UNESCO-IHE LECTURE NOTES

A Practical Guide to Activated Sludge Modelling



Sebastiaan C.F. Meijer and Damir Brdjanovic

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This book presents part of the results of the project entitled 'Development and Application of Models and Guidelines to Facilitate Decision Making in the Extension of Wastewater Treatment Plants (WWTP) and to Increase Operational Efficiency of Existing WWTPs' which was initiated under the Technical Assistance component of the Inland Water Project of Croatian Waters, financed by the World Bank Loan 7453/HR. It is based on the 'Project Report Step 3: Preparation of Design Guidelines and Criteria for Application in the Rehabilitation/Extension of Existing WWTP' prepared by a Croatian-Dutch consortium consisting of PRONING DHI d.o.o., UNESCO-IHE Institute for Water Education, and ASM DESIGN. The authors of this book wish to acknowledge the support of the client Hrvatske Vode (Ms. Tanja Ecimović, Mr. Dinko Polić and Dr. Siniša Širac) and the valuable contribution by consortium members Mr. Božidar Deduš, Ms. Vlatka Lipošćak, Ms. Vesna Rogulja, Mr. Krešimir Plantić, Mr. Tomislav Horvat, Mr. Darko Jakšić, and Prof. Dr. Marin Matošić. Authors would like to acknowledge the government of the Netherlands for co-funding the preparation of this book as a part of the (online) course 'Modeling Sanitation Systems', and Mr. Jetze Heun of UNESCO-IHE for facilitating this process. Ms. Claire Taylor deserves sincere appreciation for English editing and grammar correction, as well as Hans Emeis and Peter Stroo for graphics and cover design, respectively.

The book is primarily designed to be used as a supplementary reading for the postgraduate students of UNESCO-IHE.

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Chapter 0

Executive summary

0.1 Introduction

The application of mathematical modelling in the wastewater sector started in the early 1980s. Several models surfaced and each model used a different nomenclature and different approach. At that time, the former International Water Quality Association (IAWQ), originally formed as the International Association for Water Pollution Research (IAWPRC) in 1965, and which in 1999 became the International Water Association (IWA), played a pivotal role in the development of wastewater modelling by establishing the 'IAWPRC Task Group on Mathematical Modelling for Design and Operation of Biological Wastewater Treatment' in 1986. A year later, this group produced the Activated Sludge Model nr 1 (ASM1) and this year (1987), is considered to be 'year zero' for wastewater modelling. ASM1 is in essence a consensus model – a compromise resulting from discussions at the time between different modelling groups, most prominently from South Africa, the U.S.A., Switzerland, Japan and Denmark. Many of the basic concepts of ASM1 were adapted from the activated sludge model defined in 1980 by the group from the University of Cape Town, S.A. In the 1990s we witnessed first a sudden burst and then a gradual increase in the use of mathematical models. Nowadays there is an array of models from the ASM family that have evolved over the last two decades such as ASM2, ASM2d, ASM3, ASM3+ etc. This evolution was undoubtedly supported by the availability of more powerful computers and the fast expansion of knowledge about the metabolism of the concerned bacteria or bacterial groups (transition from the black box to opaque box knowledge). Nowadays models are regularly used in almost every plant in the US, Australia, Canada, Japan and many EU countries. For example, in 1994 in the Netherlands, Dutch practitioners started to feel that models could be useful. Soon afterwards (1995), STOWA recommended the use of the SIMBA simulator as a national standard. In 1996 the STOWA modelling and characterization protocol was published. By 1997 approximately half of all WWTP in the Netherlands were modelled. In 1999 the new protocol for model calibration was introduced. Nowadays more than 90% of the new WTTPs in the Netherlands are modelled. At the moment, from a practical perspective, models are considered sufficiently developed and reliable for most of the purposes for which they are being used. Modellers have become a standard asset of any serious consulting company in the country and even specialist modelling companies have emerged in the sector. This has been accompanied by global developments such as the establishment of the IWA Specialist Group on Good Modelling Practice and biannual specialist IWA conference on mathematical modelling that started in 1995 in Denmark. The latest encouraging developments demonstrate that modelling is no longer the preserve of the developed world, as the applications in developing countries (e.g. India) and countries in transition (South Korea, Bosnia and Herzegovina) are also increasing.

0.2 Rationale

In Croatia new legislation has been introduced which stipulates that for agglomerations larger than 10,000 PE located in environmentally sensitive areas (in total 54 in the country are declared as such), it is necessary to apply a tertiary level of wastewater treatment. For full application of the urban wastewater directive, Croatia has requested an implementation period of up until 2023 in order to complete its planned projects on wastewater collection and treatment. It has

been estimated that approximately 3.15 billion EUR will be needed to make it all happen. Given the extent of the investment and the tremendous efforts that the Republic of Croatia is currently undertaking regarding the control of man-made water pollution, it is clearly a necessity that the Croatian wastewater sector is reinforced and that capacity building in the sector takes place at an increased rate. This is not only a prerequisite for sustainable development and safeguarding the above investments, but also an opening door into the world of innovation in the wastewater sector, being not only of a technical nature, but also institutional, financial, social and economic. So far in Croatia modelling has been applied to some extent in the wastewater sector but only to sewage collection and urban drainage systems. Therefore, this innovative modelling project is considered a pioneering application of mathematical modelling on Croatian wastewater treatment practice.

0.3 Project

In Croatia, two national strategic projects are being implemented in the field of water pollution control, namely the Coastal Cities Pollution Control Project (The Adriatic Project) in the Adriatic Basin, and the Inland Waters Project in the Black Sea Basin. The objective of the Inland Waters Project is to improve water pollution control, flood protection and public water supply in the Sava, Drava and Danube river basin districts. The project entitled Development and Application of Models and Guidelines to Facilitate Decision Making in the Extension of Wastewater Treatment Plants (WWTP) and to Increase Operational Efficiency of Existing WWTP has been initiated under the Technical Assistance component of the Inland Water Project - Loan 7453/HR.

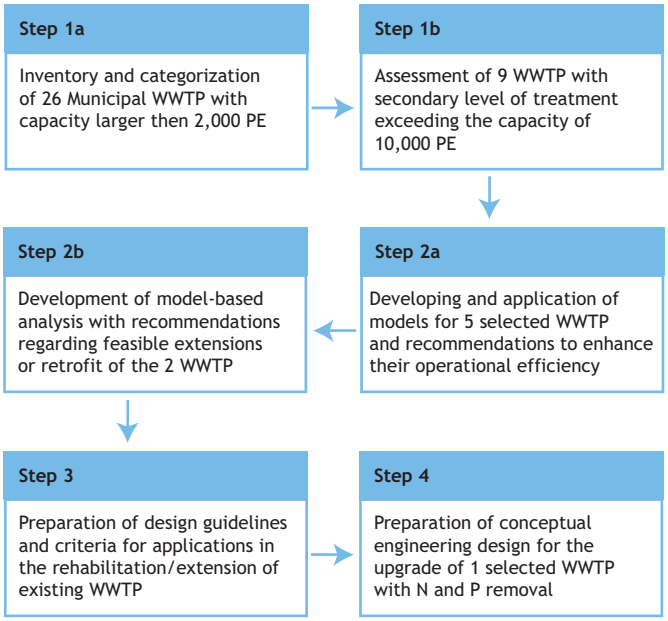

Figure 0.1 Overview of the Development and Application of Models and Guidelines to Facilitate Decision Making in the Extension of Wastewater Treatment Plants (WWTP) and to Increase Operational Efficiency of Existing WWTP.

The Contract for Consultants' Services HV/TA–A1.2.2 for this project was signed in Zagreb on 24 August 2010 by Hrvatske vode (the Client), and the Croatian-Dutch consortium consisting of PRONING DHI d.o.o., Croatia (Lead Partner*),* UNESCO-IHE Institute for Water Education, Netherlands, and ASM DESIGN, Netherlands. The project consisted of four distinctive but interrelated steps. This book concerns Step 3 of the project: Preparation of design guidelines and criteria for application in the rehabilitation/extension of existing WWTP (Figure 0.1).

