ ), sewage ( $\text{C}_{18}\text{H}_{19}\text{O}_9\text{N}$ ), biomass ( $\text{C}_5\text{H}_7\text{O}_2\text{N}$ ), methane ( $\text{CH}_4$ ), sulphide ( $\text{H}_2\text{S}$ ).**

As can be seen above, COD can only be computed when the elemental composition is available. If this is not available then an actual measurement is needed. Since the original standard measurement is time consuming and rather dangerous (due to the harsh environment) and typically many COD measurements are required, fast test kits with high reliability are now commercially available.

COD has the advantage over BOD in that it enables the mass to be balanced (closed) in the system. This is very important when including organic pollution in a mathematical model where it the mass has to be balanced (conserved). This is developed further below.

**Total Organic Carbon (TOC)** aims at measuring the total amount of carbon atoms present in the organic pollution. The technique uses a catalytic reaction at high temperature. Here also the TOC can be readily calculated when the elemental composition is known. For acetate this yields: per mole of acetate, 2 moles of

carbon dioxide are produced. This means that 60 g of acetate produces 88 g of  $\text{CO}_2$  or 24 g of carbon (because 44 g of  $\text{CO}_2$  contains only 12 g of C;  $88 \times 12/44 = 24$ ).

Obviously, TOC is only a measure for the organics that contain carbon and cannot be used for other organic components. Also, the measurement is more difficult and not easy, though fast test kits exist.

**Exercise: compute the amount of TOC present in: ethanol ( $\text{CH}_3\text{CH}_2\text{OH}$ ), sewage ( $\text{C}_{18}\text{H}_{19}\text{O}_9\text{N}$ ), biomass ( $\text{C}_5\text{H}_7\text{O}_2\text{N}$ ), methane ( $\text{CH}_4$ ).**

**Total Suspended Solids (TSS)** determines the dry solids present in the sample after drying at 105°C for 24 h. TSS includes all suspended solids and also mineral salts.

**Suspended solids (SS)** determines the dry solids in a filtered (0.24 micron) sample after drying at 105°C for 24 h. The difference with TSS is that it does not include the mineral salts.

**Settleable solids** are determined from a wastewater sample that has been presettled for 2 hours, and are the difference between the SS and the settleable solids. They are discussed in the sludge-water separation step below.

- **Nitrogen content of wastewater:**

This form of pollution can be present as ammonium ( $\text{NH}_4$ ),  $\text{NO}_3$ . Organic nitrogen can be determined as total Kjeldahl-N or total-N. Standard methods are available for these measurements.

- **Phosphorous content of wastewater:**

This form can be present as ortho-phosphate. Organic phosphorus can be determined as Total-P. Again, standard methods are available for all of these measurements.

- **Heavy metals:**

Standard methods are also available for measuring Hg, Ag, Cd, Zn, Cu, Ni, Pb, As and Cr.

### 11.2.3 Wastewater composition

The wastewater from domestic use is often expressed in the unit Population Equivalent (PE), which can either be expressed in water volume or BOD. General worldwide accepted definitions are:

- 1PE = 0.2 m<sup>3</sup>/d
- 1PE = 60 g BOD/d

These two definitions are based on fixed non-changeable values. The actual wastewater contribution from a person living in a sewer catchment, the so-called Person Load (PL), can vary considerably as shown in Table 11.2. The reasons for the variation can be: working place outside the catchment, socio-economic factors, lifestyle, type of household installation, etc. Hence, values vary regionally. PE and PL are both based on average contributions, and are used to give an impression of the loading to the wastewater treatment processes that are needed to deal with them. They should be calculated on long term time intervals (months-years).

**Table 11.2** Variations in person load

Parameter	Unit	Range
COD	g/cap.d	25–200
BOD	g/cap.d	15–80
Nitrogen	g/cap.d	2–15
Phosphorus	g/cap.d	1–3
Wastewater	m <sup>3</sup> /cap.d	0.05–0.4

The concentrations found in wastewater are a combination of pollutant load and the amount of water with which the pollutant is mixed. The daily or yearly polluting load may thus form a good basis for an evaluation of the composition of wastewater. The composition varies significantly from one location to another. At a given location the composition varies with time. This is partly due to variations in the discharged amounts of the various pollutants. However, the main reasons are the variations in water consumption by households and infiltration and exfiltration during the transport of the wastewater in the sewage system.

The composition of typical domestic/municipal wastewater is shown in Table 11.3 where concentrated wastewater (high) represents cases with low water consumption and/or infiltration. Diluted wastewater (low) represents high water consumption and/or infiltration. Stormwater further dilutes the wastewater as most stormwater components have lower concentrations compared to very diluted wastewater.

**Table 11.3** Typical composition of raw municipal wastewater with minor contributions of industrial wastewater (in g/m<sup>3</sup>) (after Henze *et al.*, 2008)

Parameter	High	Medium	Low
COD total	1,200	750	500
COD soluble	480	300	200
COD suspended	720	450	300
BOD	560	350	230
VFA (as acetate)	80	30	10
N total	100	60	30
Ammonia-N	75	45	20
P total	25	15	6
Ortho-P	15	10	4
TSS	600	400	250
VSS	480	320	200

The fractionation of nitrogen and phosphorus in wastewater has an influence on the possible treatment options for the wastewater. Since most of the nutrients are soluble, they cannot be removed by solid-liquid separation (e.g. settling, filtration). The distribution between soluble and suspended matter is given in Table 11.4.

**Table 11.4** Distribution of soluble and suspended matter for medium concentrated municipal wastewater (in g/m<sup>3</sup>) (after Henze *et al.*, 2008)

Parameter	Soluble	Suspended	Total
COD	300	450	750
BOD	140	210	350
N total	50	10	60
P total	11	4	15

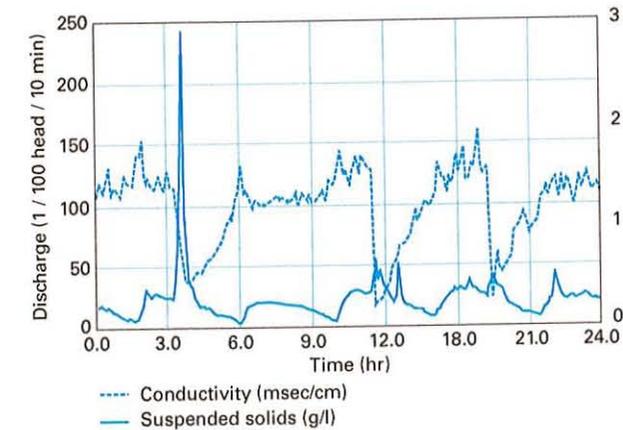
Many treatment processes are based on biodegradation. Hence, the degradable fraction is another important characteristic of wastewater. Typical values are given on Table 11.5.

**Table 11.5** Degradability of medium concentrated municipal wastewater (in g/m<sup>3</sup>) (after Henze *et al.*, 2008)

Parameter	Biodegradable	Inert	Total
COD total	570	180	750
COD soluble	270	30	300
COD particulate	300	150	450
BOD	350	0	350
N total	43	2	45
Organic N	13	2	15
P total	14.7	0.3	15

During WWF, two phenomena typically occur (in WWTP connected to combined sewer systems):

- **First flush:** This can be observed when the storm event occurs after a long period without rain, which has allowed the sedimentation of suspended solids in the sewer system. A first flush event results in a peak load of TSS to the plant; see Figure 11.5. This figure shows the same period as the second part of Figure 11.2. The peak brings about ten times the normal TSS load to the plant. This number is, however, dependent on the nature of the sewer system and the DWF period preceding the event: Figure 11.5 shows that a TSS peak does not occur during the second storm.



**Figure 11.5** Illustration of the effect of wet weather flow on conductivity (dilution) and suspended solids (first flush effect) (Henze *et al.*, 1995a)

- **Dilution:** A diluting effect occurs when there are large amounts of rainwater. Figure 11.5 illustrates this from conductivity measurements in a sewer. Unlike the first flush, dilution occurs with all storm events. Dilution is not beneficial for WWTP as the food to micro-organism ratio can be great reduced. One way to avoid this is to construct separate sewer systems for wastewater and rainwater. This is, however, very expensive, especially if a combined system is already in place.

### 11.3 TREATMENT STEPS

An overview of the different treatment steps is given in Figure 11.6.

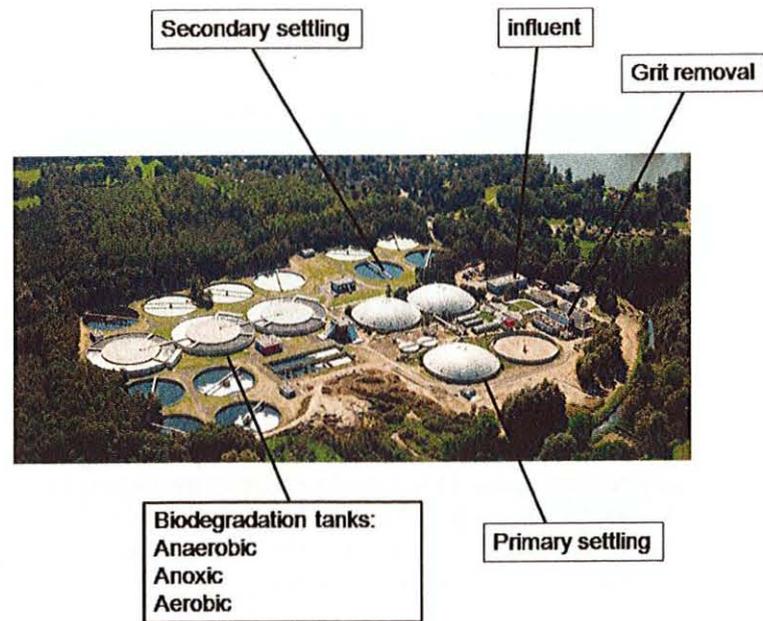


Figure 11.6 Illustration of the different treatment steps in biological wastewater treatment

### 11.3.1 Primary treatment

The primary treatment step consists of physical-chemical processes with the aim to remove:

- Grit and large material: this is achieved by filtration screens with certain pore sizes (typically 6mm followed by 1mm). This process is essential for membrane based systems to avoid clogging.
- Inert material: this material is not biologically degradable and accumulates in the reactors requiring considerable maintenance (i.e. costs). A typical example is sand. These units are referred to as sand traps and exploit the higher density of sand compared to other TSS components in wastewater.
- Difficult to degrade components: examples are oil and grease. Due to bad solubility and lower density, fat traps can be based on surface separation.
- Excess BOD: BOD is needed for nitrogen and phosphorus removal; see below. However, if the BOD load is too high compared to TN and TP (the indicative ratio is 100/10/1), it makes no sense to allow this excess BOD into the plant as it increases the aeration cost and sludge production. Therefore, this excess (that is, the slowly biodegradable BOD) is removed in primary settlers, which are sometimes preceded by a flocculation step to enhance the primary settling. The resulting 'primary' sludge is high in energy content and is typically sent to a sludge digester where energy can be recuperated in the form of biogas.

### 11.3.2 Secondary treatment

The secondary treatment step consists of two main steps: biological treatment processes and sludge-water separation. The latter produces water purified to a certain level that is either discharged to the receiving water

body or further polished in a tertiary treatment step where effluent limits are not met or if the water is intended for reuse (industry, agriculture) or drinking water production. The sludge enters the sludge treatment step, also called the 'sludge line'.

The biological treatment is performed by the activated sludge process. Activated sludge is a mixed culture of micro-organisms that typically occur in suspended flocs. An illustration is given in Figure 11.7. Different micro-organism species take care of different specific biochemical reactions. The energy gained from these reactions is used for growth and cell maintenance.

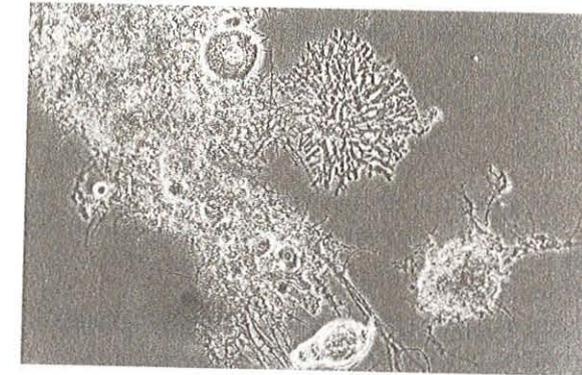
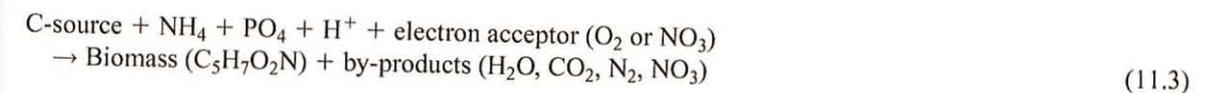


Figure 11.7 Microscopic photograph of an activated sludge floc

The growth of micro-organisms results in the multiplication of organisms. Next to an electron donor (C-source) and an electron acceptor ( $O_2$  or  $NO_3$ ), nutrients (N, P, S) and favourable environmental conditions are required. The basic reaction can be expressed in a simplified way as follows:



Since the biomass wants to grow, a number of compounds are converted:

- Organic pollutants  $\rightarrow$   $CO_2$  + biomass
- $\text{NH}_4 \rightarrow \text{NO}_3$
- $\text{PO}_4 \rightarrow$  Poly-P stored in biomass
- Organic pollutants (primary + secondary sludge)  $\rightarrow$  biogas ( $\text{CH}_4, \text{CO}_2$ )

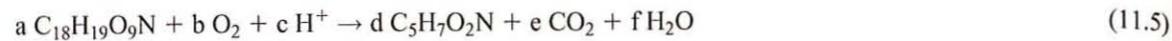
The compounds that are converted are those present in the wastewater as described above. The next important question that arises is how much of these components are converted. Process rates are typically described by the product of the *process kinetics* and the *process stoichiometry*. The former describes the rate of the conversion reaction, the latter describes the ratio of conversion of the different compounds.

Suppose the following reaction taking place (i.e. aerobic sewage degradation):



For each molecule of sewage degraded, a proportional amount of other products is used (all those components to the left of the arrow in Eq (11.4)) or produced (all components to the right of the arrow).

Adding this information can be done by including proportionality constants, yield or stoichiometric coefficients:



Typically, one of the coefficients is chosen to be equal to 1 and the others are normalised.

A reaction does not occur (i.e. the reaction rate is 0) when its sources or substrates (components to the left of the arrow in any process equation) are absent. On the other hand, when all components are available in excess, reaction rates are a maximum. The intermediate presence of components yields reaction rates between 0 and the maximum rate. This behaviour is typically described by the well-known Monod equation:

$$\mu(S) = \mu_{max} \frac{S}{K_S + S} \quad (11.6)$$

where  $\mu$  is the reaction rate (1/s)

$\mu_{max}$  is the maximum reaction rate (1/s)

$S$  is a substrate that the micro organisms can degrade for energy and growth

$K_S$  is the half saturation constant or affinity constant.

$K_S$  refers to that substrate concentration where the reaction rate equals half of the maximum reaction rate, i.e.  $\mu_{max}/2$ . This is illustrated in Figure 11.8 (left).

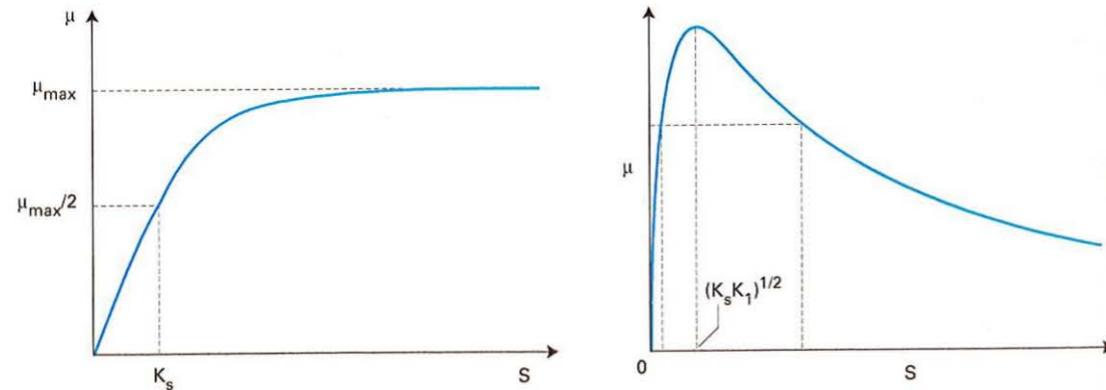


Figure 11.8 Illustration of Monod (left) and Andrews (right) kinetics

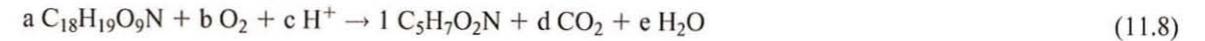
Sometimes substrates in excess can affect the maximum reaction rate negatively. In this case Andrews (or Haldane) kinetics can be used:

$$\mu(S) = \mu_{max} \frac{S}{K_S + S + S^2/K_I} \quad (11.7)$$

where  $K_I$  represents the inhibition constant.

The shape of this type of kinetics is shown in Figure 11.8 (right).

If the conversion rates for the different compounds are computed from the normalised reaction:



assuming Monod-type kinetics in the substrate and first order kinetics in the biomass concentration  $X$  (kg/m<sup>3</sup>)

$$\rho(S) = \mu_{max} \frac{S}{K_S + S} X \quad (11.9)$$

then:

$C_{18}H_{19}O_9N:$	$- a \rho(S)$	$C_5H_7O_2N:$	$+ 1 \rho(S)$
$O_2:$	$- b \rho(S)$	$CO_2:$	$+ d \rho(S)$
$H^+:$	$- c \rho(S)$	$H_2O:$	$+ e \rho(S)$

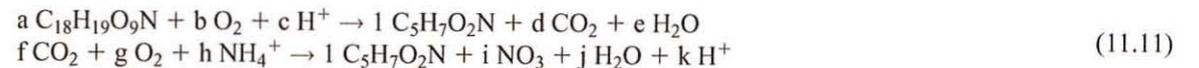
(11.10)

This means that the conversion rate of a component consists of 3 parts:

- a sign (+/-) dependent on whether it is used or produced
- a stoichiometric coefficient in the reaction
- a rate of the reaction  $\rho(S)$

When a compound appears in more than one reaction, the conversion rate takes one term per reaction.

Suppose the following reactions are taking place with respective reaction rates  $\rho_1(S)$  and  $\rho_2(S)$ :



The conversion rates of those compounds occurring in both reactions are then given by

$C_5H_7O_2N:$	$+ 1 \rho_1(S) + 1 \rho_2(S)$	$O_2:$	$- b \rho_1(S) - g \rho_2(S)$
$CO_2:$	$+ d \rho_1(S) - f \rho_2(S)$	$H^+:$	$- c \rho_1(S) + k \rho_2(S)$

(11.12)

The latter can be generalised for compound  $S_i$  taking part in  $j$  reactions:

$$r(S_i) = \sum_j \text{sign}(ji) v_{ji} \rho_j \quad (11.13)$$

where  $\rho_j$  is the rate of the  $j$ th reaction in which  $S_i$  participates (1/s)

$v_{ji}$  the stoichiometric coefficient for  $S_i$  in the  $j$ th reaction

$\text{sign}(ji)$  is the sign (-/+) indicating whether  $S_i$  is consumed or produced in the  $j$ th reaction.

As mentioned above, biological processes are impacted by the environmental conditions. Typical examples of influencing factors are temperature and pH. The former changes diurnally and seasonally, whereas the latter can be influenced by a spill in an industry, but can also be impacted by biological processes like (de)nitrification, consumption of fatty acids (e.g. acetic acid) and CO<sub>2</sub>-stripping through aeration. Since the impact on conversion rates can be rather large, they need to be accounted for.

After the measurement of the conversion rates at different temperatures in a large number of sludges, the solid black line along with a variability band in Figure 11.9 was found.

It can be clearly observed that in the lower temperature range the conversion rate increases with temperature. As a rule of thumb, the conversion rate doubles for an increase of 10°C. The exponential increasing behaviour is not valid anymore above 25°C. On the contrary, above 30°C, a decrease can be observed, due to the

denaturation of enzymes (note that this does not apply for mesophilic and thermophilic organisms present in digestion processes). However, most WWTP are operated in the exponential increasing region. Hence, reaction rates are typically corrected using an exponential equation:

$$\mu(T) = \mu(20) \cdot e^{k(T-20)} \quad (11.14)$$

Here  $\mu(20)$  is the conversion rate at 20°C (1/s)  
 $T$  is the current temperature (°C)

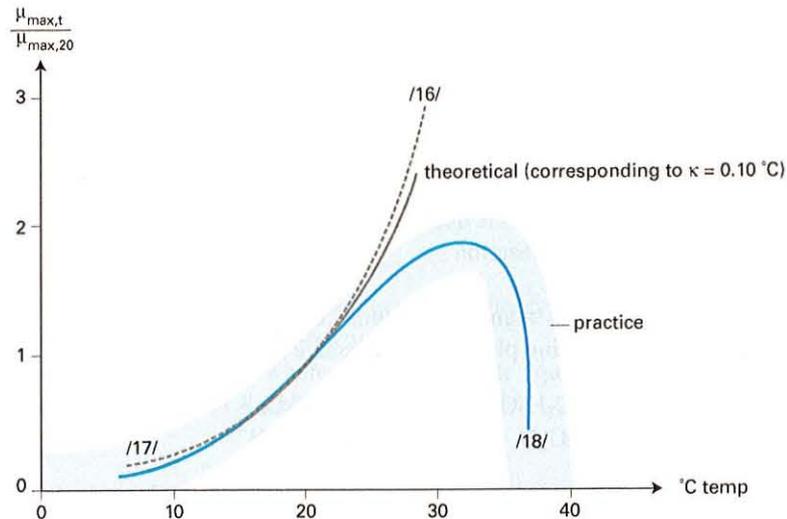


Figure 11.9 Illustration of the impact of temperature on maximum specific growth rate of micro-organisms

Knowing this temperature dependency, it can be deduced that the WWTP suffers when exposed to cold temperatures, i.e. in winter time in moderate and cold climates, when conversion rates are lower. This means that sludge residence times should be sufficiently high to avoid a sludge washout. This is especially true for slow growing organisms. As shown below nitrification, or the biological process responsible for  $\text{NH}_4$ -removal, is governed by slow growing organisms and, hence, it is vulnerable during winter. Temperature is, therefore, a parameter that must be accounted for during system design.

A similar vulnerability holds for pH variations. Micro-organisms have an optimal pH-range in which their maximum conversion rate is a maximum, roughly ranging from 6.5 to 8.5. Outside this range, a deterioration in the conversion rate occurs, as illustrated in Figure 11.10.

pH changes may occur due to processes that either produce or consume  $\text{H}^+$  or when pH shocks occur in the influent (e.g. due to an illegal discharge from industry). pH inhibition can be included in models through

$$\mu(\text{pH}) = \mu(\text{pH}_{opt}) \frac{K_{pH}}{K_{pH} - 1 + 10^{|pH - \text{pH}_{opt}|}} \quad (11.15)$$

It is often discarded when the sludge has enough buffer capacity (alkalinity) to avoid pH upsets. It is recommended to account for this phenomenon when modelling industrial WWTP.

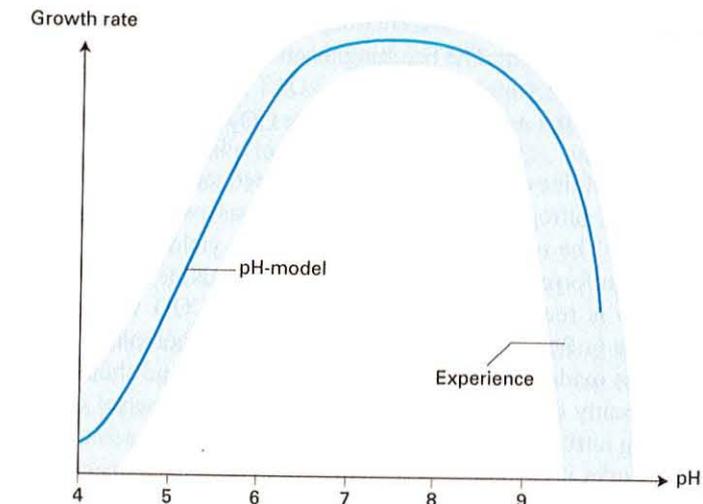
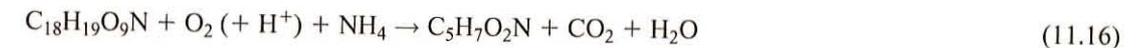


Figure 11.10 Illustration of the impact of pH on maximum specific growth rate of micro-organisms

Several specific **biological treatment processes** can be distinguished in an activated sludge system:

- **Aerobic organic substrate removal:** This process is conducted by a consortium of heterotrophic organisms, that is, organisms that use an organic C-source. Dissolved oxygen is required as electron acceptor resulting in

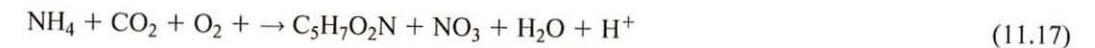


As can be seen, the biomass requires both sewage and nutrients for respiration and cell growth. The amount of substrate COD being built into biomass COD is referred to as the yield. Heterotrophic organisms exhibit a high yield of 0.4 g biomass COD/g substrate COD.

- **Biological nitrogen removal:**

An overview of the different biological nitrogen removal processes is shown in Figure 11.11.

- **Nitrification:** This process is conducted by a consortium of autotrophic organisms, that is, organisms that use an inorganic C-source such as  $\text{CO}_2$ . Dissolved oxygen is required as an electron acceptor resulting in



As can be seen,  $\text{NH}_4$  is oxidised to  $\text{NO}_3$  and the reaction has an acidifying effect. Autotrophic organisms have a lower yield: 0.24 g biomass COD/g N, and also exhibit a lower growth rate. The latter makes them vulnerable to process disturbances such as pH, T and toxic components. Therefore, nitrification is often the first process that is lost from an upset system. It is, however, highly unwanted given the toxic nature of  $\text{NH}_4$  when discharged to the surface water.

The process is actually a 2-step process ( $\text{NH}_4 \rightarrow \text{NO}_2 \rightarrow \text{NO}_3$ ) carried out by different organisms (AOB and NOB). When the first step is hampered, this can lead to a build up of  $\text{NO}_2$ , which is toxic to many biological processes in the activated sludge system. This should be avoided. Complete nitrification consists of steps 1 and 2.

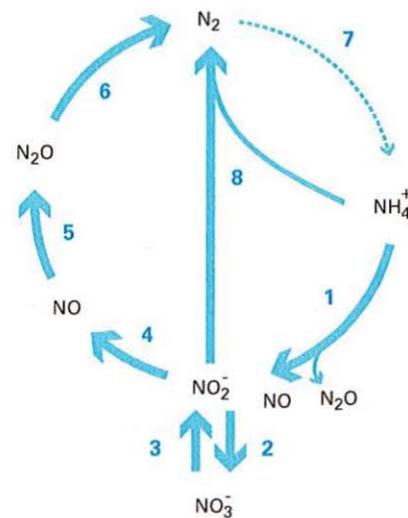
- **Denitrification:** This process is mostly conducted by heterotrophic organisms (DEN), although some steps can be performed by AOB organisms as well. Unlike aerobic organic carbon removal,

denitrification only occurs in an anoxic environment, that is, when dissolved oxygen is absent and  $\text{NO}_3$  is used as electron acceptor. The resulting reaction is



which indicates the usefulness of the reaction, i.e. the reduction of  $\text{NO}_3$  to  $\text{N}_2$  which volatilises from the system. Therefore, nitrogen is removed from the wastewater. Note that some portion of the N is built into biomass. The reaction has a relatively high yield of 0.3 g biomass COD / g substrate COD and actually performs both N and COD removal. Note that the dissolved oxygen used in the nitrification step is reused here for the oxidation of COD via  $\text{NO}_3$ . Complete denitrification consists of steps 3 to 6. Recent research has revealed that incomplete denitrification can lead to the production of nitrous oxide which is a strong greenhouse gas and should be avoided.

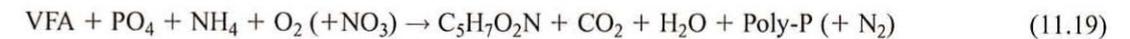
- **Anammox:** This recently discovered process is performed by special organisms that are capable of oxidising  $\text{NH}_4$  using nitrite. It is especially useful as it saves both aeration and carbon addition costs. However, it only works if no nitrate is formed, which requires a special environment to avoid step 2 in the scheme. This process is especially promising for high N-loaded wastewater streams (e.g. reject water from digester supernatant).



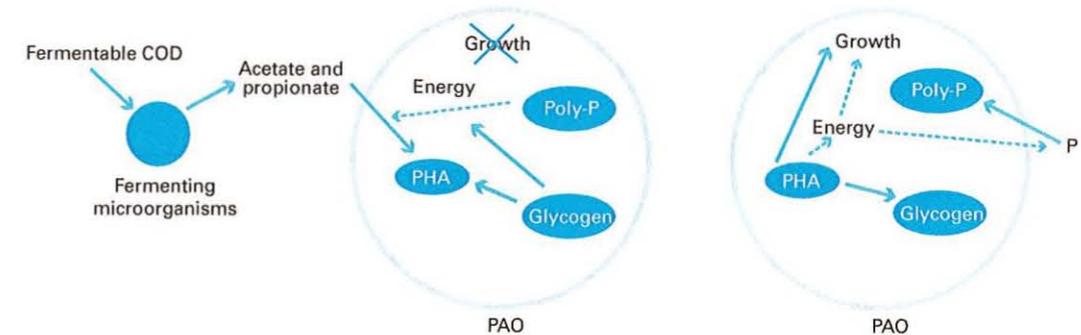
**Figure 11.11** Overview of the different processes involved in biological nitrogen removal (after Kampschreur *et al.*, 2009) (1) Aerobic ammonia oxidation (Ammonia Oxidizing Bacteria – AOB), (2) aerobic nitrite oxidation (Nitrite Oxidizing Bacteria – NOB), (3) nitrate reduction to nitrite (denitrifiers – DEN), (4) nitrite reduction to nitric oxide (AOB and DEN), (5) nitric oxide reduction to nitrous oxide (AOB and DEN), (6) nitrous oxide reduction to  $\text{N}_2$  (DEN), (7) nitrogen fixation (not relevant in most WWTPs), (8) ammonium oxidation with nitrite to  $\text{N}_2$  (Anammox)

- **Biological P-removal:** This process is also conducted by heterotrophic organisms (called Phosphorous Accumulating Organisms or PAO) and is rather complex as it requires a sequence of anaerobic (absence of  $\text{O}_2$  and  $\text{NO}_3$ ) and anoxic/aerobic conditions. The mechanism is illustrated in Figure 11.12. During anaerobic conditions, PAOs take up easily degradable organic substrate (volatile fatty acids or VFA) and store them in the form of poly hydroxyl alkanates (PHA) inside the cell. The energy required by the PAOs to take up the VFA is obtained by releasing phosphate from their internal poly-phosphate

pool. To convert the VFA into PHA reducing equivalents are needed that are provided by degradation of glycogen, another storage product that is essential for biological P-removal. During the next phase, i.e. during anoxic or aerobic conditions, PAOs oxidise the internal PHA pool and use the energy to grow, store phosphate as polyphosphate and regenerate the glycogen pool that they require in a next passage under anaerobic conditions. Thanks to the growth of the PAOs, more P is stored than was originally released, resulting in a net uptake of P from the wastewater. The net reaction is given by



Also, this process exhibits a fairly high yield of 0.3 g biomass COD/g substrate COD. As can be noted, the process performs N, P and COD removal. However, due to the complexity of the process, it is difficult to operate in a stable way in practice.



**Figure 11.12** Illustration of the behaviour of PAOs under anaerobic (left) and aerobic/anoxic (right) conditions (after Henze *et al.*, 2008)

- **Anaerobic digestion:** This process is part of the sludge treatment process. It takes settled sludges from both the primary and secondary settler as input. The process is also complex and takes place at an elevated temperature of  $35^\circ\text{C}$  where specialised organisms (acidogens, methanogens) exhibit their optimal growth conditions. In a first step large dissolved molecules and particles are extracellularly hydrolysed into small dissolved molecules by the acidogens, thereby producing acid molecules. In a second step, a second consortium of organisms, the methanogens, use the acids and hydrogen to produce biogas, i.e.  $\text{CH}_4$  and  $\text{CO}_2$ . The process can be summarised as



The organisms are slow growers, however, necessitating rather long sludge ages of about 20 days. Moreover,  $\text{H}_2$  is inhibitory and, hence, a delicate balance needs to be established to keep the process running. A more detailed overview of the 2 steps is illustrated in Figure 11.13.

The different required environmental conditions (anaerobic, anoxic, aerobic) make the plant configuration rather complex as the different conditions are established based on a spatial separation of the different processes.

A first typical configuration that was built merely for COD-removal and nitrification (if the sludge age is high enough) consisted of a single or sequence of aerated reactors (Figure 11.14a). This was common practice

in Europe back in the 1980s, when nutrient removal was not obligatory, and is still widely observed in the USA. However, in the 1990s European legislation with regard to nutrients became more stringent and existing plants were retrofitted to achieve nutrient removal as well, and new plants were built already containing nutrient removal. This led to a second typical configuration including either pre- or post-denitrification, requiring an additional anoxic tank either in front of or after the aerobic tank (Figure 11.14b) along with a recirculation flow. The latter only achieves N-removal, and, hence, to include bio P-removal, yet another configuration has to be used. The latter is often referred to the UCT-process as it was designed at the University of Cape Town (South Africa); it adds an anaerobic tank (Figure 11.14c). This also requires several recirculation flows.

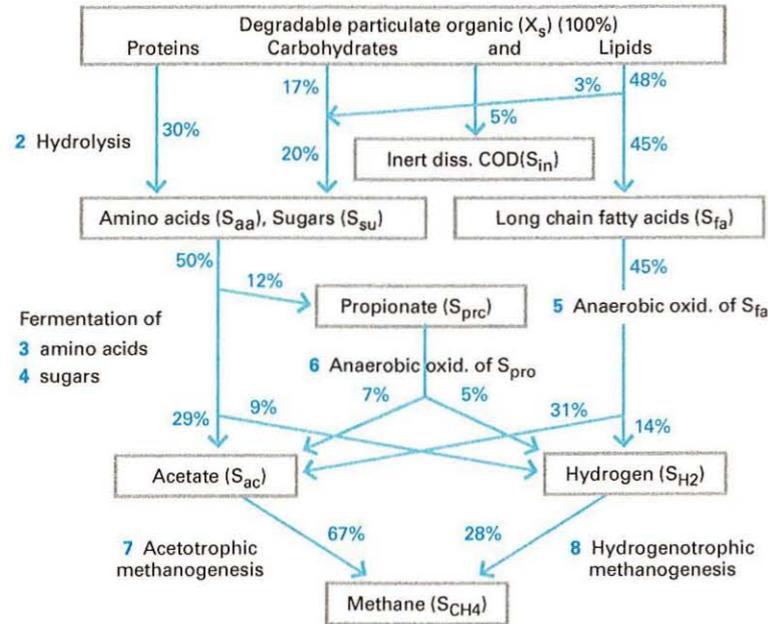


Figure 11.13 The different degradation steps in anaerobic digestion of sludge

Next to these 3 configurations, a number of modified configurations have been developed in the course of the last decades. They cannot be discussed here in detail and, therefore, the reader is referred to literature for more detailed information on the configurations (Tchobanoglous *et al.*, 2003 and Henze and Comeau, 2008).