0.4 Guidelines

The inception phase and inventory of 26 WWTP located in the Black Sea river basin that belongs to the Republic of Croatia (from which 9 large plants were visited by the Consortium) confirmed that most of the plants that employ secondary treatment are variations of an activated sludge technology. Current Croatian practice shows that modern variations of activated sludge technology still prevail regarding the plants that are in design or under construction, and that probably this technology will remain mainstream throughout this decade. Some emerging technologies, such as MBR, are a serious alternative to conventional activated sludge technology, especially in the environmentally sensitive coastal tourist areas, and thus it is no surprise that nowadays dozen of MBR systems are being built, particularly in Istria. Taking into account that at present activated sludge is the most frequently applied technology for sewage treatment in Croatia and that such a trend will probably continue throughout this decade (particularly true for the new large plants), and in anticipation of an even more rapid increase in investment in the Croatian wastewater sector, there is definitely demand and justification, and in fact the necessity, for modernization of the sector; through facilitating the introduction of innovative approaches, the practical implementation of demonstration and pilot projects and capacity building through training and exposure to state-of-the-art technologies of the sector professionals, including representatives of Croatian Waters, ministries, consultancy firms, water and sewage companies, plant operators and academia. This was one of the reasons that the Consortium partners decided to invest additional efforts in the preparation of these guidelines for the practical use of models for designing, upgrading and retrofitting wastewater treatment plants. The guidelines are intended to make practical and simple tools facilitating the use of models in the Croatian sanitation sector and encourage young and mid-career professionals to start using models in their day-to-day work. The strength of these guidelines is that they are founded on the best and proven international modelling practices and on the authors' 15 years of experience in developing and, in particular, applying the models in real-life cases. Furthermore, these guidelines have been adjusted to Croatian circumstances and sector practices by benefiting from the experience gained in the application of models on 5 WWTP carried out during earlier stages of the project.

0.5 For the reader

The guidelines contain 18 chapters. Chapter 1 gives an overview of the wastewater sector in Croatia with particular focus on plants belonging to the Croatian part of the Black Sea river basin. It also includes an overview of the current EU and Croatian legislation concerning effluent discharge standards. The role of modelling in modern wastewater management is explained in Chapter 2, which demonstrates how the modelling goals can vary during the plant life cycle ('modelling roundabout'), how various goals influence the model complexity and at which stage of the plant life cycle application of models can the most money be saved. Chapter 3 which is entitled Introduction to Activated Sludge Models is in fact textbook-type material where the models are introduced in a simplified and easy-to-understand way to generalist sanitation professional. It describes the essence of the modelling approach and the history of activated sludge modelling, and gives an example of stepwise model development. The chapter introduces

general activated sludge modelling (ASM) and various simulators. Chapter 4 builds upon Chapter 3 and introduces general modeling protocols providing a blueprint of the main steps in the modelling of a wastewater treatment plant. Chapter 5 deals with methods for quantitative influent assessment, addressing different components of the urban wastewater chain and introducing a methodology for quantification of sewage components such as domestic sewage, industrial effluents, rainwater and infiltration. A qualitative definition of wastewater (called characterization and fractionation) is described in Chapter 6, another textbook-type chapter in which various waste liquid streams are described in terms of the most important constituents and their concentrations. The chapter concludes with an overview of all the symbols used for wastewater characterization in the most common mathematical models. In order to properly prepare correct input data for the model, general guidelines on model wastewater characterization are presented in Chapter 7. This general approach is followed by a large part of the practical Dutch (STOWA) guidelines for wastewater characterization that have so far been proven to be effective in numerous modelling studies in and outside the Netherlands, including some industrial wastewater treatment plants. In a systematic step-by-step description, the characterization procedure for COD, N and P is given. Chapter 8 follows on from chapter 7 and is specifically focused on the influent characterization as applied by the BioWin simulator. The technology is based on a full plant simulator, including all the WWTP process units, an activated sludge system and, for example, anaerobic digestion, and thereby requires a more detailed explanation on the input calculations also covering, for example, pH calculations. As an additional feature, BioWin proposes an automatic calculation of the influent fractions for the general user. This chapter also includes a number of equations for COD, N and P characterization for broader understanding of the technology. A special feature that makes BioWin both practical and very useful is the ability to simulate BOD (the ASM family models only simulate COD). BOD_5 is a most common practical measurement and the possibility to directly use it in the model greatly improves the practical applicability of the BioWin software. Moreover, also recipient (river) quality models are BOD-based and therefore BioWin provides compatibility for the use of extended modelling. Chapter 9 separately also gives BOD model calculations for more traditional wastewater treatment design as a practical and useful supplement. Chapters 10 to16 describe the 7 steps in the modelling protocol as applied in the 5 case studies in this project. The first step deals with the plant inventory – a systematic guide to the collection of basic data and information from the concerned WWTP. The next step includes the checklist of treatment units of a hypothetical generic WWTP that is the basis for a systematic description (inventory) of a wastewater treatment plant. The description leads to different types of process flow diagrams of the wastewater treatment plant that will be used in the course of the design project. In a similar approach, Chapter 12 describes the inventory of plant flows and measurement points which are used for designing an efficient grab sampling plan for collecting plant information. Also in Chapter 12 is an inventory made up of all the regular day-to-day sampling to be used in the project. Chapter 13 presents a methodology for activated sludge plant assessment and describes methods to evaluate raw plant data and to filter out possible errors which affect the model reliability and also the results of the design study; these errors eventually lead to an increased cost of building an upgrade because a larger safety margin is needed when the measured plant data leaves room for uncertainty. Understandably, a relatively large amount of effort is put into this topic, as is also in the STOWA protocol presented in Chapter 16. The data is checked, based on the mass balance determination concerning the hydraulic flow and the COD, N and P content of process flows and activated sludge. Calculations of sludge age are also presented with some examples. As secondary settlers are (almost) a standard part of activated sludge systems, a methodology for secondary settlers' assessment is described in Chapter 14, including the five most common empiric settler design procedures. Again this is traditional wastewater treatment design merged with state-of-the-art model application, which makes a strong combination.

When one embarks on a new modelling task, it is very unlikely that the available (routinely collected) data will be sufficient for modelling purposes. Therefore, it is more the rule than the exception that additional sampling has to be carried out, and regularly a proper program has to be prepared. Given the importance of such additional information and the effort and cost involved, it is necessary to include basic guidelines for the preparation of such a program in this document (Chapter 15). The last step (Chapter 16) elaborates on the methodology applied in model calibration and the main steps thereof. For the beginner model designer this chapter will be most valuable, introducing a simple and understandable calibration approach. In literature, too many examples are found of impractical procedures that do not lead to accurate results. The approach proposed here was adopted by STOWA and developed by the authors. Chapter 17 includes a plan and further division of the works into practical tasks. A detailed sample plan and time estimation covering all the steps in the guidelines is included in the form of a Gantt chart (MS Project) and an orientation timeline is provided for each model step. The guidelines conclude with a set of final remarks and recommendations.

0.6 Short description of the guidelines

The developed guidelines originate from the Dutch experience of model design in activated sludge modelling (Model Based Design or MBD). The aim of the guidelines (the model protocol) is to reduce the complexity of model design projects. Therefore, the model design is divided into smaller sub-tasks. Seven different phases in wastewater treatment plant modelling are distinguished according to the general scheme shown below.

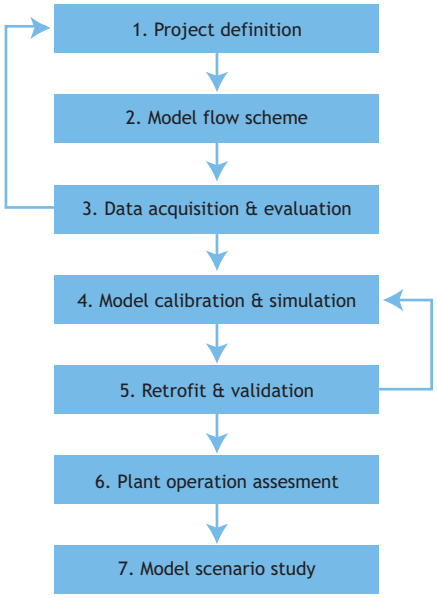

Figure 0.2 General structure of the guidelines showing the 7 phases of modelling an activated sludge plant

In the protocol each step is addressed in a standard, practical, feasible and (time) realistic manner. The method was demonstrated in this project in a practical Croatian context for five

WWTP, namely: Varaždin, Čakovec, Vinkovci, Zagreb and Belišće. The guidelines present the practical and applied methodology in a generic and explanatory way. The aim is for them to be used as a practical manual and check list for new design projects, thereby providing a quality standard for modelling design studies in the future development of wastewater treatment in Croatia towards the implementation of EU regulations and improved environmental safety standards.

The proven key factor for the successful introduction of model-based wastewater treatment design is a practical standardization of the methods, models and the simulation software. These guidelines are the foundation for a common approach and application in wastewater treatment; they have been put into practice in a Croatian context and thereby further developed into a more tailor-made practical document. Emphasis was given in the protocol to the local conditions and, where needed, translations were made of key technical documentation published elsewhere. The guidelines provide a knowledge base and practical advice, which is useful for project management, logistical organization, quality control and technological development. The guidelines provide managers with a rapid project overview and guidelines for quality control, consultants with a knowledge base for design projects and developments, and modellers, operators and laboratory personnel with practical 'hands on' advice on how to improve their operations towards modern standards in the context of wastewater treatment model design. Based on the information presented in the guidelines, the training program started in 2011 on Croatia-wide capacity building regarding the wastewater modelling for which the foundations were laid down by this project.