To overcome some of the shortcomings of the conventional activated sludge process, including the sensitivity of some processes to sludge age and the spatially complex configurations needed, an alternative technology has been developed, namely, biofilm reactors. If the residence time of organisms in the process is smaller than the inverse of their growth rate, the organisms are washed out of the system. Organisms try to counteract this by forming either flocs that settle in the secondary clarifier or by growing as a biofilm attached to a surface. This allows them to stay sufficiently long in the system to survive.

An old technology that uses this biofilm principle is the trickling filter. It uses plastic carrier material that serves as surface for the biofilm to grow on. Water is then trickled over the carriers from the top, and oxygen is provided through natural ventilation. This has the advantage of being simple, performs reasonably and exhibits cheap aeration. The latter cuts costs, making this an appealing alternative for developing countries. Disadvantages of the technology are clogging and the attraction of flies that feed on the biomass.

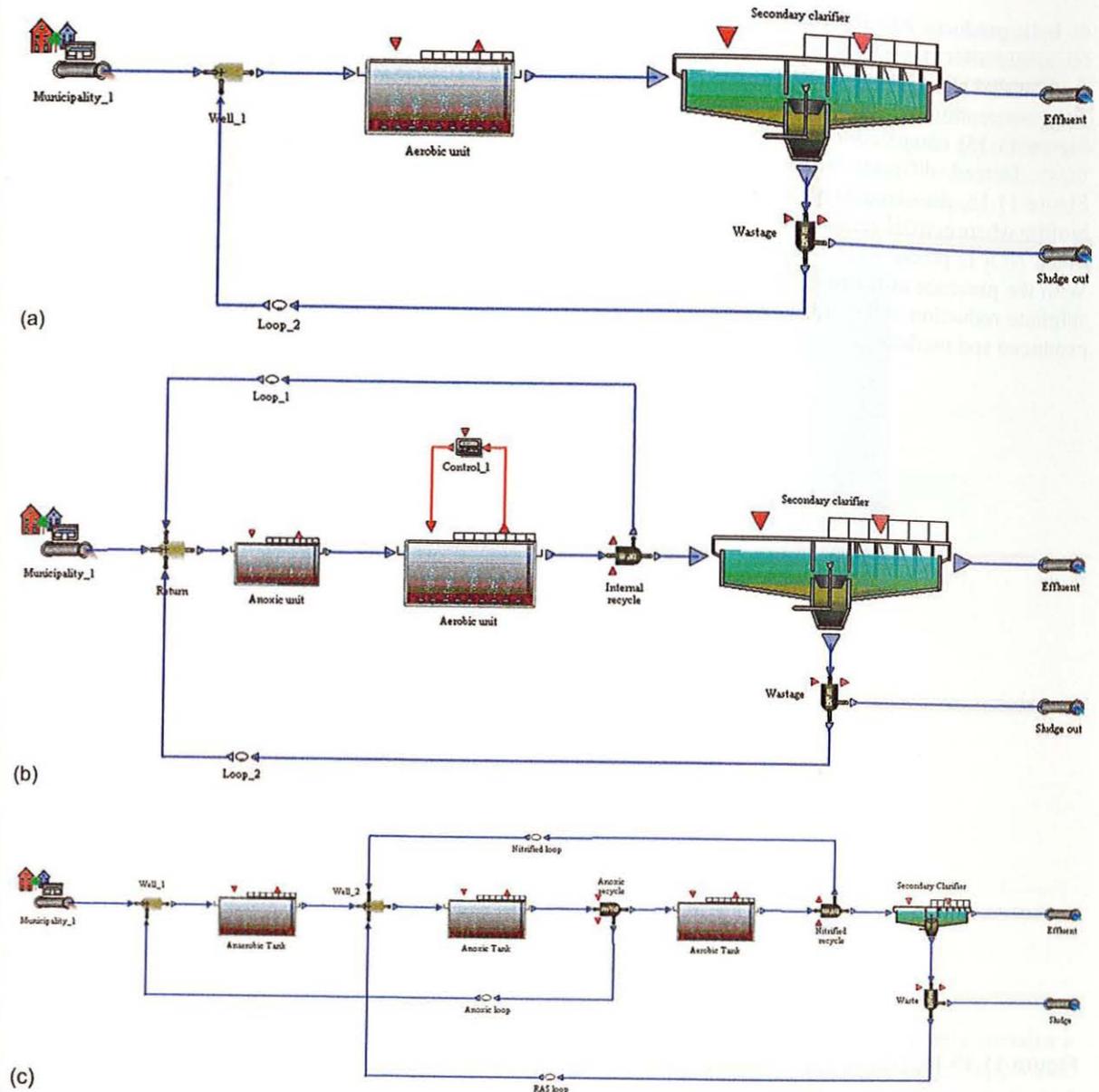


Figure 11.14 Illustration of different typically used configurations of activated sludge systems: (a) carbon removal and nitrification; (b) predenitrification system and (c) UCT process (screenshots courtesy of WEST®; www.mostforwater.com)

More recent processes based on biofilm technology are Moving Bed Bio-Reactors (MBBR), integrated Fixed-film Activated Sludge (IFAS) and (an)aerobic granule reactors.

Biofilm processes govern microbial conversion in a thin film of micro-organisms sitting on a carrier (in the case of granule reactors, the biomass acts as carrier itself). An important feature of biofilms is the diffusion

of bulk products into the biofilm, giving rise to gradients along the depth of the biofilm. These gradients obviously affect the actual processes taking place locally and their conversion rates. However, an advantage is the fact that also electron acceptors like dissolved oxygen and nitrate need to diffuse. This leads to zones with different conditions, allowing several of the previously described processes to take place in one location (see Figure 11.15) compared to having to construct separate tanks for all of them and use expensive recirculation flows. Instead, different layers in the biofilm perform different processes simultaneously. As indicated in Figure 11.15, dissolved oxygen only diffuses to a certain depth providing only a small aerobic top layer of the biofilm where aerobic carbon degradation and nitrification can take place (reference the bump in  $\text{NH}_4$  in this layer). Since  $\text{NO}_3$  is produced in this aerobic layer, it diffuses deeper into the biofilm, resulting in an anoxic zone. With the presence of readily biodegradable BOD,  $\text{NO}_3$  is depleted. This results in a third anaerobic zone where sulphate reduction and methane formation take place (note that in the outer aerobic and anoxic layers  $\text{SO}_4$  is produced and methane is oxidised).

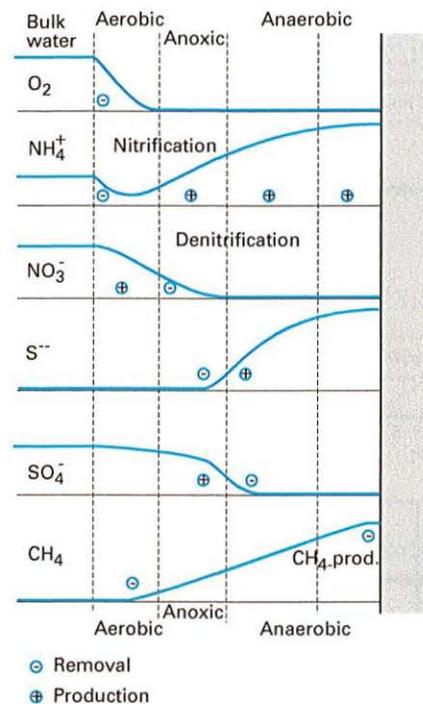


Figure 11.15 Illustration of the build-up of different zones due to diffusion limitations in a biofilm

Seemingly elegant from a theoretical point of view, making this biofilm technology work requires good control of the biofilm thickness which is a tedious task, influenced by both biological growth and physical forces (e.g. shear).

### Water-sludge separation

As a final step in the secondary treatment, the purified water and the sludge need to be separated. This can be achieved through either gravitational settling or membrane filtration.

So-called secondary settlers are used with **gravitational settling**. Figure 11.16 shows an example of a circular clarifier (rectangular clarifiers also exist, but the principle is the same). Sludge enters the clarifier in the centre energy dissipating well and then flows into the flocculation well where large flocs are formed that settle faster. The sludge then flows in radial direction towards the peripheral launder or weir. In the mean time, the sludge flocs can settle out and form a sludge blanket. The performance of the settling is dependant on the sludge settling properties. As the settling velocity is decreasing with increasing sludge concentration, the design sludge concentration in the biological tanks is actually limited to 3–4 g/l by the settling step.

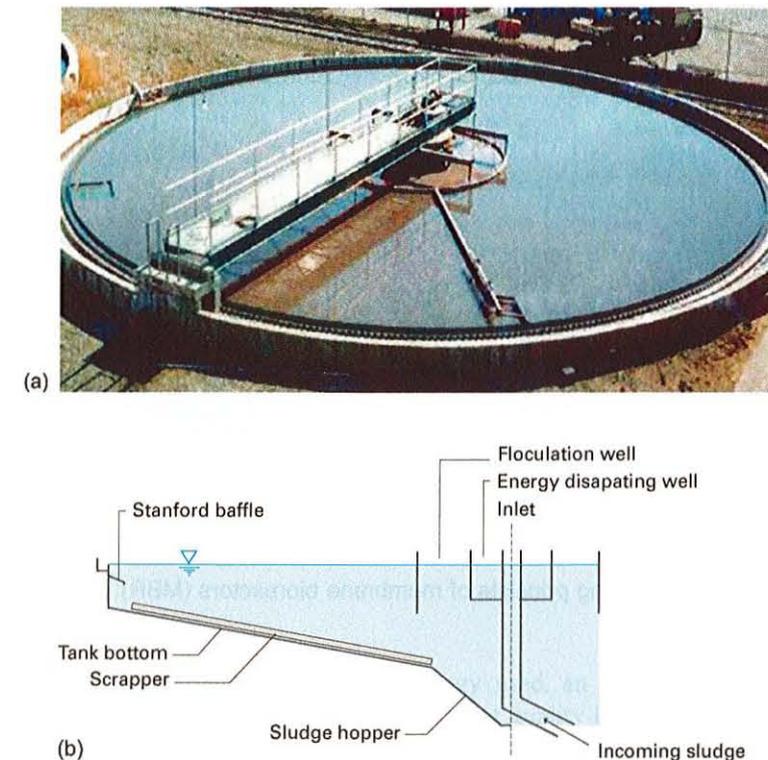


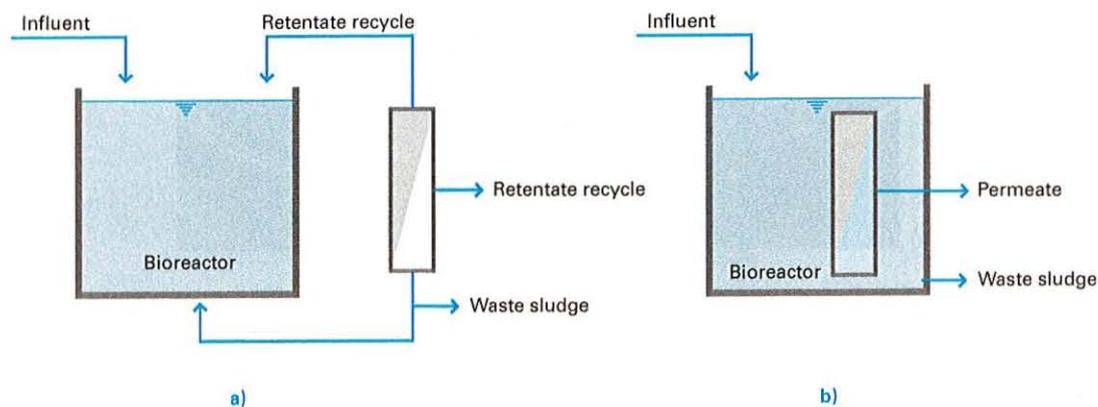
Figure 11.16 Illustration of a circular secondary clarifier (a) and an axisymmetric sideview (b)

The clarified water flows over the weir and forms the effluent of the treatment plant. It typically contains a non-settleable fraction which is in the order of 5–50 mg SS/l. The sludge moves towards the sludge hopper (the combination of the floor slope and a sludge scraper that is attached to the bridge that rotates constantly around the clarifier at a slow velocity). The collected sludge exhibits an elevated sludge concentration of about 8–10 g/l and is either directly disposed of or fed to the sludge treatment line.

More detailed information on the working principles of secondary clarifiers can be found in the IWA Scientific and Technical report (Ekama *et al.*, 1997).

An alternative technology that has been used in the last decade is membrane filtration in so-called membrane bioreactors (MBR). Instead of gravity, this uses a pressure drop over a membrane with pore sizes smaller than the size of a microbial cell (1  $\mu\text{m}$ ). Therefore, the process is often referred to as microfiltration. The process has some clear advantages: the effluent quality is superior (0 mg SS/l), biomass in the biological reactors can be operated

at higher concentrations, which also implies a smaller footprint as less volume is required. However, the process has the major disadvantage of the fouling of the membrane surface. The latter is cured using air scour and liquid movement. However, these make MBRs very energy consuming. Even though the unit price of membranes has dropped significantly over the last decade, the technology is still twice as expensive from an operational point of view (€1 vs €0.5/m<sup>3</sup> of treated water). This has kept the technology from conquering the wastewater treatment market. MBRs are designed in two configurations, either submerged or side-stream (Figure 11.17). The former is the most widely used as it does not require an additional cross-flow over the membrane unit. However, the sidestream configuration can be operated with significantly higher fluxes (50 vs 20 l/m<sup>2</sup>/h), which reduces the installed membrane surface to treat the same volume of water. Also, a variety of membrane products exist. Submerged systems either use flat sheet or hollow fibre membranes. Both operate in the outside-in mode. Sidestream MBR use tubular membranes. For more detailed information on MBR, see Judd, 2006 or Henze *et al.*, 2008.



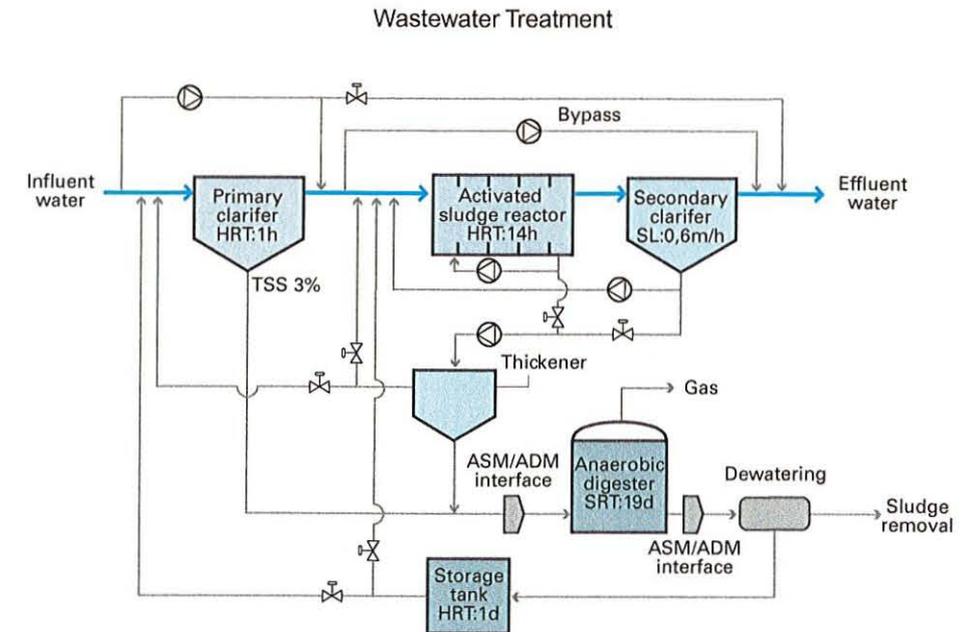
**Figure 11.17** Illustration of the working principle of membrane bioreactors (MBR): sidestream configuration (a) and submerged configuration (b)

### Sludge line

In the early days of the activated sludge process, the excess sludge produced was directly disposed of by incineration, landfill or as an agricultural fertiliser. However, due to stricter legislation, the options of disposing sludge have decreased drastically in Europe. The main options remaining are incineration or sludge digestion in which biogas and, hence, energy is recovered from this waste product.

The classic way of sludge treatment includes a sludge thickening step in order to increase the dry solids concentration drastically. This is then followed by a sludge dewatering step, which is typically brought about by using a coagulation step (using polymers) and a belt press or centrifuge. The removed water is recycled to the entrance of the biological tank.

A more recent practice of sludge treatment is to use digestion. The digestion step is included in between the thickening and dewatering step; see Figure 11.18. Since the digestion step releases both N and P into the liquid, the return flow from the digester to the entrance of the plant ('reject water stream') contains high loads of NH<sub>4</sub> and PO<sub>4</sub>. Either, this extra load should be accounted for at the plant design stage, or it should be reduced through an additional treatment step (e.g. Sharon-Anammox). Recently, efforts have also been made to precipitate the nutrients as struvite ((NH<sub>4</sub>)MgPO<sub>4</sub>·6H<sub>2</sub>O) which allows their recovery and separation. However, this requires additional crystallisation and separation steps.



**Figure 11.18** Illustration of the layout of the Benchmark Simulation Model no. 2 (BSM2), an example of a plant-wide model (after Nopens *et al.*, 2009)

A major difference in the dewatering effluent between the classic and digestion approach is the fact that nutrients are again released during the digestion process. Recycling them to the entrance of the plant results in an extra nutrient-load to the plant. This needs to be accounted for during design. Recently, efforts are being made in the field of nutrient recovery, by trying to precipitate the nutrients into struvite, which can be directly reused as fertiliser.

### 11.3.3 Tertiary treatment

Depending on the effluent requirements and the technology used, an additional treatment step might be required. MBR technology typically delivers a superior effluent quality (with all TSS removed), however, at a significantly higher cost. Other technologies might suffer from some remaining TSS that contributes to the effluent quality. Typical ways of dealing with this is to add a tertiary polishing step consisting of sand filtration (TSS removal and biofilm-based (de)nitrification), and micro- or nanofiltration. To remove micro-pollutants ozonation or the addition of Powdered Activated Carbon (PAC) may be adopted, and in some situations even reverse osmosis is applied to generate high purity water.

For more information on this matter the reader is referred to Tchobanoglous *et al.*, 2003.

## 11.4 MODELLING OF WASTEWATER TREATMENT PLANTS

In general, mathematical process modelling can be used for two purposes:

- Increase understanding of processes by testing certain hypotheses and confronting them with experimental data. This is referred to as 'model use for process analysis'
- Use well-established models for the design of the system and optimise its operation.

Hug *et al.* (2009) provide a more extensive overview of objectives for modelling, the type of modellers and the environment in which the modelling work takes place (Table 11.6).

**Table 11.6** Categories that structure the demands on modellers for different jobs with wastewater treatment models (after Hug *et al.*, 2009)

Objective of model work:	Type and level of model work:	Environment for model work:
<ul style="list-style-type: none"> <li>Academic research (as graduate student, research staff or faculty)</li> <li>Industrial research and development of processes and equipment</li> <li>Plant design and optimization</li> <li>Process control</li> <li>Model-based support for plant operation</li> <li>Model-based support for strategic and political decisions</li> <li>Communication of a complex or dynamic system behaviour</li> <li>Software development</li> <li>Approving modeling studies (e.g. as supervising agency)</li> <li>Teaching and training with different targets at different levels</li> </ul>	<ul style="list-style-type: none"> <li><i>Basic model users</i> (performing simple simulations without need for in-depth understanding, possibly advised by advanced model user)</li> <li><i>Advanced model users</i> (performing complex simulation studies requiring sound knowledge of possibilities and limitations)</li> <li><i>Model developers</i> (developing or adjusting model equations and structure)</li> <li><i>Software programmers</i> (implementing models into software, might need to deal with user interfaces or with numerical solutions)</li> <li><i>Instructors</i> (training one of the above listed groups of modellers)</li> </ul>	<ul style="list-style-type: none"> <li>Universities (learning, teaching, research)</li> <li>Consulting companies and equipment suppliers of different specialty and size</li> <li>Wastewater treatment plants</li> <li>Software developers and suppliers</li> <li>Supervising agencies</li> <li>Professional organizations</li> </ul>

In this chapter the focus is on the description of models established through system analysis. When modelling a WWTP, models have to be selected to describe:

- Hydraulics
- Mixing
- Biological processes
- Aeration
- Sludge-water separation: settling or filtration

For each process a variety of models exist ranging from very simple to very complex. The selection of the model complexity depends on the objective of the modelling study. Hence, the problem definition is the first issues that the modeller should resolve. The art of selecting the model complexity can be based on personal expertise or on dedicated model selection techniques. The latter is, however, beyond the scope of this chapter and can be found elsewhere (Dochain and Vanrolleghem, 2001). For further discussion on these issues see Chapter 6.

### 11.4.1 Modelling hydraulics

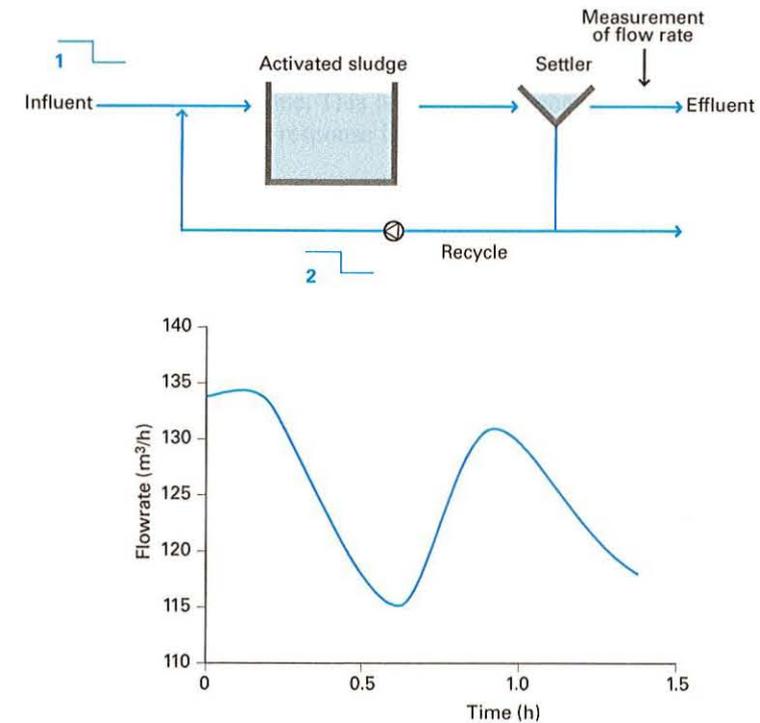
In Chapter 5 mass balance leads to an equation consisting of a transport term and a rate term. The former needs to reflect the hydraulic behaviour of the system. In a WWTP completely stirred tank reactors (CSTRs) are often used. The hydraulics may not be accounted for in detail, implying that incoming and outgoing flow rates are the same and the reactor volume is therefore constant. This is a valid assumption when steady state system behaviour is modelled. However, when one is interested in accurate dynamic behaviour, more detailed modelling is required to correctly capture the related phenomena. This is illustrated by Figure 11.19, which

is taken from de Clercq *et al.* (1999). Here, a step is imposed on the inflow at  $t = 0$  and an additional step in the recycled flow after  $t = 0.5$  h. The results show that a flow measurement at the back of the WWTP does not continue to contain steps, but rather a smooth transition. This cannot be modelled by a fixed volume tank because it would predict a step down for the decrease in inflow rate and not even notice the step down in the recycled flow. If this is to be accounted for, there is a need to use either a detailed hydraulic model (e.g. InfoWorks or CFD), or a variable volume approach where

$$\frac{dV}{dt} = Q_{out} - Q_{in}$$

$$Q_{out} = f(V) \quad \wedge \quad f(V) = \alpha(V - V_{min})^\beta \tag{11.21}$$

where  $V$  is the volume ( $m^3$ )  
 $V_{min}$  is the minimum volume ( $m^3$ )  
 $Q_{out}$  is the outflow ( $m^3/s$ )  
 $Q_{in}$  is the inflow ( $m^3/s$ )  
 $\beta$  is a constant



**Figure 11.19** Illustration of the measured and simulated dynamic response on a step down change in the influent flow rate (at  $t = 0$ ) and the recycle flow rate (at  $t = 0.5$  h) (after De Clercq *et al.*, 1999)

The actual outgoing flow rate is computed based on a power law depending on the degree with which the current volume  $V$  exceeds the minimum volume of the tank ( $V_{min}$ ). In the latter case,  $Q_{out}$  is zero. The parameter  $\beta$  is determined by the shape of the outflow works and takes values between 1 and 2. When applying

this model to the example in Figure 11.19, a good description of the measured flow rate at the outflow can be achieved. The variable tank approach is, however, not used frequently because it adds equations to the model and slows down the computations numerically. In some cases, such as the modelling of MBR, this approach is recommended in order to deal with hydraulic shocks while maintaining a constant flux. If the shocks are ignored, unrealistically high fluxes need to be allowed through the membrane.

### 11.4.2 Modelling of mixing

A proper description of the mixing in tanks can be done with the aid of the advection-dispersion equation:

$$\frac{\partial C(x, t)}{\partial t} + U(x, t) \frac{\partial C(x, t)}{\partial x} - D \frac{\partial^2 C(x, t)}{\partial x^2} = \rho \quad (11.22)$$

where  $C$  is the concentration ( $\text{g}/\text{m}^3$ )

$U$  is the velocity ( $\text{m}/\text{s}$ )

$\rho$  is the rate or conversion term (biological sources and sinks) ( $\text{g}/\text{m}^3/\text{s}$ )

As this partial differential equation (PDE) requires specific solvers that are computationally slow, another approach based on a discretisation of the space may be used. This results in the so-called tanks-in-series (TIS) model

$$\frac{dV_k C_k}{dt} = Q_k (C_{k-1} - C_k) + \rho_k \quad \text{with } k = 1, \dots, N \quad (11.23)$$

where  $N$  is the number of TIS

$V_k$  is the settling velocity ( $\text{m}/\text{s}$ )

Several approaches exist to determine the number of TIS to be used:

- Empirical approach:

$$N = \frac{7.4}{WH} L Q_{in} (1 + r) \quad (11.24)$$

where  $W, H, L$  respectively represent the dimension of the tank, i.e. width, height, length, (m)

$Q_{in}$  represents the incoming flow rate ( $\text{m}^3/\text{s}$ )

$r$  is the recycle ratio.

This is obviously a simple and cheap method of determining the number of TIS. However, the method is less accurate compared to more complex approaches. It can also be observed that the number changes drastically during wet weather flow when  $Q_{in}$  can increase up to 3 times.

- Tracer test: An alternative approach is to perform a so-called tracer study by injecting a pulse or step of an inert tracer at the reactor inlet. The inert nature of the tracer is important as the substance should not be adsorbed by the sludge and should not be degraded. The time series of recovered tracer is then measured at the outlet of the reactor under study. Models with different numbers of TIS can then be constructed and made to fit the observed data. This option is obviously more time-consuming and expensive, but is more accurate. Note that this also does not account for any wet weather flow, unless the tracer test is repeated with different inflow rate conditions.

Typical model-based responses to tracer injection are given in Levenspiel (1999). The pulse injection case is reproduced in Figure 11.20. It should be noted that any recycling should be switched off when performing a tracer test to allow for the simple interpretation of the tracer test data.

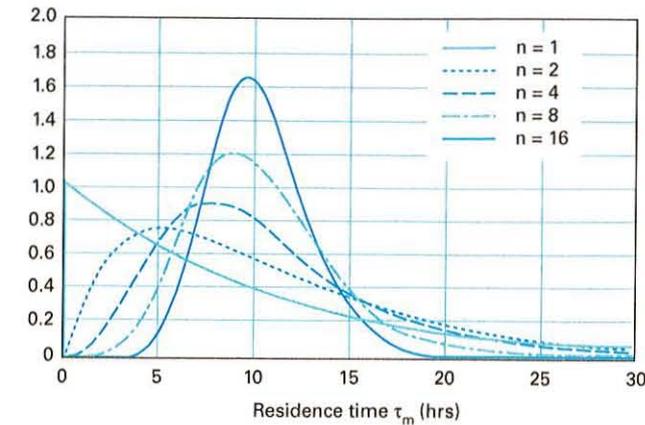


Figure 11.20 Illustration of the effect of a different number of CSTRs in series to describe the mixing behaviour; the y-axis represents the residence time distribution (RTD),  $\tau_m$  represents the dimensionless time based on the mean residence time in all  $N$  tanks

From Figure 11.20 it can be observed that using one tank gives rise to the typical exponential decay model response of a CSTR. Increasing  $N$  results in the build up of a peak, which becomes higher in absolute value and whose occurrence is shifted further in time. This is the typical behaviour of a complete plug-flow system. When  $N$  reaches infinity, a perfect plug flow response is approximated, that is, all the tracer appears at once at the outlet.

An example of a tracer test performed at full-scale is shown in Figure 11.21. The plant consists of 3 parallel lanes, where one lane itself consists of 2 parallel lanes. The question is: how to model this plant from a hydraulic point of view. A tracer test can be performed, where a pulse of inert tracer is introduced in the influent splitter (large arrow) and the recovered tracer is measured at the outlet of the 3 lanes (Erlenmeyers).

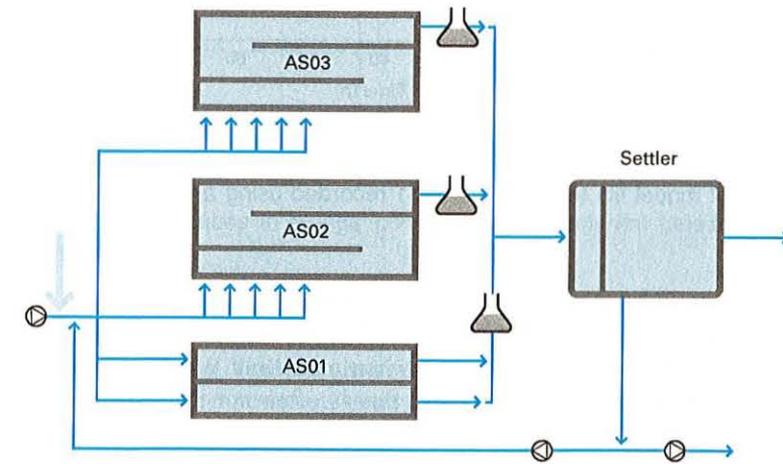


Figure 11.21 Illustration of the setup of a tracer study: point of tracer addition (large arrow) and sampling points (Erlenmeyers)

The results of the recovered tracer concentration versus the time of occurrence in lane AS01 with different TIS models are shown in Figure 11.22. It can be observed that using one TIS is not sufficient to model the mixing behaviour. However, when using two TIS with different volumes, the performance of the model is satisfactory. Similar results are obtained for the other 2 lanes. Note that different volume reactors are often required to fit the data.

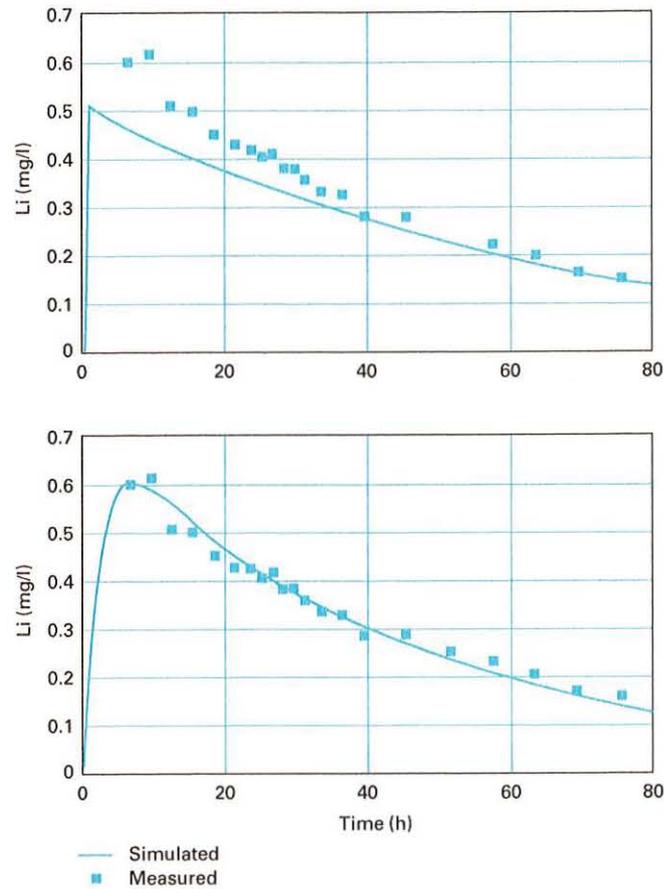


Figure 11.22 Illustration of model fits to RTDs in lane 1 recorded using a tracer test: using 1 CSTR (top) and 2 CSTRs in series (bottom)

The configuration that can then be used in a simulator platform is given in Figure 11.23. As can be seen, all lanes can be modelled using two TIS; a total of 8 CSTRs is needed to model the system.

Flow splitting works also deserve some attention when modelling WWTP. Splitting works are typically modelled as ideal flow splitters. However, in reality this is often not the case, and this can have a huge impact on the calibration part of a WWTP modelling exercise. One elegant way to investigate an influent splitting work is to investigate the sludge concentrations that occur in the different lanes. Theoretically, these should be the same when an equal loading is provided. The respective sludge concentrations in the different lanes for the previous example are given in Figure 11.24. In the case of an equal flow distribution,

a line through the origin with slope 45° should be found. It can clearly be observed that this is the case for lanes 1 and 3. However, lane 2 exhibits a much larger sludge concentration, suggesting that a higher sludge load is going to that lane. Closer investigation of the influent splitting works reveals shortcut flows of both the influent and the return activated sludge from the secondary settlers. Accounting for this results in the slightly modified model configuration (Figure 11.23 right) including a more detailed influent splitting works description.