The structure of the guidelines is a seven-step generic approach in preparation for a location-specific wastewater treatment design study. The first step is the inception phase where the main goal is getting a complete project overview. It contains the visits to the plant location, collecting all kinds of information based on the checklists provided in the guidelines. The results of the inception phase are a definition of the project objectives and realistic planning of the available time and resources. The estimated invested time for this phase is 4 to 5 days including plant visits and writing a short report on the conclusions.

Following the inception phase, the modelling work is carried out. In a desk study of an estimated 1 to 2 days using the preliminary collected information, an initial model is constructed and the first simulations take place. Thereby, an early overview of the project is obtained and the modelling requirements can be overseen, resulting in a more detailed specification of the project objectives as well as of time and resources planning. Also for the modeller who already has hands-on experience with (general) modelling software, this stage provides the possibility to refresh his/her modelling skills through further training.

From the initial desk study, the focus then shifts to acquiring the operational and design plant data. Therefore, a comprehensive sampling and measurement plan is designed which should be communicated to all personnel involved during a new (or repeated) plant visit(s). Again, the guidelines here serve as a checklist for all the steps to be taken in the preparation of data acquisition. At this stage of the project, substantial financial resources are used for sampling, analysis and personnel time investment. Therefore efficient planning will definitely save some cost. At the same time, additional data acquisition will ensure that all the information necessary to bring the projects to a successful completion (to get reliable results) is collected. To obtain plant measurements, an estimated 2 to 3 weeks are needed to cover the complete procedure of sampling and analytical work. When this data is made available, the project can proceed, and therefore, this project stage is critical for the time planning. The collected information is then

checked for easily observable errors and (in)consistencies. Because actual (real) plant measurements are used, errors will frequently occur. Therefore, the protocol provides a set of necessary steps to check and correct the collected plant information, thereby providing the highest reasonably possible accuracy in the model results. The data acquisition phase is finalized by drafting a report presenting all the raw and corrected plant information in a single tabular overview. This table serves as the starting point for the model simulations and is also a reliable reference regarding the operational performance of the plant as it is based on measured (observed) information. The total time investment in this phase depends on the amount of data collected, and is estimated for a small project at 3 weeks and for a larger project at 4 weeks (including preparation and practical execution).

In the model calibration and simulation phase that follows, the guidelines focus on the more technical aspects of the activated sludge modelling protocol. A step-by-step explaination of how the model should be fitted to the measured data (reality) is given. In scientific literature the model calibration is often indicated as a complicated and time-consuming step in the model application. However nowadays these conclusions are considered outdated as the presented guidelines recommend a methodology for calibration that is not only practical and easy to apply, but also results in a highly accurate model description. By using these guidelines, a modeller can obtain a fully calibrated model within 2 to 5 days, depending on the experience of the modeller. For the novice modeller it is advised to reserve 2 to 3 additional weeks, so that the user (modeller to be) can get acquainted with the software, the background theory of ASM models, and model calibration. If possible, he/she should follow specialist training on this topic.

The model simulations are followed by a traditional design phase where all the results from the model and plant measurements are calculated and reported, leading towards a traditional wastewater treatment plant design. Thereby, state-of-the-art wastewater treatment modelling is embedded in the traditional design practice where model-based design and traditional design are compatible. This next step in the guidelines provides the tools to present the modelling results in such a way that others (without modelling skills) can understand and judge them. Thus, wastewater treatment modelling obviously supports the traditional design and operation of WWTP. Templates of the summary tables are presented in the guidelines to compare the modelling results to the measured data and thereby the model is validated for use in a scenario study analysis. Preparing the traditional WWTP design and validating the activated sludge model is a desk study that will take approximately 2 to 5 working days.

In the concluding phase a final operational plant assessment is made and disseminated. Interpreting the design data requires expert judgment. Drawing conclusions based on the plant performance and the results of the model application will take approximately 1 to 2 days, depending on the experience of the designer. Also the model design accuracy is evaluated and design safety factors are introduced. The accuracy of the model predictions can be determined from the differences between the evaluated measured plant data and the results obtained by the calibrated model. A valuable aspect of modelling is the statistical accuracy that is available regarding the plant design and which is the basis for the determination of more realistic safety factors. From the financial perspective, this is very relevant - safety incorporated into design procedures directly increases the building and operational cost of wastewater treatment and therefore the determination of realistic and acceptable safety factors often deliver financial benefit.

From this point on, the model can demonstrate its added value by using it for various types of extrapolation, e.g. simulating conditions that might not (yet) exist. These are often called ‘what-

if scenario' studies. The model can be used to check how the plant will perform under full load (design capacity), in other words, whether the plant will indeed be capable of reaching the designed effluent standards under full load. If the WWTP is already overloaded, the model can be used to alter the design and extend the plant (volumes and capacities) to test the effect of an upgrade or retrofit on the effluent quality. Simulations may also reveal the possibility for operational improvements, savings (energy and chemicals), and improvement in efficiency in terms of effluent quality, biogas production, sludge minimization, and aeration requirements. The power (possibility) of performing realistic (dynamic and static) scenario studies is a unique feature of the modelling approach; there is no other technique/methodology by which these sorts of predictions can be carried out within the given margin of accuracy. On the other hand, these results can only be achieved if all the recommended steps are systematically and accurately followed according to the guidelines, including the modelling protocol.

Overall, the guidelines describe the methodology simplified in the generic flowchart shown below.

The authors hope that you will find these guidelines useful and wish you good luck in using them!