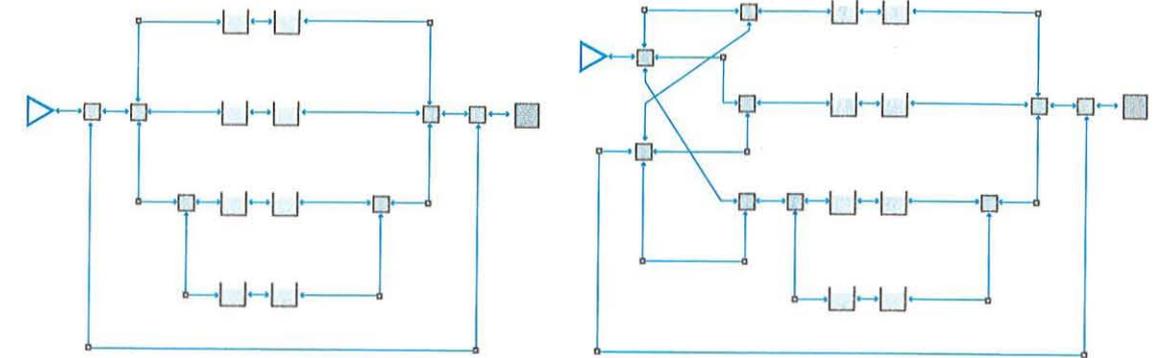


Figure 11.23 Illustration of how to account for flaws in a flow splitter (for explanation, see text)

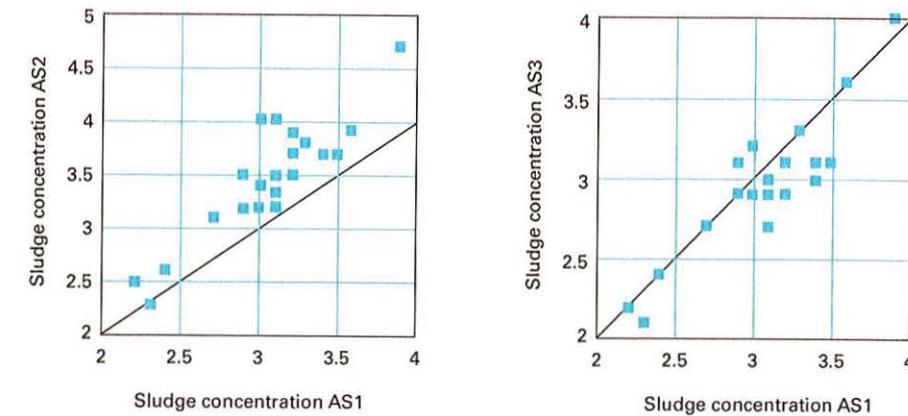


Figure 11.24 Illustration of differences in sludge concentration in different lanes caused by an improperly functioning flow splitting works

Another instructive example of a tracer test is shown in Figure 11.25 where samples are taken at different locations along a reactor. This clearly illustrates the ability of the TIS approach to model the mixing behaviour.

With regard to setting up a tracer experiment, some recommendations can be formulated:

- Choice of tracer: the tracer should be a non-biodegradable, non-adsorbing compound. Often used tracer materials are a Lithium salt and rhodamine.
- Injection of the tracer: the tracer should be injected as close as possible to the entrance of the reactor under study

- Data collection: samples should be taken as close as possible to the exit of the reactor under study. Samples should be collected at least during three times the hydraulic residence time. During this period about 20–50 samples should be taken.

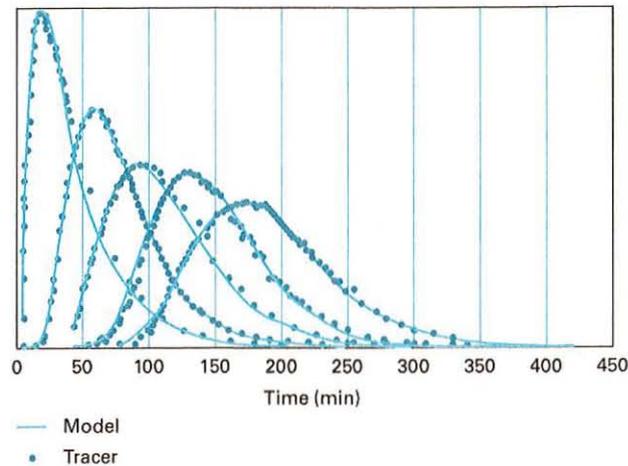


Figure 11.25 Conductivity measurements at different locations along a long reactor and a TIS-model describing the mixing properties of the reactor

### 11.4.3 Biological process modelling

Modelling of the biological processes in activated sludge started back in the 1970s and 1980s. The first milestone was the Activated Sludge Model no. 1 – ASM1 (Henze *et al.*, 1987). This was published as a Scientific and Technical Report from IAWQ and was innovative in respect to:

- nomenclature: all soluble components were addressed by a capital ‘S’, whereas particulate material carried the capital ‘X’. Recently, an extension of this standardized nomenclature has been presented (Corominas and Rieger, 2010)
- the goal of the model and, hence, its focus was on a good prediction of sludge production, oxygen consumption and nitrogen removal
- the COD based modelling: as mentioned above, when using COD, it is possible to close the mass balance, which is not possible for BOD, for example.
- the use of a Gujer matrix for the compact representation of a set of ordinary differential equations; an example of a Gujer matrix is given in Figure 11.26, which is composed of the following parts:
  - the left column lists all  $j$  processes accounted for in the model
  - the top row lists all the  $i$  different components taking part in the processes
  - the right most column lists the process rates for the respective processes in the left column
  - the core part of the matrix represents the stoichiometric coefficients.
  - the right bottom cell indicates the kinetic (occurring in the right column) and the stoichiometric (occurring in the matrix core cells) parameters
  - the bottom row indicates the units that the different components are expressed in.

From the Gujer matrix one can derive the conversion term of the mass balances for all components by summing all the products of the stoichiometric coefficient  $v_{ij}$  of process  $j$  and the respective process rate  $\rho_j$  (only the process in which the component  $i$  takes part).

Components $i$	1: $S_O$	2: $S_S$	3: $X_{Hl}$	Process rate equation $\tilde{\rho}_j$
List of processes $j$				
Aerobic growth	$\frac{1}{Y_H} + 1$	$-\frac{1}{Y_H}$	+1	$\mu_H^{max} \cdot \frac{S_S}{K_S + S_S} \cdot X_H$
Lysis		+1	-1	$b_H \cdot X_H$
Observed transformation rate $r_i$	$r_i = \sum_j v_{ij} \cdot \rho_j$ [ $M_i L^{-3} T^{-1}$ ]			
Definition of stoichiometric parameters:	Dissolved oxygen ( $O_2$ )	Dissolved organic substrate (COD)	Heterotrophic biomass (COD)	Definition of kinetic parameters:
$Y_H$ Heterotrophic yield coefficient [ $M_H M_S^{-1}$ ]				$\mu_H^{max}$ Maximum specific growth rate [ $T^{-1}$ ]
				$K_S$ Saturation coefficient for substrate [ $M_{COD} L^{-3}$ ]
				$b_H$ rate constant for decay [ $T^{-1}$ ]

Figure 11.26 Illustration of a Gujer matrix

From Figure 11.26, the following processes can be deduced to take place:

- Heterotrophic aerobic growth on readily biodegradable substrate ( $S_S$ )



where  $Y$  is the yield coefficient, i.e. the amount of biomass produced per unit substrate  $S_S$ . The equation is normalised to produce 1 mole of biomass

- Biomass decay



The equation expresses the amount of dissolved oxygen that is required for endogenous respiration (i.e. respiration in the absence of an exogenous substrate). Note that we do not need to know what the biomass decays into.

When all the compounds are expressed in consistent units, all horizontal sums of the stoichiometric rows should equal 0, which otherwise provides a verification of mass leaks. This is called the continuity check. It can easily be verified that continuity is maintained for both processes and, hence, the entire model (note that oxygen is expressed as negative COD).

Finally, the mass balances for the 3 components can be written down easily using the Petersen matrix:

- Biomass  $X_B$

$$\frac{dX_B}{dt} = \frac{Q}{V} (X_{B,in} - X_B) + 1 \frac{\tilde{\mu} S_S}{K_S + S_S} X_B - 1 b X_B \quad (11.27)$$

where  $\mu$  is the growth rate (1/s)

The first term on the right hand side is the transport term (not derivable from the Petersen matrix). Term 2 and 3 are the respective contributions of the processes 1 and 2 to the conversion rate of  $X_B$ . The stoichiometric coefficients are shown explicitly for clarity.

- Readily degradable substrate  $S_S$

$$\frac{dS_S}{dt} = \frac{Q}{V}(S_{S,in} - S_S) - \frac{1}{Y} \frac{\mu S_S}{K_S + S_S} X_B \quad (11.28)$$

- There is only one process consuming  $S_S$ .
- Dissolved oxygen  $S_O$

$$\frac{dS_O}{dt} = \frac{Q}{V}(S_{O,in} - S_O) - \frac{1-Y}{Y} \frac{\mu S_S}{K_S + S_S} X_B - lbX_B \quad (11.29)$$

Two processes are consuming dissolved oxygen. As  $S_{O,in}$  is typically zero, a term for aeration should be added. This process is, however, modelled separately and not included in the Gujer Matrix.

Aeration can be modelled in different ways. An often used model is one based on the oxygen transfer coefficient  $k_L a$ . This coefficient is the product of  $k_L$ , the oxygen mass transfer coefficient from gas to water, and  $a$ , the surface area of the gas-water interface. It is therefore a measure for the amount of  $O_2$  that is transferred from the gas phase to the liquid phase, where it becomes available for the biomass. Different aeration systems are used. Surface aerators bring the liquid in close contact with the headspace air (e.g. by splashing droplets into the air). Fine bubble diffusers are installed at the bottom of a tank and blow in compressed air as bubbles. For both systems the following reaeration model can be used (and is added as a term in the  $S_O$  mass balance of the ASM type models) expressed as Specific Oxygen Transfer Rate ( $SOTR - g O_2/l/d$ ):

$$SOTR = k_L a (S_{O,sat} - S_O) \quad (11.30)$$

The other parameter used in this model,  $S_{O,sat}$  is the saturated dissolved oxygen concentration in water. The reaeration rate is therefore dependent on the driving force being the difference between the actual  $S_O$  and the saturated dissolved oxygen concentration and the oxygen mass transfer. The saturated dissolved oxygen concentration is heavily dependent on temperature, whereas  $k_L a$  depends on sludge concentration.

The different COD components considered in ASM1 are given in Figure 11.27.

As can be observed, the total COD is broken down into 3 main fractions: biodegradable, non-biodegradable and active biomass. The former two fractions are further split into soluble and particulate fractions yielding

- soluble (readily) biodegradable COD:  $S_S$
- particulate (slowly) biodegradable COD:  $X_S$
- soluble inert COD:  $S_I$
- particulate inert COD:  $X_I$

The biomass COD is split up into a heterotrophic ( $X_{BH}$  or  $X_H$ ) and an autotrophic portion ( $X_{BA}$  or  $X_A$ ), both governing specific processes as explained above.

The flow of COD in ASM1 is shown in Figure 11.28.

As can be observed, a readily biodegradable substrate  $S_S$  is used as a substrate in the aerobic growth of the heterotrophic biomass  $X_H$ . The latter decays and produces a fraction that is slowly biodegradable ( $X_S$ ) and a fraction that is inert, which accumulates in the system.  $X_S$ , originating from biomass decay and also present in the influent can be hydrolysed back to  $S_S$ . Hydrolysis is the process where biomass produces enzymes to break down large molecules that are too large for them to take up through their cell membrane

into small molecules that the membrane can accommodate. In the top group of processes in Figure 11.28 are the processes governing the nitrogen removal. Ammonium ( $S_{NH}$ ) is used along with dissolved oxygen ( $S_O$ ) to produce autotrophic biomass ( $X_A$ ) and nitrate ( $S_{NO}$ ).  $X_A$  further decays partially into particulate inerts ( $X_I$ ) and particulate biodegradable COD ( $X_S$ ).

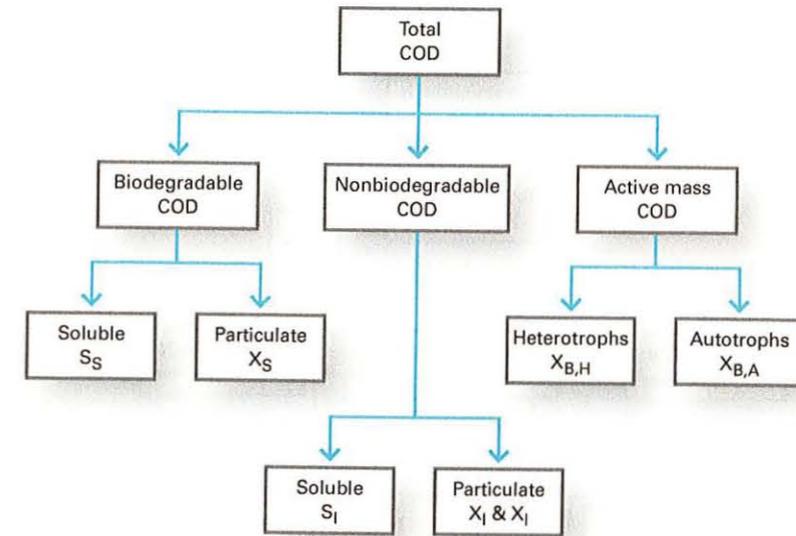


Figure 11.27 COD components of ASM1

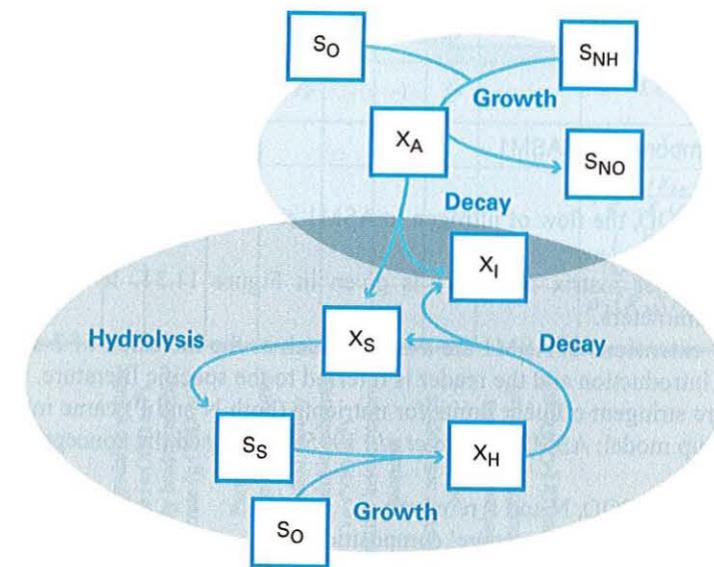


Figure 11.28 Flow of COD in ASM1

The different N components considered in ASM1 are given in Figure 11.29. Total nitrogen is the sum of nitrate ( $S_{NO}$ ) and total Kjeldahl-N (TKN). The latter is further broken down into ammonium  $S_{NH}$ , organically bound N and active biomass N ( $X_{NB}$ ), (the latter not being an actual component in ASM1, but merely defined through a N-fraction of biomass (i.e. 8%)). The organically bound N is partitioned further into soluble and particulate fractions which both have their biodegradable ( $S_{ND}$  and  $X_{ND}$ ) and non-biodegradable ( $S_{NI}$  and  $X_{NI}$ ) fractions. Again, the latter two are not actual components, but rather fractions.

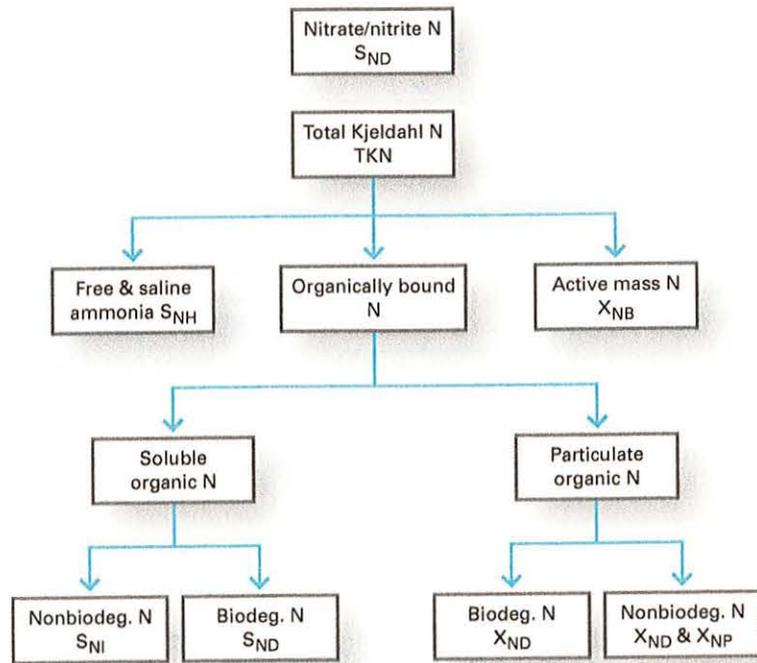


Figure 11.29 Nitrogen components of ASM1

In a similar way as for COD, the flow of nitrogen in ASM1 can be summarised in a self-explanatory graph; see Figure 11.30.

Finally, the complete Gujer matrix of ASM1 is given in Figure 11.31. It consists of 8 processes, 13 components and has 19 parameters.

In the literature many extensions of ASM1 are available such as the inclusion of 2-step nitrification. This is beyond the scope of this introduction and the reader is referred to the specific literature.