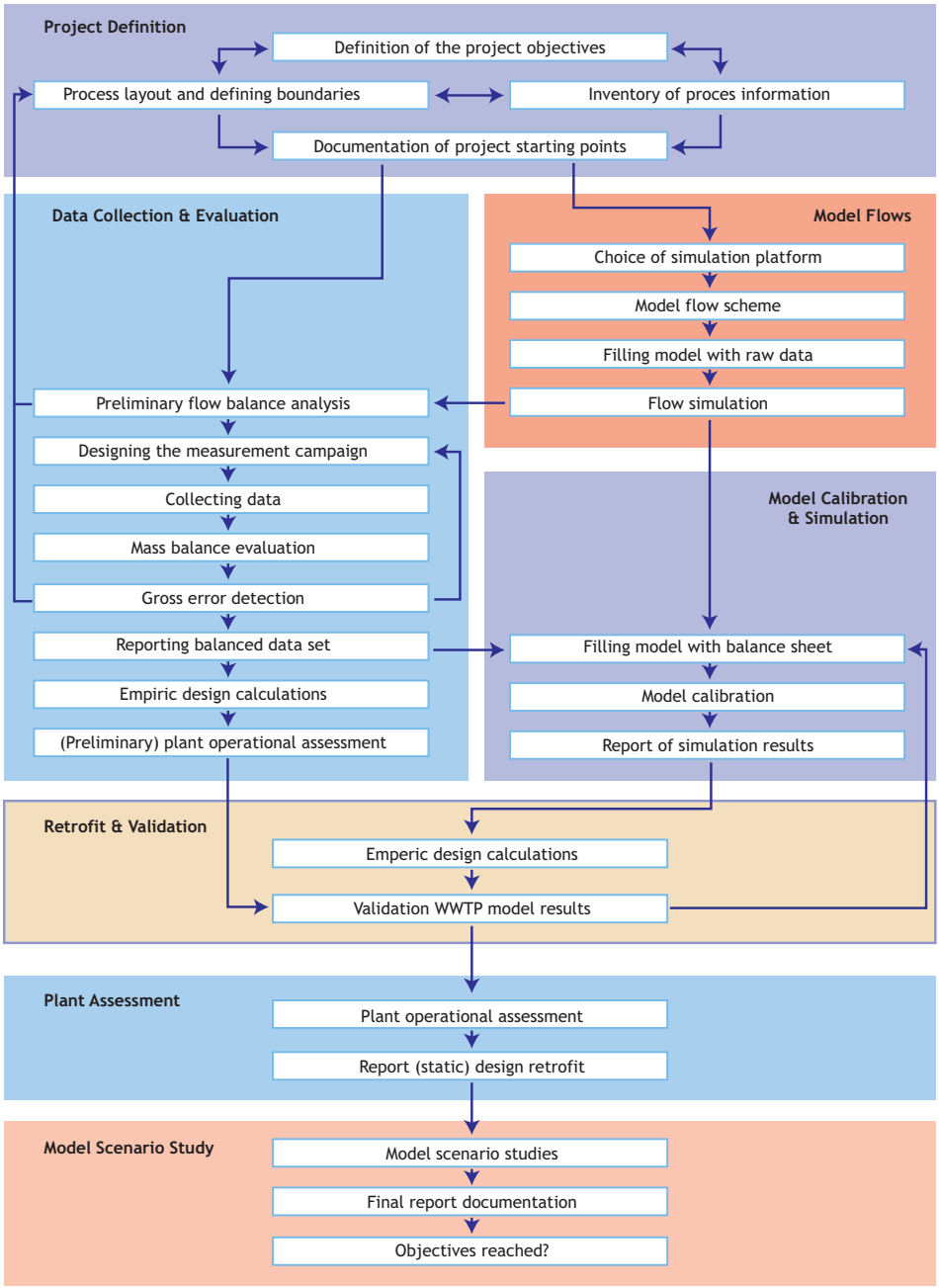

Figure 0.3 Methodology applied to activated sludge modelling

Chapter 1

Wastewater management in Croatia

1.1 Institutional framework

The Ministry of Regional Development, Forestry and Water Management has statutory authority in water management, which also includes water protection. Water management also falls under the competencies of the Ministry of Environmental Protection, Physical Planning and Construction. Within these ministries, there are units for inspection supervision.

The Water Act[1] established the authorities of Hrvatske vode (Croatian Waters) as a legal body for water management which, among other responsibilities, prepares programmes, plans and other acts for protection of water from pollution, and also conducts the tasks by which these plans are implemented. Hrvatske vode is assigned to collect the water protection fee that is used for water protection and the execution of related expert tasks and especially, for development of infrastructure for water protection in the form of financial support to units of local self-government.

Wastewater collection and treatment in Croatia is currently dominated by the public sector (a total of 127 municipal utilities, utility companies and own plants of units of local self-government) with one exception – the concession for wastewater treatment plant (WWTP) in Zagreb (the largest plant in Croatia with capacity of 1.2 million PE[2]). Units of local self-government competent for utility services decide on the programme of construction and programme of maintenance of facilities and elements of utility infrastructure. Each utility service provider (municipal utility, utility company or concession holder) adopts its own plan of development and maintenance, which then must be harmonized with the programme of the local self-government.

For the purposes of water management, the territory of the Republic of Croatia is divided into six distinctive river basin districts (RBDs) according to the new legislations[3]: (*i*) Mura and upper Drava RBD, (*ii*) Danube and lower Drava RBD, (*iii*) Upper Sava RBD, (*iv*) Middle and lower Sava RBD (*v*) North Adriatic RBD and, (*vi*) South Adriatic RBD, and 34 catchment areas. These guidelines are prepared under the framework of Inland Water Project that concerns the first four RBDs which ultimately make part of the Black Sea Basin. The last two RBDs fall under the Adriatic Basin. The new Water Act divides territory of the Republic of Croatia in two river basins, Black Sea Basin and Adriatic Basin.

[1] The Water Act, *Official Gazette* No. 153/09
[2] PE stands for Person Equivalent
[3] The Water Act, *Official Gazette* No. 153/09, The Decision of RBD borders, *Official Gazette* No. 79/10, The Ordinance on the boundaries of the area under the basin, small basin and sectors, *Official Gazette* No. 97/10

1.2 EU and Croatian effluent discharge legislative

In 2004, the Republic of Croatia became a candidate for membership in the European Union, by which it accepted the obligation of harmonizing the national legislation with the *acquis communautaire*. The negotiations on chapter 27. Environment started in February, and ended in December 2010. At the thirteenth meeting of the Intergovernmental Conference on Accession of the Republic of Croatia to the European Union held in Brussels on 30 June, 2011, all the 35 chapters were closed, and the negotiations on accession of the Republic of Croatia to the European Union were formally finished.

Council Directive 91/271/EEC concerning urban waste water treatment was adopted on 21 May 1991 to protect the water environment from the adverse effects of discharges of urban waste water and from certain industrial discharges. On 27 February 1998 the Commission issued Directive 98/15/EC amending Directive 91/271/EEC to clarify the requirements of the Directive in relation to discharges from urban waste water treatment plants to sensitive areas which are subject to eutrophication. This had the effect of amending Table 2 of Annex I (Table 1.2). Commission Decision 93/481/EEC was adopted on 28 July 1993 and defines the information that Member States should provide the Commission when reporting on the state of implementation of the Directive according to Article 17, and specifies the format in which the information should be provided. This Decision was adopted in accordance with Article 18 of the Directive.

Harmonization of water legislation with the EU *acquis* began in 2005 by the amendments to the Water Act and Act on Financing Water Management and, at the end of 2009, Water Act and Act on Financing Water Management, completely in accordance with the EU requirements, were adopted. By adopting these acts, preconditions for passing of the implementation regulations from the field of water quality were made. The Water Act (153/09) and the Ordinance on Limit Wastewater Emissions Values (The Official Gazette 87/10) introduced the Urban Waste Water Directive (Directive 91/271/EEZ) into the Croatian legislation. According to the Urban Waste Water Directive, collection and adequate treatment of wastewater is obligatory for all agglomerations in excess to 2.000 PE (294 of such in Croatia). Only 4 agglomerations are larger than 150,000 PE, 85 agglomerations are in the size of 10,000-150,000 PE, and 205 agglomerations are small agglomerations, with the size of 2,000-10,000 PE. Agglomeration sizes are based on the data from the 2001 census, and they will be changed in accordance with the data from the 2011 census. The level of wastewater treatment and the term in which a wastewater treatment plant should be built depend on the size of the agglomeration and the sensitivity of the area into which wastewater is discharged. In June 2010, the Decision on the Designation of Sensitive Areas (The Official Gazette 81/10) was made. According to the recommendation of the European Commission, the whole Danube river basin district has been proclaimed a sensitive area due to eutrophication. The land part of the Adriatic Basin has also been proclaimed a sensitive area due to specific features of karstic areas (Figure 1.1). For agglomerations larger than 10,000 PE in ecologically sensitive areas, the tertiary level of treatment should be implemented, i.e. along with removal of the organic load, the compounds with nitrogen and phosphorus should also be removed. There are 54 such agglomerations in Croatia. For complete implementation of this directive, Croatia requested a transition period until the end of 2023 so that the works on construction of the wastewater drainage and treatment would be finished. It was estimated that over 23 billion kuna (EUR 3.15 billion) will be necessary for implementation. It will be possible to finance a large part of the expenses for achieving the European standards in the field of water protection through structural funds intended for member countries of the European Union.

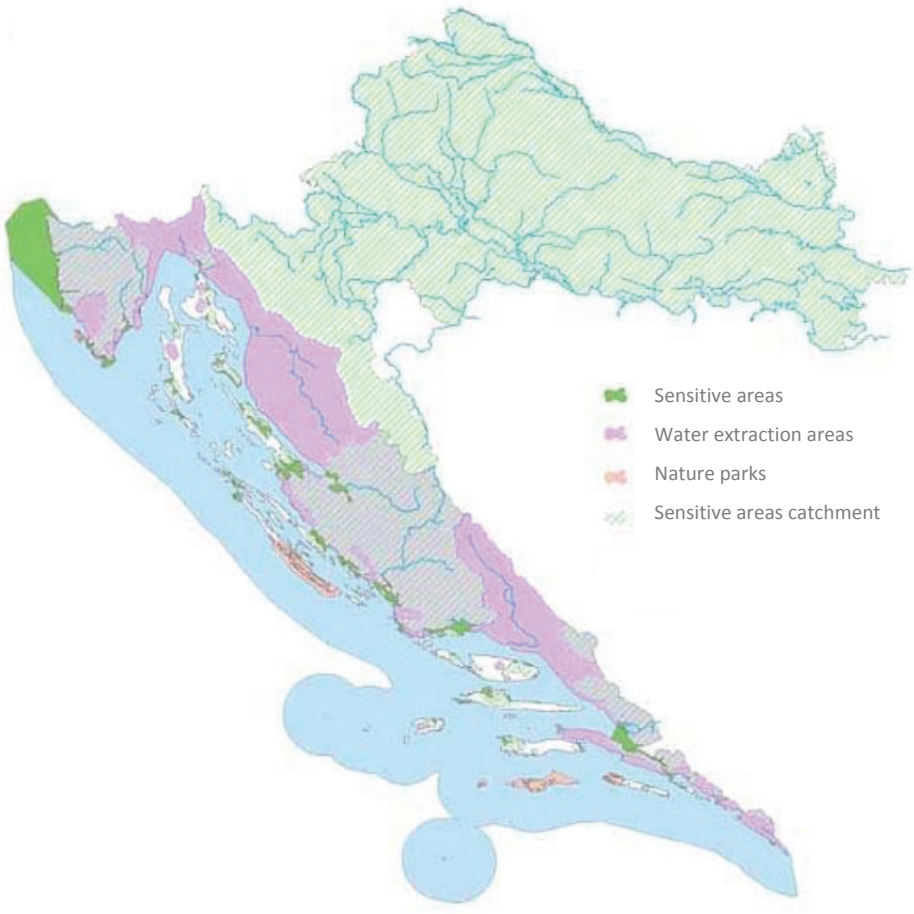

Figure 1.1: Map of Croatia with marked area sensitivity (The Official Gazette 81/10)

The Ordinance on Limit Wastewater Emissions Values (The Official Gazette 87/10) prescribes the limit values of emissions in industrial and municipal wastewater, before they are discharged into public sewerage structures or into septic tanks or sump pits, as well as in all the treated or untreated wastewater that is discharged into water. Table 1.1 and Table 1.2 present the values which must be met by treated wastewater before it is discharged into a natural recipient.