During the 1990s more stringent effluent limits for nutrients (both N and P) came into force. This led to the development of a follow-up model: ASM2 (Henze *et al.*, 1995a). It reused the concepts of ASM1 and its major assets are

- combined processes for COD, N and P removal
- a cell internal structure using an 'average' composition
- a TSS state variable in place of X
- chemical precipitation

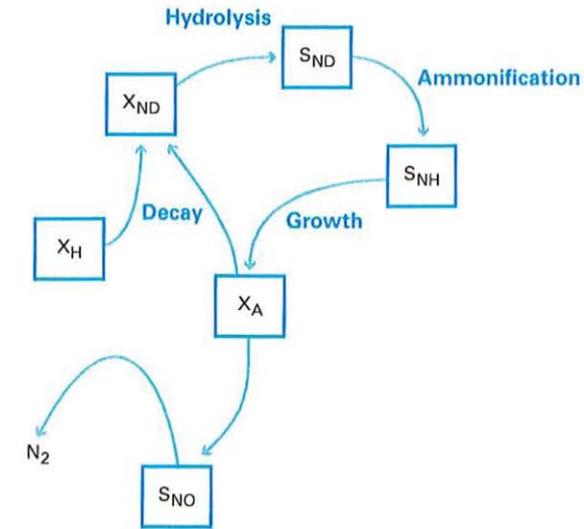


Figure 11.30 Flow of N in ASM1

<i>i</i>	1	2	3	4	5	6	7	8	Process rate
<i>S<sub>I</sub></i>	<i>S<sub>S</sub></i>	<i>X<sub>I</sub></i>	<i>X<sub>S</sub></i>	<i>X<sub>B,H</sub></i>	<i>X<sub>B,A</sub></i>	<i>X<sub>U</sub></i>	<i>S<sub>O</sub></i>		
<b>J</b>	<b>Process</b>								
1		$\frac{1}{Y_H}$			1			$-\frac{1-Y_H}{Y_H}$	$\mu_{mH} \left( \frac{S_S}{K_S + S_S} \right) \left( \frac{S_O}{K_{OH} + S_O} \right) X_{B,H}$
2		$\frac{1}{Y_H}$			1				$\mu_{mH} \left( \frac{S_S}{K_S + S_S} \right) \left( \frac{K_{OH}}{K_{OH} + S_O} \right) \left( \frac{S_{NO}}{K_{NO} + S_{NO}} \right) \eta_E X_{B,H}$
3						1		$-\frac{4.57 - Y_A}{Y_A}$	$\mu_{mA} \left( \frac{S_{NH}}{K_{NH} + S_{NH}} \right) \left( \frac{S_O}{K_{OA} + S_O} \right) X_{B,A}$
4				$1 - f_P$	-1		$f_P$		$b_H X_{B,H}$
5				$1 - f_P$	-1		$f_P$		$b_A X_{B,A}$
6									$k_a S_{ND} X_{B,H}$
7		1		-1					$k_H \frac{X_S / X_{B,H}}{K_x + X_S / X_{B,H}} \left[ \left( \frac{S_O}{K_{OH} + S_O} \right) + \eta_A \left( \frac{K_{NH}}{K_{NH} + S_{NH}} \right) \left( \frac{S_{NO}}{K_{NO} + S_{NO}} \right) \right] X_{B,H}$
8									$\rho_7 (X_{ND} / X_S)$
	Unbiodegradable soluble matter (g COD m <sup>-3</sup> ) Readily biodegradable substrate (g COD m <sup>-3</sup> ) Particulate unbiodegradable matter Enmeshed slowly biodegradable substrate (g COD m <sup>-3</sup> ) Active heterotrophic biomass Active autotrophic biomass Unbiodegradable particulates from cell Oxygen (g (-COD) m <sup>-3</sup> )								

Figure 11.31 The Gujer Matrix for Activated Sludge Model No. 1

It is beyond the scope of this introduction to describe ASM2 in detail. Some specifications are given instead. The Gujer Matrix of ASM2 contains 19 processes and 19 components and not all variables are determined upfront. Continuity is guaranteed for COD, N, P and charge. ASM2d is a minor extension of the original ASM2 including anoxic P-uptake and PAO growth which was not included in ASM2, but yet observed in practice (Henze *et al.*, 1999). Therefore, ASM2d is the most frequently used model when a system including bio-P removal is to be modelled.

ASM3 was introduced in 2000 (Henze *et al.*, 2000a). This is merely a rewrite of ASM1 correcting some deficiencies observed during 10 years of using ASM1 in practice. Two major changes in concept were made:

- Endogenous respiration instead of death-regeneration
- Shift from hydrolysis to storage → cell internal components

Continuity is guaranteed for COD, N and charge.

A summary of the family of ASM models and their applicability is given in Table 11.7.

**Table 11.7** Overview of the specific features and applicability of the ASM family of models

Model	Used for
ASM1	COD and N removal, death-regeneration
ASM2	COD, N and P removal
ASM2d	Extension of ASM2 with anoxic P-uptake
ASM3	COD and N removal, endogenous respiration; metabolic storage

Nowadays ASM1 is still the most widely used model in practice since it has proven its robustness and has a rather limited number of parameters compared to ASM2 and ASM3. More details of all models can be found in Henze *et al.*, 2000a and Henze *et al.*, 2008.

Next to the ASM models, a dedicated model for anaerobic digestion has been developed (Anaerobic Digestion Model Nr 1, ADM1). This is not discussed here, but details can be found in Batstone *et al.* (2002).

#### 11.4.4 Settler modelling

There are several incentives for modeling primary and secondary settling processes. A review of different settling models can be found in Ekama *et al.* (1997).

For primary clarifiers, a good prediction of the load (and COD/N ratio) leaving the primary clarifier and entering the activated sludge reactors is of great importance as there is an optimum for efficient N-removal.

For secondary settlers incentives are:

- a sludge balance between biological tanks and clarifiers
- the dynamics of sludge motion between biological tanks and settler
- effluent quality (effluent suspended solids – ESS)
- settler performance: sludge blanket height (SBH)
- the introduction of control systems for polymer dosage in case of bad settling
- sludge production (thickening)
- settler design: placement of baffles

Settler models are classified according to their spatial detail:

- 0D (point settler)
- 1D (homogeneous in  $x$ - and  $y$ -direction)

- 2D (homogeneous in  $y$ -direction)
- 3D (non-homogeneous)

Besides their spatial description, settler models use a settling velocity model. This is discussed below. In some cases also processes can be included. Examples are the inclusion of hydrolysis in primary settler models and the inclusion of denitrification in secondary settler models to predict the amount of  $N_2$  production which could generate rising sludge ( $N_2$ -bubbles take sludge flocs to the surface of the clarifier). In what follows, some of the important settling models are discussed in more detail.

0D-models or point settlers are ideal separators without volume. A point settler can be regarded as an ideal flow splitter, thereby conserving mass. The model is given by

$$\begin{aligned} Q_f X_f f_{ns} &= Q_{eff} X_{eff} \\ Q_f X_f (1 - f_{ns}) &= Q_u X_u \end{aligned} \quad (11.31)$$

where  $Q_f$  and  $X_f$  respectively represent the feed flow ( $m^3/s$ ) and feed concentration ( $g/m^3$ ) to the settler,  $f_{ns}$  is the non-settleable fraction.

$Q_{eff}$  and  $X_{eff}$  are the respective effluent flow rate ( $m^3/s$ ) and effluent suspended solids ( $g/m^3$ )

$Q_u$  and  $X_u$  are the respective underflow flow rate ( $m^3/s$ ) and underflow sludge concentration ( $g/m^3$ )

The only degree of freedom or parameter is the non-settleable fraction, which can in some models be an increasing function of flow rate to mimic the effect of turbulence in the settler which affects the ESS. The major advantage is its simplicity and it can therefore be used if none of the above incentives is part of the modeling exercise. However, the major problem of this model is that it has no residence time. Some fixes have been proposed to correct this problem:

- point settler with volume mimicking the sludge retention using a number of CSTRs with the following equation

$$\frac{dM_X}{dt} = Q_f X_f - Q_{eff} X_{eff} - Q_u X_u \quad (11.32)$$

- point settler sequenced by a volume representing the sludge blanket 1D-models discretise the 1-D PDE

$$\frac{\partial X}{\partial t} = \frac{\partial}{\partial z} \left( D \frac{\partial X}{\partial z} \right) - \frac{\partial}{\partial z} (V_s X) - RX \quad (11.33)$$

using a finite difference method for the first derivative:

$$\frac{\partial X}{\partial z} = \frac{(X(j) - X(j+1))}{h}$$

The latter is multiplied by the sludge settling velocity  $V_s$ , which is discussed below. This assumes that  $X$  is uniform in the horizontal plane and that no vertical dispersion occurs ( $D = 0$ ). Moreover,  $R$  is often ignored assuming no biological reactions. The discretisation of the 1D-model is illustrated in Figure 11.32. 10 layers are chosen, where layer 6 is used as the influent layer. The resemblance of the discretised model to the actual solution of the 1D-PDE is illustrated in Figure 11.33. The left figure shows the fit of a discretised 10-layer model to an experimental dataset. The right figure shows the PDE-solution for different values of the dispersion

coefficient  $D$ . It can be observed that both model outputs are similar. It seems that the dispersion coefficient is inversely related to the number of layers in the model. The figures also illustrate what type of information is provided by a 1D settler model: effluent and underflow sludge concentrations, sludge blanket height and total sludge mass.

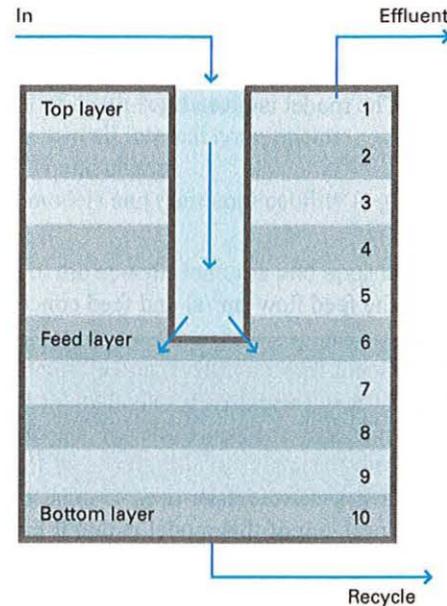


Figure 11.32 Illustration of the discretisation of the 1D settler model

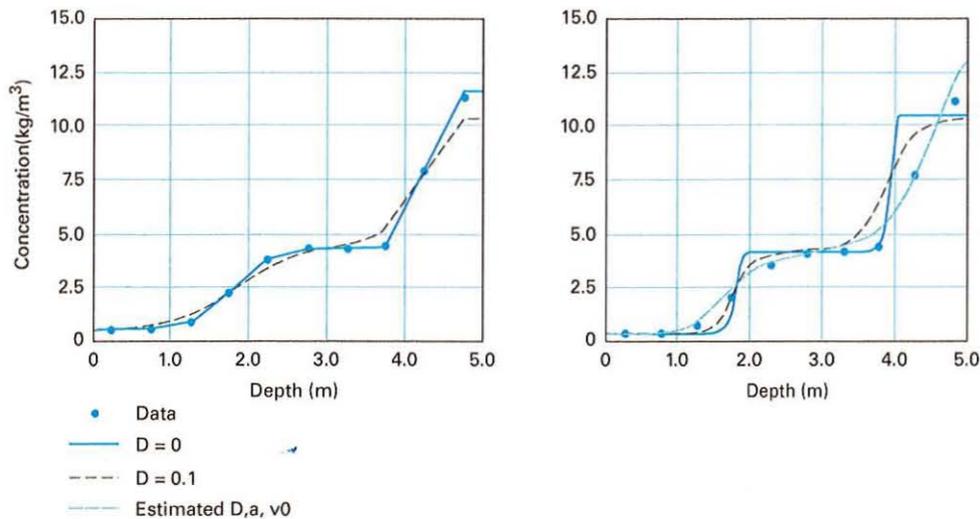


Figure 11.33 1D-settler performance of the 10-layer discretised model (left) and the PDE solution for different values of  $D$  (right)

2D and 3D models solve the Navier-Stokes equations for fluid dynamics. These are discussed in Section 5.2.1. A typical output of a computational fluid dynamics (CFD) clarifier model is given in Figure 11.34. It contains detailed 2D information concerning velocity and solids concentration, and clearly shows the sludge blanket and the concentration gradient.

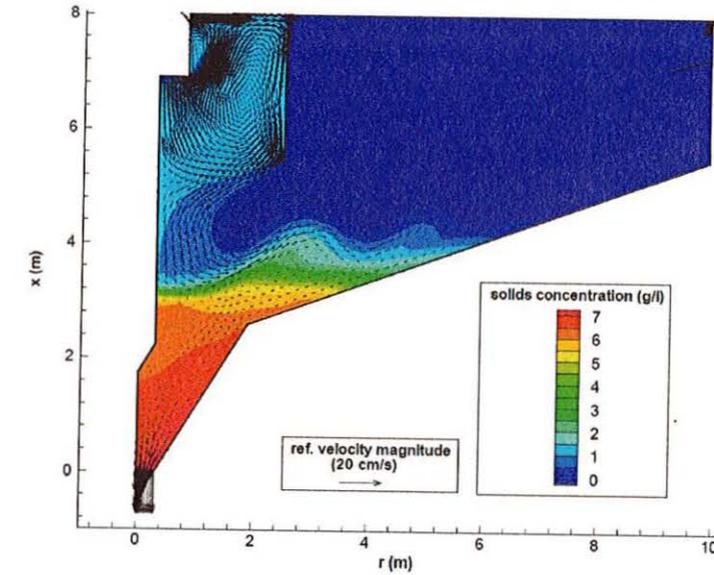


Figure 11.34 Illustration of a typical 2D-settler model output using Computational Fluid Dynamics

Next to the spatial detail, settling models need an expression for the settling velocity  $V_s$ . The latter is a sludge property and is typically measured using a batch settling test. The latter uses a 1 L cylinder where the sludge is allowed to settle and the supernatant-sludge interface is recorded versus time. This results in batch settling curves as illustrated in Figure 11.35.

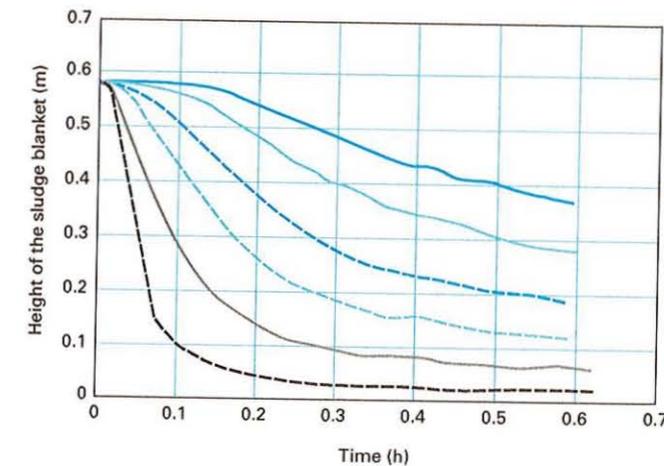


Figure 11.35 Batch settling curves for different initial sludge concentrations (from bottom to top: 3; 5; 7.7; 9.7; 12.7 and 15.6g/L).

The curve consists of a couple of phases. After an initial transient phase, there is a linear part  $c$ , which is called the hindered settling velocity. This means that the sludge is not in contact, but the flocs can sense one another during settling. This linear part is used as a measure for the settling velocity  $V_s$ . After a while, compression settling kicks in, that is, the sludge flocs are actually in contact with one another and feel a resistance to further settlement. The way the curve then behaves depends on the compression properties of the sludge. It is clear that the batch settling curves change shape when the initial sludge concentration is altered. Higher concentrations result in less steep slopes, indicating slower settling. The different linear slopes are then typically plotted against the sludge concentration.

Several models have been proposed to fit  $v_s$  versus  $X$  datasets. The most widely used are:

- Vesilind (1968):

$$v_s = ke^{-nX} \quad (11.34)$$

- Takacs *et al.* (1991):

$$v_{sj} = v_0 e^{-r_h(X_j - X_{min})} - v_0 e^{-r_p(X_j - X_{min})}$$

with  $0 \leq v_{sj} \leq v_{00}$  (11.35)

Both models are illustrated in Figure 11.36. The latter model is actually an extension of the former, correcting for the ridiculously large settling velocities predicted by the Vesilind equation at small sludge concentrations.

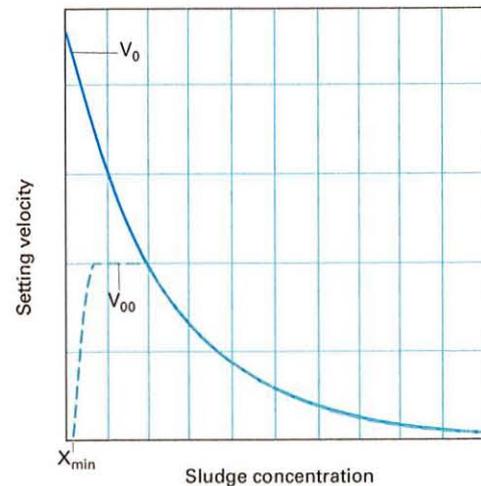


Figure 11.36 Illustration of the Vesilind equation (black) and the Takacs (blue) settling models

The spatial model combined with the sludge settling velocity models make up the typically used settler models and can predict the fate of particulate material in clarifiers.

#### Possible extensions

Next to the fate of particulates, there is also a need to deal with soluble materials because clarifiers typically have a large volume and therefore cause the solubles to have a residence time which causes a delay in the

outflow of these components to the effluent. There are three options to deal with this:

- The solubles residence time is not modeled as such: this can be used when the time delay of the solubles in the effluent is of no interest
- The solubles residence time is modeled as a CSTR, which is able to mimic the delay to the effluent by introducing a minimal number of equations (i.e. one equation per soluble component, or 6 additional equations in the case of ASM1)
- A mass balance is written down for every soluble component in each layer of the model. This is the most detailed way of handling the problem, but introduces more equations to the system (i.e. 9 additional equations per soluble component, or 54 additional equations in the case of ASM1)

The effect of using a different number of layers for soluble components is illustrated for ammonium in Figure 11.37 for the three cases above.

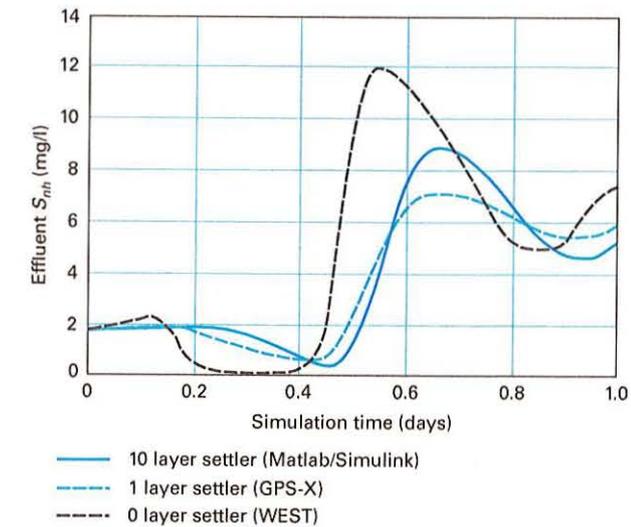


Figure 11.37 Illustration of the impact of assuming different layers to represent the soluble components in a settler model

Also, specific treatment is necessary for the particulate material. The reason for this is that in ASM several particulate components are present, whereas in the settler model only the TSS concentration is used. There are again three ways of dealing with the problem:

- A step response for a particulate component is defined and used to construct the dynamics of the settler underflow for that particulate component
- A lump-delump approach can be used to lump (i.e. sum) the different particulate components. During this lumping, the fractions of the different components are computed and stored. At the underflow, the different components are reconstructed from the same fractions depending on the predicted underflow sludge concentration.
- A propagation approach can be used in which for each layer a mass balance is written down for each particulate component. Again, this is the best way of dealing with the problem, albeit again adding more equations.

In both cases, the most sophisticated implementation is needed when modelling biological reactions in what are often called **reactive settler models**. These models predict denitrification activity which produces nitrogen

gas production rates that can be coupled with loss of sludge through flotation. This process is illustrated in Figure 11.38.

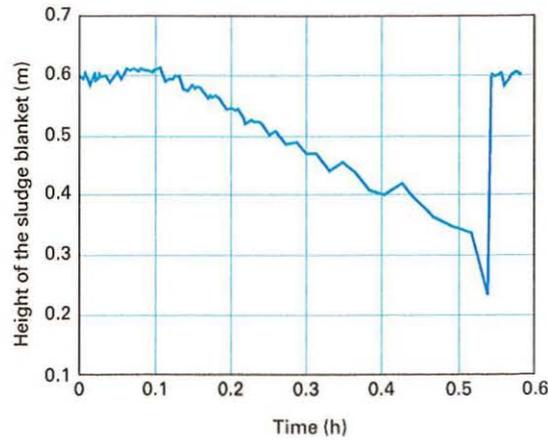


Figure 11.38 Illustration of the denitrification impact on sludge blanket height prediction in a reactive settler model

A second example of reactive settlers is the modeling of hydrolysis and ammonification processes in primary settlers.

A final model extension is for a polymer dosage. The idea is that the settling parameters are dependent on the polymer concentration. The effect of the polymer concentration on the parameters is modelled to deduce the clarifier performance. This can be achieved by measuring the batch settling curves after the addition of different concentrations of the polymer, and repeating the measurements for different sludge concentrations. An illustration of polymer addition at a fixed sludge concentration is shown in Figure 11.39. As can be observed, the settling velocity (i.e. slope of the linear part of the batch settling curve) increases as the polymer dosage increases.

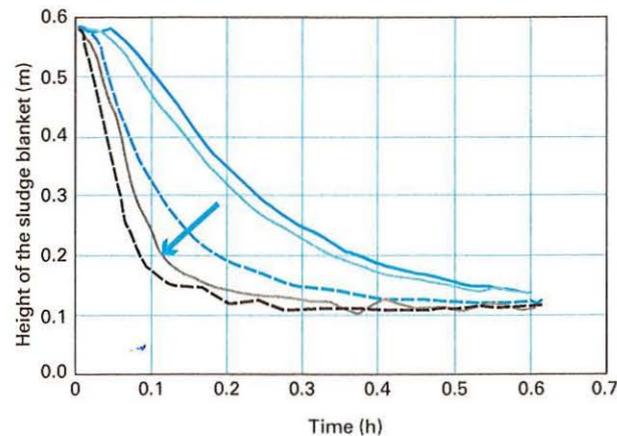


Figure 11.39 Illustration of the effect of polymer addition on batch settling curves at one sludge concentration

When this exercise is repeated for different sludge concentrations, a three dimensional graph is obtained as shown in Figure 11.40. In order to incorporate these results into a settler model, an empirical relationship can be sought between the settling velocity, the sludge concentration and the polymer addition. For the case shown, this was found to be:

$$V_s = 10.59e^{-X/(1.54P+2.5)} \tag{11.36}$$

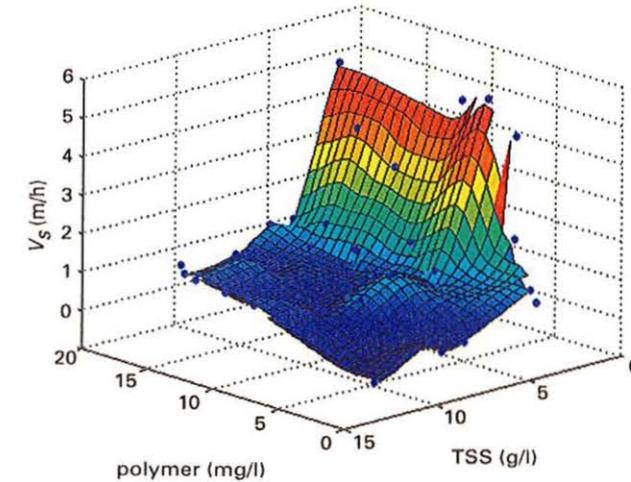


Figure 11.40 Three dimensional effect of polymer addition on batch settling curves at different sludge concentrations

### 11.4.5 Membrane filtration modelling

The detail with which the filtration step is modelled depends on the goal of the study. If there is no interest in the filtration behaviour of the plant, a perfect settler model can be used with a non-settleable fraction of zero (i.e. complete retention of solids). In this case, nothing can be said about the filtration efficiency or the dynamic variation of the transmembrane pressure (TMP). If the latter is needed, a more sophisticated model should be used. However, research is currently ongoing to develop these models. Most of them are based on Darcy's law. This gives the relationship between the pressure drop over the membrane and the permeate viscosity, filtration flux and the several contributions to the filtration resistance:

$$\Delta P(t) = \eta_p J_G (R_m + R_{irr} + R_c) \tag{11.37}$$

where  $R_m$  is the membrane resistance (1/m)

$R_{irr}$  the irreversible fouling resistance (1/m)

$R_c$  the filter cake resistance (1/m)

The former, also referred to as the clean water resistance, is easily determined by filtering clean water over a virgin membrane and logging the pressure drop versus the flux. The second resistance is due to pore blocking and is related to the reduction in the available membrane surface for active filtration, which can be modeled using a simple mass balance:

$$R_{irr}(t) = R_m \left( \frac{A_0}{A(t)} - 1 \right) \tag{11.38}$$

$$\frac{dA(t)}{dt} = -\sigma_{COD} C_b(t) Q(t)$$

where  $A_0$  is the clean membrane surface ( $m^2$ )

$A(t)$  is the dynamic evolution of the membrane surface ( $m^2$ )

$\sigma_{COD}$  is the fouling propensity of the sludge water COD ( $m^2/g$ )

$C_b$  is the bulk water COD concentration ( $g/m^3$ )

$Q$  is the filtration flow rate ( $m^3/s$ )

The contribution of the cake layer to the filtration resistance is based in the filter cake mass and its permeability. Also here, a mass balance can be used for the filter cake mass dynamics, which in its turn is dependent on the fraction of the bulk COD that is able to reach the filter cake. This can be empirically related to the critical flux and the shear:

$$R_c(t) = \alpha \frac{w(t)}{A_0}$$

$$\frac{dw(t)}{dt} = C_d(t)Q(t) \tag{11.39}$$

$$C_d(t) = C_b(t) \left[ \frac{J_L(t)}{J_m} \right]^n$$

where  $\alpha$  is filter cake permeability,

$A_0$  is the clean membrane surface ( $m^2$ )

$R_c$  is the filter cake resistance ( $1/m$ )

$Q$  is the infiltration flow rate ( $m^3/s$ )

$w(t)$  is the dynamic filter cake mass (kg)

$C_d$  is the COD fraction that is able to reach the membrane

$J_L$  is the current operational flux ( $l/m^2/h$ )

$J_m$  is the critical flux above which all particulate material deposits on the membrane ( $l/m^2/h$ )

$n$  is a fitting parameter.

An illustration of the model to predict cyclic dynamic transmembrane pressure between two backwash events is shown in Figure 11.41. This illustrates that the TMP-increase between two backwash events gradually increases in slope, indicating irreversible fouling. Once a certain maximum TMP is reached, a chemical cleaning is required. The slope itself depends on the cake filtration.

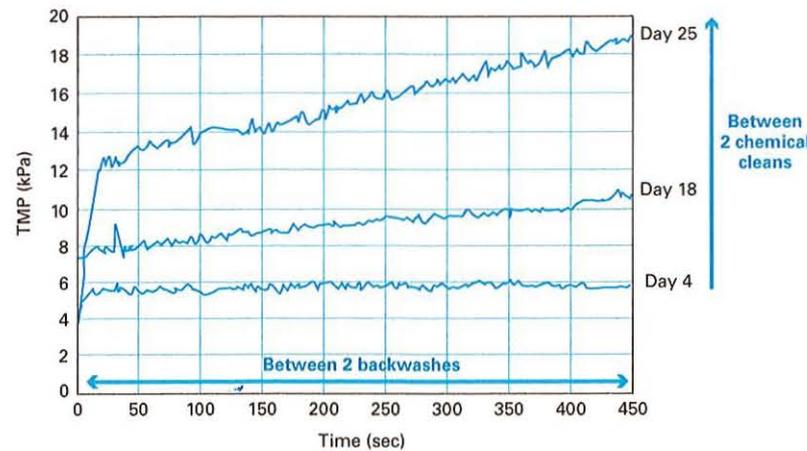


Figure 11.41 Illustration of short-term cyclic dynamic TMP (after Jiang, 2007)

Finally, Figure 11.42 illustrates long-term dynamic TMP. This illustrates the evolution of the irreversible fouling.

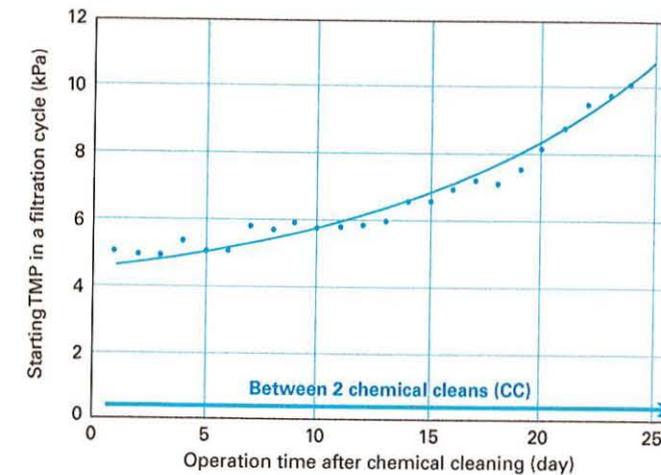


Figure 11.42 Illustration of long-term dynamic TMP (after Jiang, 2007)

### 11.5 SIMULATION SOFTWARE

A wastewater treatment simulator is software that allows the modeller to simulate a wastewater treatment configuration. A detailed overview of simulators can be found in Olsson and Newell (1999) and Copp (2002). General purpose simulators can be distinguished from specific wastewater treatment simulators. The former exhibit a high degree of flexibility, but the modeller has to supply the models that are to be used. Hence, the use of a specific software product depends on the skills of the modeller. However, every modeller should at least know what the model he or she is using contains. Blindly using models without knowing their content is dangerous, or quoting Rieger *et al.* (2010): ‘a powerful tool is a more dangerous tool: it places more responsibility on the craftsmen who forge it, and requires a well trained operator for its efficient use’.

Examples of specific commercial wastewater treatment simulators are (in alphabetical order):

- [www.wrcplc.co.uk/software](http://www.wrcplc.co.uk/software)
- WEST ([www.mostforwater.com](http://www.mostforwater.com))

### 11.6 OUTLOOK

Wastewater technology is moving forward in many directions. It focuses on increasing the performance of the existing systems in many classical facets including resource consumption (energy, chemicals, construction materials, ...) and environmental impact (COD, nitrogen, phosphorus, ...). Increasingly, the use of improved operation schemes involving automatic control is supplementing improved design approaches to reach these objectives. For both approaches (operation and design) models have turned out to be key tools and there has been widespread acceptance of them in practice during the last decade.

The wastewater engineer is however, faced with increasing challenges: he needs to be aware of a multitude of design options (configurations) and operation modes. He must be able to discuss with control engineers

and think along with them to increase process performance and he must be able to use advanced (dynamic) modelling tools to support the design and operation scheme he proposes.

But there is more to come and in some instances the challenge has already materialised. For instance, the horizon of the wastewater engineer is receding further and further away. Plant-wide modelling becomes a standard and requires the engineer to evaluate all aspects of a treatment plant, in particular the sludge treatment train (thickening, dewatering, digestion, ...) and the return liquors (sidestreams) that affect the water line of the treatment plant. He must now be able to install sidestream treatment systems (Anammox, Babe, Demon, OLAND, Sharon,...) that are efficient on their own to deal with, for instance, the nitrogen load coming from digesters. Modelling challenges are such that the engineer must either be able to master a supermodel which follows all the relevant components throughout the plant or he must be able to set up interfaces between more traditional models (e.g. ASM1 and ADM1) so that the material flows between water and sludge line are properly described.

There are also developments in wastewater technology due to new environmental concerns. Attention is increasingly being given to micropollutants, in particular to such chemicals as personal care products, heavy metals, endocrine disrupting compounds, pharmaceuticals, etc. This is leading to the development of a wide range of new treatment technologies involving ozonation, activated carbon adsorption, membrane filtration, etc. Corresponding models are starting to be developed, but much basic research is still required to master these new technologies. In any case, the challenge is for the wastewater engineer to get to grips with the technology toolbox that is being developed.

Another important development is the rapidly increasing time span over which a treatment plant is being evaluated in detail. Awareness of climate change is being introduced in the evaluation of treatment plants, not only in terms of how the plant is exposed to the effects of climate change (e.g. through more extreme rain events) but also to what extent the plant is contributing to climate change through its resource use (energy, chemicals, ...) and directly to greenhouse gas emissions (methane, nitrous oxide, ...). The first models that evaluate greenhouse gas emissions and perform life cycle assessments to evaluate the overall impacts of treatment systems are being applied; these can be expected to become mainstream in years to come.

Finally, we may be at a brink of a paradigm shift in wastewater treatment: Sustainability demands that we acknowledge wastewater as a renewable resource from which water, materials (e.g., nutrients, bioplastics), and energy can be recovered. By shifting away from today's paradigm, which focuses on what must be removed from wastewater, to a new paradigm focusing on what can be recovered, treatment systems may begin to be described as resource recovery systems – a conceptual transformation that could allow the perceived impact of wastewater on communities to become a positive contribution.

## 11.7 CASE STUDY: UPGRADING LARGE WASTEWATER TREATMENT PLANTS: USE OF MODELLING AS A DECISION-MAKING TOOL IN SARAJEVO (BOZANIA AND HERCEGOVINA)

### 11.7.1 Introduction

Since the introduction of activated sludge models (Henze *et al.*, 1987 and Henze *et al.*, 2000a), mathematical modelling has become an irreplaceable tool for the upgrade, retrofit, troubleshooting and optimization of wastewater treatment plants (Brdjanovic *et al.*, 2000; Pinzon *et al.*, 2007; Brdjanovic *et al.*, 2007 and Henze and Comeau, 2008). This case study describes the use of mathematical models in a decision-making process concerning the reconstruction of a wastewater treatment plant (WWTP) in the city of Sarajevo, the capital of Bosnia and Herzegovina.

### 11.7.2 Modelling of Sarajevo sewage system

The Sarajevo sewage system includes a sanitary sewage system and a rainwater drainage system (the total length of sewer pipes in both systems is about 1,000 km with diameters up to 1.80 m). Although in theory the systems are separate, they suffer from cross connections, excessive infiltration and backflow from the river. The WWTP (with a maximum flow of 4.3 m<sup>3</sup>/s and a designed capacity of 600,000 PE) was constructed in 1984 as a conventional activated sludge system for the removal of suspended solids (SS) and organic matter (COD) with anaerobic sludge digestion and biogas reuse for energy production. The plant was operational from 1984 until 1992. Severe problems were occasionally experienced because of an excessive accumulation of sand, grit and gravel at the inlet to the plant. Therefore, the plant operation was often interrupted, usually during and/or after periods of heavy rain or melting snow (Peric *et al.*, 1991). Several mitigation studies (Peric *et al.*, 1991 and JICA, 1999), etc have been carried out so far and all recommend the construction of a large sand trap in front of the existing inlet. However, the inlet is only the place where the problem is manifested; it is not necessarily the location of its origin. A recent model-based sediment transport study (DHI, 2009), using the DHI software package MIKE URBAN<sup>®</sup>, showed that indeed it may well be feasible to build several smaller sand traps within the sewer networks as an alternative to the centralized solution from the '90s.