Sewage sludge which originated from the wastewater purification process is deposited and/or used in accordance with special environment protection regulations. In accordance with that, the Ordinance on Management of Sewage Sludge When Used in Agriculture (The Official Gazette 38/08) was passed. The Ordinance was harmonized with the Sewage Sludge Directive 86/278/EEZ. Other ways of sludge management have not been completely legally stipulated.

Table 1.1 Limit values for wastewater subject to secondary treatment

Indicator	Limit value	Lowest % of load reduction
Suspended Solids (SS)	35 mg/L (>10,000 PE)	90
	60 mg/L (2,000 to10,000 PE)	70
Biochemical Oxygen Demand (BOD_5)	25 mgO_2/L	70–90
Chemical Oxygen Demand (COD_{Cr})	125 mgO_2/L	75

Table 1.2 Limit values for wastewater subject to tertiary treatment

Indicator	Limit value	Lowest % of load reduction
Total Phosphorus (TP)	2 mgP/L (10,000 – 100,000 PE)	80
	1 mgP/L (>100,000 PE)	
Total Nitrogen (TN)	15 mgN/L (10,000 – 100,000 PE)	70–80
	10 mgN/L (>100,000 PE)	

1.3 Wastewater collection and treatment overview

In Croatia, 43% of the population (approximately 2 million inhabitants) is connected to a sewerage system (Table 1.3). The collected urban wastewater of 28% of the population (approximately 1.25 million inhabitants) is treated at WWTP (Table 1.4). In comparison with most of the EU member states, Croatia falls behind, which is illustrated by Figure 1.2.

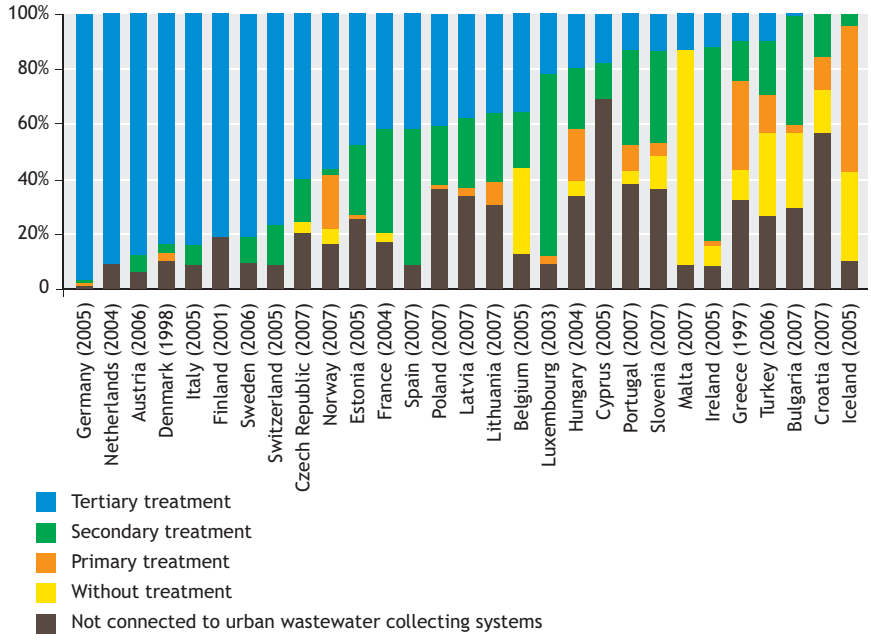

Figure 1.2 Comparison of wastewater treatment coverage in Europe[4] and Croatia

[4] http://epp.eurostat.ec.europa.eu

From Table 1.3 it can be seen that in settlements with more than 50,000 inhabitants the connection of population to public sewerage systems is 81% while in settlements with less than 2,000 inhabitants this comes not higher than 6%.

Table 1.3 Connection rate of Croatian population to public sewerage systems in 2007[5]

Settlement size	Population				
	Connected		Not connected		Total
Inhabitants		%		%	
< 2,000	111,717	6	1,640,828	94	1,752,545
2,000 – 10,000	267,602	37	455,883	63	723,485
10,000 – 50,000	501,527	74	172,036	26	673,563
> 50,000	1,042,126	81	245,741	19	1,287,867
Total	**1,922,972**	**43**	**2,514,488**	**57**	**4,437,460**

Comparing the connection rate to a public sewerage system by region, the highest connection rate is in the area of the Primorje-Istria RBD and amounts to 58%, and the lowest is in the Drava and Danube RBD and amounts to 28% (Table 1.4).

Table 1.4 Urban wastewater collection and treatment systems per RBD in 2007[4]

RBD	Inhabitants	Inhabitants connected to the sewerage system	Inhabitants connected to WWTP with different level of treatment				
			PL	PR	SC	TR	TO
		%					
Sava RBD	2,211,902	46.6	0.0	3.4	25.4	0.0	28.8
Drava and Danube RBD	833,927	28.0	0.0	1.9	8.2	1.5	11.6
Black Sea Basin	3,045,829	41.5	0.0	3.0	20.6	0.4	24
Primorje-Istria RBD	536,854	58.0	43.1	0.4	5.5	0.0	49.0
Dalmatia RBD	854,777	31.0	11.6	17.2	0.5	0.0	29.3
Adriatic Basin	1,391,631	47.2	23.7	10.7	2.4	0.0	36.8
Total Croatia	**4,437,460**	**43.3**	**7.4**	**5.4**	**14.9**	**0.3**	**28.0**

Public sewerage systems in the Croatian part of the Black Sea Basin are mostly constructed as mixed, (i.e. collecting both wastewater and rainwater), while the recently constructed sections are mostly separate. In the Adriatic Basin, separate sewage systems (i.e. collecting wastewater and rainwater separately) are more frequently in place. On the level of the Republic of Croatia, 50% of public sewerage systems are mixed, 41% separate, and 9% combined[3]. Due to aging many of the public sewerage systems are not watertight, and therefore require reconstruction.

[5] Water Management Strategy, *Official Gazette* No. 91/08

Wherever possible, rainwater is separated from wastewater, and new separate systems are constructed as this brings advantages in treating more concentrated wastewater.

Out of the total quantity of urban wastewater treated in Croatia, 46% undergoes preliminary treatment (removal of larger fractions and sand from sewage) and primary treatment (removal of suspended solids), 53% undergoes secondary treatment (biological treatment targeting organic compounds). Less than 1% undergoes tertiary treatment (removal of nutrients such as nitrogen and phosphorus)[6].

The largest number of existing WWTP is in the tourist areas of the Primorje-Istria RBD, namely in Istria and the Kvarner Bay, and the lowest in the areas of the Drava and Danube RBD (Table 1.4). Currently, there is only one WWTP with tertiary treatment in Croatia, which is located in Koprivnica and has the capacity of 100,000 PE (Figure 1.3).

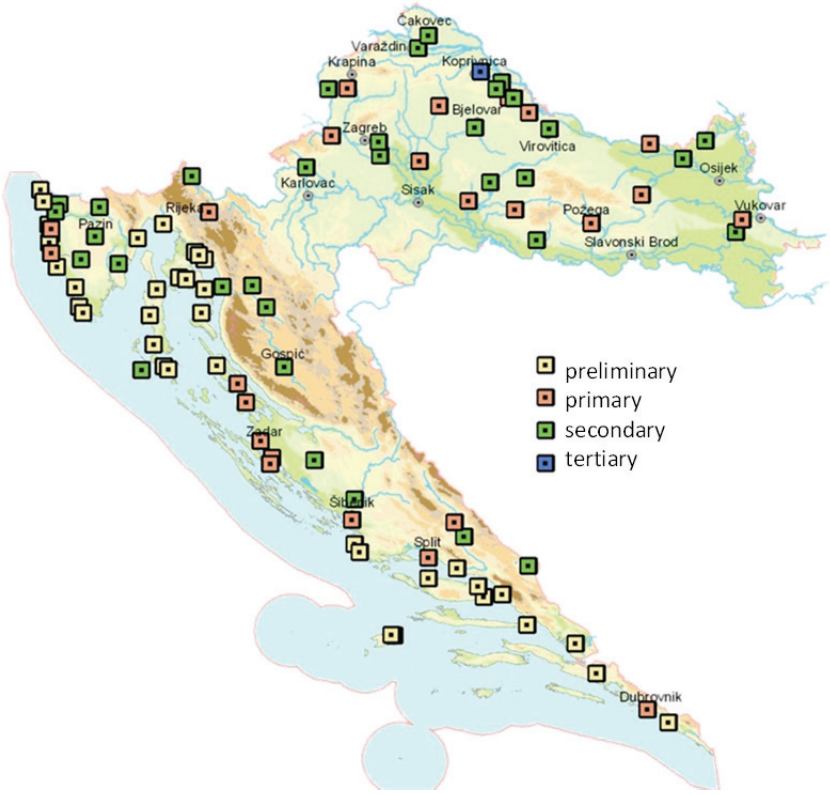

Figure 1.3 Map of Croatia with locations of WWTP larger than 1,000 PE

[6] Water Management Strategy, *Official Gazette* No. 91/08

1.4 Efficiency of existing WWTP

Most of the plants built in the 80's have inadequate capacity, because the planning foresaw a large increase in the population and industrial production, which has not occurred. Equipment employed at treatment plants is old and often malfunction. Also, there is constant lack of skilled operators.

In current practice often the efficiency of the plant operation cannot be evaluated because there is no protocol in place for sampling and monitoring the plant operation and therefore, the evaluation, based on routinely collected data is often unreliable and could lead to incorrect conclusions. The new regulations[7], adopted in 2010, prescribe the obligation for wastewater sampling at WWTP as flow proportional 24-hour composite samples at a permanent location at the plant inlet (influent) and outlet (effluent) in order to determine the achieved load reduction. Since the manner and frequency of sampling are prescribed in the water rights permits, it is necessary to issue new water rights permits harmonized with the provisions of the new regulations as soon as possible, at least within the next 5 years.

In general, lack of specific knowledge of plant management can be noted, especially of biological (activated sludge and nutrient removing) processes. Since there is no academic programme available in Croatia where all aspects of wastewater treatment plants and their management are taught, an improvement in the communication between the universities and plant managers would result in bilateral transfer of knowledge and experience and a positive influence on the capacities of plant managers and students. Therefore, there is a need for continuous professional training of plant managers and mutual exchange of their experiences.

1.5 Industrial wastewater

In Croatia, industrial wastewater must be treated prior to discharge into public sewerage systems or natural recipients. Currently, this is the case with 77% of wastewater discharged into public sewerage systems and 66% of wastewater discharged into natural recipients. Treatment prior to discharge into public sewerage systems includes treatment to the level of quality of urban wastewater, and prior to discharge into natural recipients to the level prescribed by the above mentioned regulations.

For certain types of industry, regulations in accordance with the Council Directive 96/61/EC and 2008/1/EC concerning integrated pollution prevention and control, prescribe characteristic parameters and limit values for industrial wastewater discharged on public sewage systems. The conditions of wastewater discharge, quantities, maximum allowed values of specific effluent parameters, and the recipient are prescribed by water rights permits. Obligatory wastewater testing prior to discharge is also prescribed as control of compliance with the discharge conditions. For industrial wastewater, the regulations allow prescribing composite samples, if necessary. Both current and composite wastewater samples must be taken during the technological process. Analyses are conducted by authorised laboratories and the holder of the water rights permit has the obligation of sending the test results to Hrvatske vode and the competent water rights inspection. Based on wastewater analyses results, industrial discharges are currently monitored at approximately 1.500 locations in Croatia. Based on effluent quantity and quality and on the type of recipient in each case the water protection fee is charged by Hrvatske vode.

[7]The Ordinance on Wastewater Emission Limit Values, *Official Gazette* No. 87/10

1.6 Sludge

Currently, most sludge generated as the result of operation of (biological) wastewater treatment plants is stabilized in lagoons within the plant, after which it is disposed at landfills. The exceptions are the two larger plants; in Zagreb sludge undergoes anaerobic digestion, and in Koprivnica sludge is solidified. Use of sludge in agriculture is regulated as well[8]. This ordinance prohibits the use of treated sludge in karstic areas, shallow or skeletal karstic soils, and in the coastal areas and zones of sanitary protection of water extraction wells.

1.7 Investments

The required funds for wastewater management and infrastructure shall be raised from several sources including water protection fee, state budget, budgets of units of local self-government, funds of municipal utilities, loans of international financing institutions and EU funds. Since 2007, Croatia has access to the Instrument for Pre-accession Assistance with a budget of €53.5 million for environmental protection; with the accession to the EU in 2013., Croatia will gain access to Structural Funds and Cohesion Funds (Table 1.5).

Table 1.5 Estimated dynamics of investments into wastewater collection and treatment

Area/PE	2,000 – 10,000	10,000 – 15,000	15,000 – 150,000	>150,000
Sensitive areas	end 2030 secondary or other	end 2018 tertiary	end 2018 tertiary	end 2018 tertiary
Normal areas	end 2030 secondary or other	end 2030 secondary	end 2023 secondary	end 2023 secondary
Less sensitive areas	end 2030 other	end 2030 primary or secondary	end 2023 secondary	end 2023 secondary

The developmental priorities defined in the Water Management Strategy[9] are systems:

- According to size, with regard to the existing and planned loads (population and industry connected to the public sewerage system);
- That will achieve full functionality from the connection, collection, sewerage and treatment to adequate discharge of treated wastewater while complying with the technical-sanitary conditions of service provision (water-tightness, overflows, temporary sludge disposal etc.);
- In areas where deterioration of water quality took place (surface-, coastal- and ground-water);
- In areas that are considered to be at risk due to the lack of constructed public sewerage systems;
- In catchment areas whose intake capacities are exposed to combined pressures of several sources of pollution;
- Whose construction achieves balanced development of utility infrastructure and sanitary conditions for the life of population in the national territory.

[8] Ordinance on Management of Wastewater Treatment Sludge when used in Agriculture, Official Gazette No. 38/08

[9] Water Management Strategy, *Official Gazette* No. 91/08

1.8 National strategic projects

In Croatia, two national strategic projects are implemented in the field of water protection: the Coastal Cities Pollution Control Project (The Adriatic Project) in the Dalmatian Basin, and the Inland Waters Project in the Black Sea Basin.

The objective of the national Adriatic Project is protection of transitional and coastal waters from pollution by construction, rehabilitation and expansion of urban wastewater collection and treatment systems and construction of wastewater treatment plants with marine outfalls. The estimated duration of the project is 10 years and implementation costs €280 million. The project is co-financed by World Bank loan proceeds, in the share not exceeding 50%, while other costs are covered by units of local self-government (19%), Hrvatske vode (9%), and state budget of the Republic of Croatia (22%). The loan received from the International Bank for Reconstruction and Development (IBRD) shall be repaid by the final beneficiaries of the loan – municipal utilities (through the increased water tariff) and Hrvatske vode**.**

The project shall be implemented in three phases in numerous settlements along the Adriatic coast. Phase one of the project (2005-2008) is finished. Phase two of the project is in the progress.

Prioritization of the systems was performed by the application of weight distribution on the basis of the following criteria:

- Population equivalent (criterion that reflects the quantity of pollution);
- Recipient sensitivity to wastewater discharge (criterion of area sensitivity);
- Impact on development of tourism (impact of system completion on tourist capacities);
- Level of system completion (priority is given to projects that enable completion of systems so they could achieve full functionality);
- Level of preparation of project documents (priority is given to projects with documents at a higher level of detail);
- Area development (criterion that gives priority to less developed areas);
- Size of investment in relation to number of population;
- Size of investment into phase one of construction in relation to number of population.

The objectives of the national Inland Waters Project is, aside from water protection, also development of flood protection systems and public water supply systems in the Sava and Drava and Danube RBDs. The project has two components, namely Technical Assistance and Investments. The total value of the project is estimated at €105 million, and is co-financed by World Bank funds. The Loan amount is €100 million, and Hrvatske vode shall ensure remaining €5 million. According to the decision of the Croatian Government, loan repayment shall be financed from: (i) state budget funds up to 50% of the total loan, (ii) direct revenue of Hrvatske vode in the amount of 25%, and (iii) funds of final beneficiaries in the amount of 5-30%, depending on the financial capacities of units of local self-government participating in the project. The duration of the project is estimated at 4 years (until the year 2012). With the objective of water protection, the project includes the development of urban wastewater collection and treatment systems in Ogulin, Virovitica, Našice, Vukovar, Ilok, Ivankovo, Cerna, Otok and Komletinci.