The WWTP has been out of operation since 1992 as a result of devastation during the recent war (1992–1995). Untreated sewage has been discharging to the nearby river for almost two decades. The local government is currently planning to rehabilitate both a part of the network (addressing the sediment transport issue too) and the WWTP, under the framework of a substantial reconstruction programme to be financed by a loan from the World Bank. Four main questions arose during the decision-making process regarding the WWTP, namely: (i) should the plant be reconstructed in its original state as in 1984?, (ii) should it be upgraded by nutrient-removal (nitrogen and phosphorus) facilities to satisfy more stringent effluent requirements (EU standards) in the future?, (iii) what is the most appropriate technological process to be applied?, and (iv) what are the possible extent and scope of the reconstruction, given the available financial resources?

These questions were addressed (ASM Design, 2009) by the application of ASM models incorporated in the simulation software BioWin<sup>®</sup> of EnviroSim Associates Ltd; see Figure 11.43. In total, a dozen technological and engineering options (Figure 11.44) were considered, which took into account the original design, the present and future composition and quantity of sewage (up to 2030), the choice of wastewater treatment technology (an existing anoxic-oxic A/O system, or a future upgrade to an anaerobic-anoxic-oxic A2O/UCT system or oxidation ditch/Carousel<sup>®</sup> system), the choice of an aeration system (existing surface aeration by turbines, or an alternative with fine bubbles aeration by air diffusers), and the amount and composition of the sludge (and later biogas) generated in the process.

The model simulation showed that for all the scenarios which involve the adoption of EU standards for nutrient removal, the existing plant will not be able to achieve these standards. Therefore even for minimal improvement (e.g. nitrification), a large additional volume and aeration capacity is required. In order to reach such standards the sludge age needs to be extended from the designed 2–3 days to 10–15 days and the activated sludge section needs to be 3–5 times the current volume. Furthermore, 150% of the current secondary settling volume will be required. Extension by one, and later another, secondary settling tank will be necessary. The study showed that by the reconstruction of the primary treatment units, a reduction of an equivalent load of 320,000 PE will be achieved. Reconstruction of digestion to a full anaerobic digestion and biogas production as in the original design will also be necessary. Operating the existing aeration tanks with a diffused bubble aeration system will result in BOD, COD and SS concentrations in the effluent of below 25, 125 and 35 mg/L, respectively. It is possible to extend the existing A/O plant by an additional tank volume of 48,000 m<sup>3</sup> and retrofit it into an A2O (UCT) process to reach a nutrient level in the effluent of 10 mgN/L and 1 mgP/L (current EU standards for plants larger than 100,000 PE).

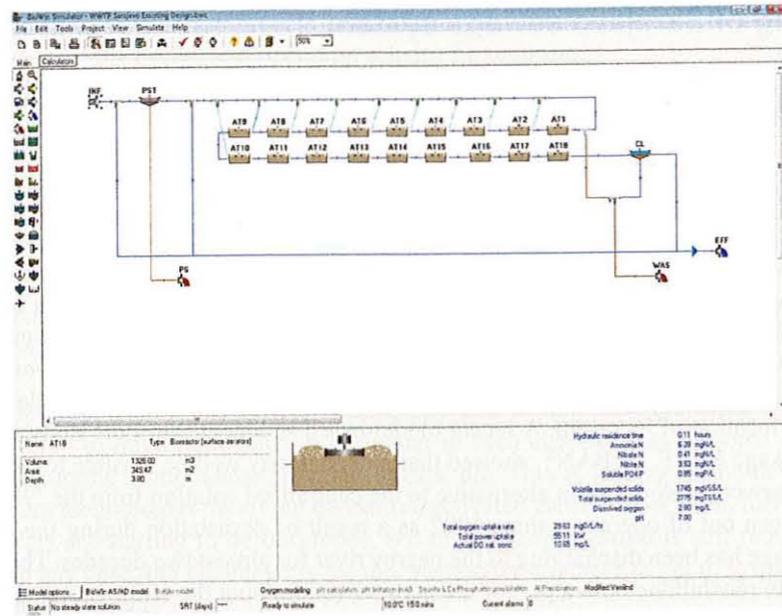


Figure 11.43 Hydraulic plant scheme in the BioWin simulator

Simulation Scenario Overview						
No.	Description	Influent pre-treatment	Biological Tanks	Configuration	Aeration Type	Removal performance
0	Existing reactors for BOD removal - installed surface aerator capacity, settled influent	Settled	Existing	Plug-flow	Turbines	BOD
1	Existing reactors for BOD removal - new surface aerator capacity, settled influent	Settled	Existing	Plug-flow	Turbines	BOD
2	Existing reactors for BOD removal - bubble aerators, settled influent	Settled	Existing	Plug-flow	Bubble	BOD
3	Existing reactors for BOD removal - bubble aerators, raw influent	Raw	Existing	Plug-flow	Bubble	BOD
4	Nitrifying Extended Volume Plug flow - bubble aerators, settled influent	Settled	Extended	Plug-flow	Bubble	BOD/Nitrifying
5	Nitrifying Extended volume Plug flow - turbines + bubble aeration, settled influent	Settled	Extended	Plug-flow	Turbines/Bubble	BOD/Nitrifying
6	<b>Denitrification, Extended Volume Plug flow - bubble aerators, settled influent</b>	Settled	Extended	Plug-flow	Bubble	<b>BOD/NIT/Denitrification</b>
7	Extended Volume Plug Flow for BOD removal - bubble aeration, raw influent	Raw	Extended	Plug-flow	Bubble	BOD
7a	Extended Volume Plug Flow for BOD removal - bubble aeration, raw influent	Raw	Extended	Plug Flow	Bubble	BOD
7b	Extended Volume Plug Flow for BOD removal - bubble aeration, raw influent	Raw	Extended	MLSS recirculation	Bubble	BOD
7c	Extended Volume Carrousel for BOD removal - bubble aeration, raw influent	Raw	Extended	Carrousel	Bubble	BOD
8	Nitrifying Carrousel type activated sludge system - bubble aeration, raw influent	Raw	Extended	Carrousel	Bubble	BOD/Nitrifying
9	Denitrifying Carrousel AO process - raw influent, bubble aeration	Raw	Extended	Carrousel/AO	Bubble	BOD/NIT/Denitrification
10	<b>Denitrifying Plug flow AO process - raw influent, bubble aeration (EU standard)</b>	Raw	Extended	Plug Flow/AO	Bubble	<b>BOD/NIT/Denitrification</b>
11	A2O Plug flow process - raw influent, bubble aeration (EU standard)	Raw	Extended	Plug flow/A2O	Bubble	BOD/NIT/DEN/Bio-P
12	A2O Carrousel process - raw influent, bubble aeration (EU standard)	Raw	Extended	Carrousel/A2O	Bubble	BOD/NIT/DEN/Bio-P

Figure 11.44 Overview of scenarios examined in this model-based study

The study concludes that: (i) by reconstruction back to its original state and the replacement of surface aeration by a diffused air system the plant will be able to reach the current local standards regarding suspended solids and organic matter; (ii) in the case of the requirement for nutrient removal the plant will need substantial upgrade in terms of volume and aeration capacity; (iii) the existing A/O system is considered appropriate for current effluent quality standards, however the most appropriate technological process to be applied including nutrient removal is the modified UCT process (A2O); and (iv) based on the results of this study it will be possible to determine the scope of the reconstruction to match the available financial resources, given that the costs of preferred options are known.

The study clearly shows the usefulness of mathematical modelling in a decision-making process regarding investments in urban infrastructure in which a compromise often has to be sought between the prioritization of investments and the environmental benefits, as is the case in this study.

### 11.7.3 Acknowledgement

This article was prepared by Sebastian C. F. Meijer (ASM Design, Alexandar Numankade 205, 3572KX Utrecht, The Netherlands) and Damir Brdjanovic (Department of Urban Water and Sanitation, UNESCO-IHE Institute for Water Education, Westvest 7, 2611 AX Delft, The Netherlands and Department of Biotechnology, Delft University of Technology, Julianalaan 67, 2628BC Delft, The Netherlands).

### 11.8 KEYWORDS

By the end of the chapter you should have encountered the following terms; ensure that you are familiar with them!

- |                          |                 |                            |
|--------------------------|-----------------|----------------------------|
| Activated sludge         | Eutrophication  | Primary treatment          |
| Aerobic                  | Exfiltration    | Secondary treatment        |
| Anaerobic                | First flush     | Settleable solids          |
| Anammox                  | Floc            | Sludge blanket             |
| ASM1, ASM2, ASM3         | Gujer matrix    | Stoichiometric coefficient |
| Biofilm                  | Infiltration    | Substrate                  |
| Biofilter                | Membrane        | Suspended Solids           |
| Biological Oxygen Demand | Micro-organisms | Tertiary treatment         |
| Biomass                  | Micropollutants | Total Organic Carbon       |
| Chemical Oxygen Demand   | Nitrogen        | Total Suspended Solids     |
| Denitrification          | Nutrient        | Tracer                     |
| Dilution                 | Phosphorus      | Wastewater                 |
| Dry weather flow         | Polymer         | Wet weather flow           |

### 11.9 QUESTIONS

- What are the primary characteristics of domestic wastewater?
- Describe the three main forms of treatment of wastewater
- What is the Gujer matrix, and how is it applied?
- How are treatment processes modelled mathematically?
- What is membrane filtration, and how does it work?

### 11.10 REFERENCES

ASM Design (2009). *Model Design Study WWTP Sarajevo; Model-Based Process Technology Selection for Rehabilitation of WWTP*. ASM Design, Sarajevo.

Batstone D., Keller J., Angelidaki I., Kalyuzhnyi S., Pavlostathis S. G., Rozzi A., Saunders W., Siegrist H. and Vaviliri V. (2002). *Anaerobic Digestion Model No. 1 (ADM1)*. IWA Task Group for Mathematical Modelling of Anaerobic Digestion Processes. IWA Publishing, London.

- Brdjanovic D., Muthaiwala M., Moussa M. S., Army G. and van Loosdrecht M. C. (2007). Use of plant-wide modeling for optimization and upgrade of a tropical wastewater treatment plant in a developing country. *Water Sci. Technol.*, **56**(7), 21–31.
- Brdjanovic D., van Loosdrecht M. C. M., Versteeg P., Hooijmans C. M., Alaerts G. J. and Heijnen J. J. (2000). Modelling COD, N and P removal in a full-scale WWTP Haarlem Waarderpolder. *Water Res.*, **34**, 846–858.
- CEC (1991). *Directive Concerning Urban Waste Water Treatment*. 91/271/EEC. CEC, Brussels.
- CEC (2000). *Directive Concerning Water Framework – Integrated River Basin Management for Europe*. 2000/60/EEC. CEC, Brussels.
- Copp J. (2002). *The Cost Simulation Benchmark: Description and Simulator Manual*. Office for Official Publications of the European Community, Luxemburg, p. 154.
- Corominas L. and Rieger L. (2010). New framework for standardized notation in wastewater treatment modelling. In: *Proceedings of the 2nd IWA-WEF Wastewater Treatment Modelling Conference*, Mont-Sainte-Anne, 1–3 June, Quebec, Canada.
- De Clercq B., Coen F., Vanderhaegen B. and Vanrolleghem P. A. (1999). Calibrating simple models for mixing and flow propagation in waste water treatment plants. *Water Sci. Technol.*, **39**(4), 61–69.
- DHI (2009). *Preparation of Investment Analysis and Modelling Study for the Sewer Collection Network in Sarajevo, Bosnia and Herzegovina: Model-Based Analysis of Sarajevo's Sewer System*. DHI, Horsholm.
- Dochain D. and Vanrolleghem P. (2001). *Dynamical Modelling and Estimation in Wastewater Treatment Processes*. International Water Association, London.
- Ekama G., Barnard J., Gunthert F. W., McCorquodale J. A., Parker O. S. and Wahlberg E. J. (1997). *Secondary Settling Tanks: Theory, Modelling, Design and Operation*. Scientific and Technical Report No. 6. IAWQ, London.
- Henze M., Grady C. J., Gujer W., Marais G. V. R. and Matsuo T. (2000a). *Implementation of the Activated Sludge Model, Activated Sludge Models ASM1, ASM2, ASM2d and ASM3*. IWA Publishing, London.
- Henze M., Grady C. P. L. J., Gujer W., Marais G. V. R. and Matsuo T. (1987). *Activated Sludge Model No. 1*. IAWQ Scientific and Technical Report No. 1. IAWQ, London.
- Henze M., Grady Jr. C., Gujer W., Marais G. V. R. and Matsuo T. (1987). A general model for single-sludge wastewater treatment systems. *Water Res.*, **21**(5), 505–515.
- Henze M., Gujer W., Gujer W., Marais G. V. R. and Matsuo T. (1995a). *Activated Sludge Model No. 2*. IAWQ, London.
- Henze M., Gujer W., Mino T. and van Loosdrecht M. C. (1999). Activated Sludge Model No. 2d. *Wat. Sci. Technol.*, **39**(1), 165–182.
- Henze M., Gujer W., Mino T. and van Loosdrecht M. C. (2000b). *Activated Sludge Models ASM1, ASM2, ASM2d and ASM3, IWA Task Group on Mathematical Modelling for Design and Operation of Biological Wastewater Treatment*. IWA Scientific and Technical Report, IWA, London.
- Henze M., van Loosdrecht M. C. M., Ekama G. A. and Brdjanovic D. (2008). *Biological Wastewater Treatment: Principles, Modeling, and Design*. IWA Publishing, London.
- Hug T., Benedetti L., Hall E. R., Johnson B. R., Morgaroth E., Nopens I., Rieger L., Shaw A. and Vanrolleghem P. A. (2009). Wastewater treatment models in teaching and training: the mismatch between education and requirements for jobs. *Water Sci. Technol.*, **59**(4), 745–753.
- Henze M. and Comeau Y. (2008). Wastewater characterization. In: M. Henze, M. C. van Loosdrecht, G. Ekama, D. Brdjanovic (eds.), *Biological WasteWater Treatment: Principles, Modelling and Design* (pp. 33–52), IWA, London.
- Jiang T. (2007). *Characterization and Modelling of Soluble Microbial Products in Membrane Bioreactors*. PhD thesis, Ghent University, Ghent.
- JICA (1999). *The Feasibility Study of the WasteWater Treatment Plant of Sarajevo City in Bosnia and Herzegovina*. Final Report. Tokyo Engineering Consultants Co., Ltd/Nihon Suido Consultants Co., Ltd, Tokyo.
- Judd S. and Judd C. (2006). *The MBR Book: Principles and Applications of Membrane Bioreactors in Water and Wastewater Treatment*. Elsevier Science Ltd, Oxford.
- Kampschreur M., Poldermans R., Kleerebezem R., van der Star W. R. L., Haarhuis R., Abrna W. R., Jetten M. S. M. and van Loosdrecht M. C. M. (2009). Emission of nitrous oxide and nitric oxide from a full-scale single-stage nitrification-anammox reactor. *Water Sci. Technol. J. Int. Assoc. Water Pollut. Res.*, **60**(12), 3211.
- Levenspiel O. (1999). *Chemical Reaction Engineering*. Wiley, Hoboken, New Jersey.
- Nopens I., Batstone D., Copp J. B., Jeppson O., Volcke E., Alex J. and Vanrolleghem P. A. (2009). An ASM/ADM model interface for dynamic plant-wide simulation. *Water Res.*, **43**(7), 1913–1923.
- Olsson G. and Newell B. (1999). *Wastewater Treatment Systems: Modelling, Diagnosis and Control*. IWA Publishing, London.
- Peric M., Brdjanovic D. and Razzorov P. (1991). Upgrade of the wastewater treatment plant in Sarajevo (in Bosnian). *Naša Vodoprivreda*, **5/6**, 41–51.

- Pinzon A., Brdjanovic D., Moussa M. S., Lopez Vazquez G. M., Meijer S. C. F., van Straten H. H. A., Janssen A. J. H., Amy M. G. and van Loosdrecht M. C. M. (2007). Modelling oil refinery wastewater treatment plant. *Environ. Technol.*, **29**(11), 1273–1284.
- Rieger L. and Takacs I. (2010). Editorial: Status and future of wastewater treatment modelling. *Water Sci. Technol.*, **61**(4), 821–823.
- Takács I., Patry G. and Nolasco D. (1991). A dynamic model of the clarification-thickening process. *Water Res.*, **25**(10), 1263–1271.
- Tchobanoglous G., Burton F. and Stensel H. D. (2003). *Metcalf & Eddy Wastewater Engineering: Treatment and Reuse*. McGraw-Hill, NY.
- UNESCO (2009). *The 3rd United Nations World Water Development Report: Water in a Changing World (WWDR-3)*. UNESCO Publishing/Earthscan, Paris, London.
- USEPA (2004). *EPA Water Subtopics*. EPA. <http://www.epa.gov/ebtpages/water/html> (accessed 17 October 2010).
- Vanrolleghem P., Benedetti L. and Meirlaan J. (2005). Modelling and real-time control of the integrated urban wastewater system. *Environ. Modell. Software*, **20**(4), 427–442.
- Vesilind P. A. (1968). Theoretical considerations: design of prototype thickeners from batch settling tests. *Water Sewage Works*, **115**, 302–307.