Depending on area sensitivity, the WWTP planned to be constructed or currently under construction equipped with secondary and some are extended to tertiary treatment. In order to

achieve secondary treatment, biological treatment systems are applied - mostly conventional systems with active sludge (CAS) and activated sludge sequencing batch reactor (SBR) systems.

In Croatia, there are specific cases when treated wastewater is discharged into karstic water bodies, and there is risk of intensive infiltration into the underground. In these areas it is necessary to achieve high effluent quality, and therefore membrane bioreactor (MBR) plants are also increasingly considered and applied (Istria).

Under the framework of the Inland Waters Project the project entitled Development and Application of Models and Guidelines to Facilitate Decision making in the Extension of Wastewater Treatment Plants (WWTP) and to Increase Operational Efficiency of Existing WWTP included an inventory of 26 WWTP from which 9 largest plants were visited (Table 1.6 and Figure 1.4) and some general observations are listed below:

Table 1.6. Municipal wastewater treatment plants in the Republic of Croatia that belong to the Black Sea Basin and have the capacity over 2,000 PE (source: Hrvatske vode)

WWTP	Capacity (PE)	Level of treatment
1 Koprivnica	100,000	Tertiary
2 Zagreb	1,200,000	Secondary
3 Virovitica	421,200	Secondary
4 Belišće	240,000	Secondary
5 Varaždin	140,000	Secondary
6 Čakovec	91,000	Secondary
7 Bjelovar	50,000	Secondary
8 Vinkovci	43,000	Secondary
9 Velika Gorica	35,000	Secondary
10 Daruvar	23,000	Secondary
11 Samobor	20,000	Secondary
12 Beli Manastir	8,000	Secondary
13 Garešnica	7,000	Secondary
14 Topusko	6,000	Secondary
15 Đurđevac	4,000	Secondary
16 Kumrovec	3,000	Secondary
17 Zaprešić	61,000	Primary
18 Požega	33,500	Primary
19 Kutina	25,000	Primary
20 Križevci	25,000	Primary
21 Našice	20,000	Primary
22 Ivanić Grad	20,000	Primary
23 Pakrac-Lipik	20,000	Primary
24 Pitomača	11,500	Primary
25 Donji Miholjac	8,700	Primary
26 Krapinske Toplice	6,500	Primary



Figure 1.4 Location of municipal wastewater treatment plants in the Republic of Croatia that belong to the Black Sea Basin and have the capacity over 2,000 PE (source: Hrvatske vode)

- In the Black Sea Basin in the Republic of Croatia, 26 plants with the capacity over 2,000 PE have been built until now. Two of these plants are not functional (Samobor and Topusko), while another eleven from the 2,000-10,000 PE plant category and one from the >10,000 PE plant category (Velika Gorica) do not comply with the conditions prescribed by the Ordinance on Wastewater Emission Limit Values (The Official Gazette, number: 87/10), and one has a special temporary status (Bjelovar[10]).
- Some of the larger plants are already partially or completely outdated (e.g. Velika Gorica and Daruvar) and would have been written off already and replaced by new ones according to foreign standards; the average age of 26 observed WWTP is approximately 19 years (Figure 1.5).
- In most of the cities, the type of sewerage is the combined one, with the exception of rare parts that were built as distribution systems. In some cases, where the distribution systems were designed, there are problems in practice (sanitary sewerage connected to the rain drainage, and vice versa), as well as illegal connections which transform the distribution systems into partially mixed ones. Inflow of external water, particularly of infiltration, has been mentioned as a problem in almost every plant, which is often an indicator of (serious) shortcomings in the drainage system.
- In general, one gets an impression that, in Croatia, the problem issue of precipitation water and its influence on WWTP has not been given sufficient attention by practice, particularly that of some time back, because excessive inflows into the plants are frequent even where stormwater retention basins exist, or they are insufficient by their volume or

[10] For WWTP Bjelovar, the Water Permit valid until 27 October, 2014, allows higher values for KPK, BPK_5 and suspended material than the limit values prescribed by the Ordinance.

have not been designed well. Also, the function of stormwater overflows and their effect on the environment has not been sufficiently elaborated in the documentation that was available to consultants (this statement does not refer to CWWTP Zagreb). Therefore, a wider implementation of modern methods and aids is recommended, which include mathematical models for designing of WWTP and optimization of the drainage system in the Republic of Croatia, not only for large, but also for smaller settlements.

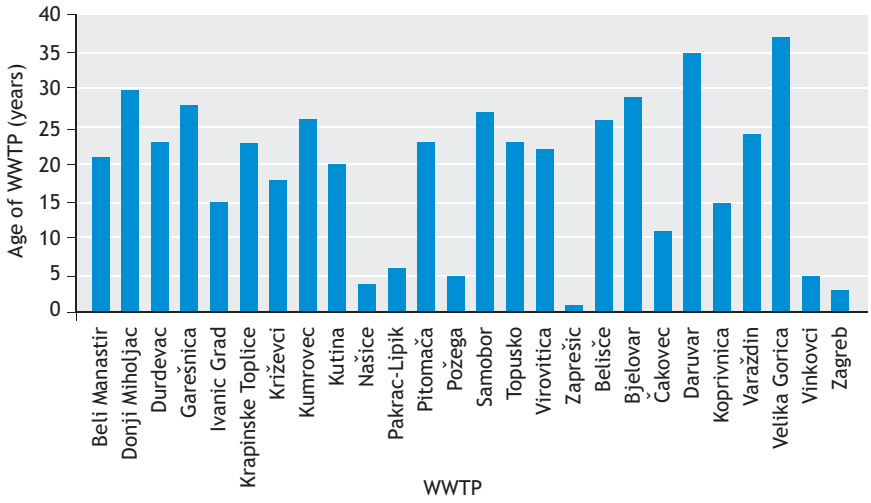

Figure 1.5 Years of age of WWTP in the Republic of Croatia belonging to the Black Sea Basin with the capacity > 2,000 ES

- In the Republic of Croatia, the dominant is WWTP Zagreb, the Zagreb central plant, which is one of the largest and, both, by the equipment, and the execution, the most modern in Southeast Europe. The plant implements traditional process technology which includes biological removal of nitrogen and biological-chemical removal of phosphorus, which are becoming a standard practice in new and many accession member countries of EU. This is important because it is a fact that both, the present, and the designed capacities of the Zagreb WWTP are approximately close to the sum of the same of all the other WWTP in the Black Sea Basin in the Republic of Croatia. Due to its size and contribution to the total number of PE comprised by treatment in the Republic of Croatia, WWTP Zagreb has a crucial influence on all the executed collective statistical analyses and data at the level of the state, because it contributes to the overall installed WWTP capacity in the Black Sea basin with 50% (expressed in PE), i.e. it receives 65% of the total quantity of treated wastewater in this basin, and, for example, contributes to removal of organic pollution (BPK_5) with 62% at the same time. Therefore, it is recommended that any more serious data analysis takes in consideration the situation with and without CWWTP Zagreb since, on the contrary, the same may lead to wrong conclusions.
- On the basis of the mostly collected and a smaller part of the adopted data, the following rough assessments can be made:

The total designed and existing capacity of WWTP in the Republic of Croatia belonging to the Black Sea Basin is approximately 2.4 and 1.7 million PE, respectively (Figure 1.6);

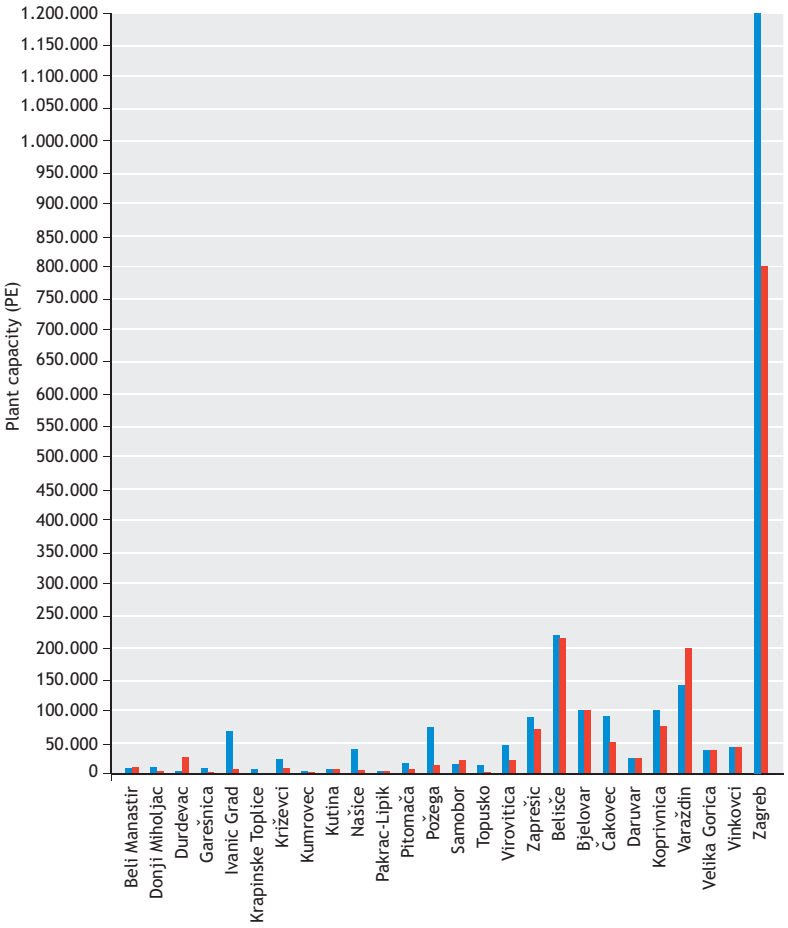

Figure 1.6 Designed and existing capacity (PE) of WWTP in the Republic of Croatia belonging to the Black Sea Basin with the capacity over 2,000 PE (data mostly from 2009)

The total influent to WWTP in the Republic of Croatia belonging to the Black Sea Basin is approximately 155 million m^3 annually (Figure 1.7), while the installed hydraulic capacity is higher by approximately 20%;

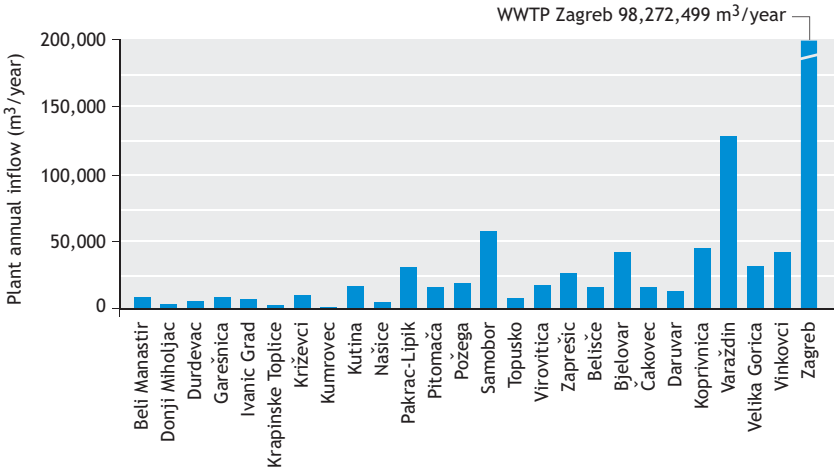

Figure 1.7 Existing annual inflow (m^3) to WWTP in the Republic of Croatia belonging to the Black Sea Basin with the capacity over 2,000 PE (data mainly from 2009)

The total annual quantity of the organic matter (BPK_5[11], KPK), suspended material (ST), and nitrogen (TN) and phosphorus (TP) which was removed in WWTP in the Republic of Croatia, which belong to the Black Sea Basin, is given in the following table:

Table 1.7 The total average annual quantity of main indicators of wastewater pollution removed in WWTP in the Republic of Croatia belonging to the Black Sea Basin, and their efficiency in relation to the total wastewater load (calculated on the basis of the actual data from 2009 and earlier years, and the adopted hypotheses)

Parameter	BOD_5	COD	N	P	SS
Quantity	34,306 tons/year	64,596 tons/year	2,523 tons/year	496 tons/year	31,252 tons/year
Efficiency	89 %	86 %	41 %	55 %	88 %

- Industrial wastewater is a problem everywhere in the world, and Croatia is not an exception either. Most of the 9 visited WWTP have noticed the problems due to discharging of polluted or insufficiently treated industrial effluents, depending on whether the industries do or do not have their pre-treatment, and even if they do, the question is whether the same works; a special problem is sudden discharging of the effluent with very high concentrations of the organic and suspended material, and sometimes also of nitrogen or phosphorous compounds, which has a very negative effect on plants operation.
- In general, one gets an impression that the designing practice does not give the necessary attention to detailed characterization of raw wastewater by use of modern approaches (protocols), which is the consequence of rare application of mathematical modelling in designing, that "pushes" the designers to a more detailed, systematic and analytical

[11] *Translator's remark: abbreviation from Croatian "Biološka potrošnja kisika", i.e. "Biological Oxygen Consumption"*

approach to determining the parameters of quality of the inlet water into the plant. On several plants, a lack of an integrated approach to the problem of collection and treatment of wastewater was noticed, which was manifested on the example where wastewater with insufficient easily degradable organic load reaches the plant, while several major industries in the system 'eliminate' such a load by pre-treatment, instead of considering the response option with the industry so that it would be allowed to discharge the "pollutions" in such cases, in order to stimulate the desired processes on the plant in that way.

- All the 9 plants with a larger capacity, which have at least the level II treatment, have a different technology designed by various consulting companies, and use different equipment, and each has its own 'philosophy' of approach to water treatment. The applied technologies are based on different variants of the activated sludge system, and they include traditional oxidation channels, with or without primary sedimentation (Čakovec, Daruvar), a modern conventional system (Zagreb, Vinkovci), various variants of the two-stage biological (AB) system (Belišće, Bjelovar, Varaždin), a SBR system (Koprivnica), and the traditional activated sludge system (Velika Gorica). These differences are understandable because the plants were built in the period of 40 years, however, from the perspective of high investments in the sector that are strategically planned to be implemented soon, it would be very advisable to examine the possibility of standardization of both, industrial process, and the equipment on the overhauled and new plants. That would contribute to a better state in the sector, easier exchange of findings, handling and mobility of plant operators¸ and it would eventually enable construction of central storages with spare parts that would service a group of plants with the same or similar equipment. Additionally, servicing of a plant and the equipment would be easier and cheaper, and occupational safety and health in extraordinary circumstances would be higher.
- Mathematic modelling for support in designing of WWTP has not been applied in practice in Croatia in the Black Sea Basin, which gives an additional motivation for implementation of this design. Due to that, we believe that the potential for implementation of modelling in the Republic of Croatia is great.
- On most of the older plants, the problem of unpleasant odours has not been solved adequately, nor has it been anticipated in designs. Additionally, regarding occupational safety and health, on older plants, shortcomings were noticed, such as excessive noise, risk of inhaling aerosol from biological treatment with surface aeration, and sometimes, insufficiently secured approaches to (on) the structures.
- On 15 of 26 plants, some kind of biological treatment was implemented, with 12 of those having (modern) systems with diffuse aeration (fine air bubbles) implemented, while on three plants aeration is achieved by surface aerators (turbines or rotors). It seems that surface aeration is slowly disappearing in practice in Croatia, which is a positive trend.
- All the observed plants were submitted to laboratory tests of water samples (except for two which are not functional). There is no reason not to believe that laboratory technicians on plants and authorized laboratories carry out the analyses correctly, although some data received from the managers are not logical. Resolving of such illogical points or contradictory data was not approached in this project. The level of details during monitoring of the plants oscillates considerably and depends on legal regulations, on the size and complexity of the plants, and the motivation of the personnel. The trend that most of the plants monitor only some parameters in the plant influent and effluent (such as BOD_5, COD and SS) has been noticed, while plants which monitor the efficiency of industrial units within the same, by sampling before, inside and behind particular structures, are less frequent. There is evidently space for improvement in monitoring of

plants operation, which may often point to points where the efficiency can be increased, and expenses reduced.

- Traditionally, with the exception of WWTP in Zagreb and Koprivnica, sludge treatment has not been given sufficient attention. That was emphasized on a number of plants where sludge treatment was incomplete, was not operating partially or completely, or was not anticipated by the design at all. In comparison with water treatment, sludge treatment is way behind on these plants, and it seems that the same goes for knowledge and experience of the personnel in working with the same. Therefore, it is recommended to quickly approach integral resolving of the problem of treatment of sludge produced in WWTP, and to start introducing modern industrial processes (of water treatment) which have a reduced production of sludge, and the corresponding sector personnel training for their eventual implementation.
- Considering the situation regarding disposing of sludge to dumpsites gives a lot of space for pessimism because, on many WWTP, the personnel does not have a practical and long-term solution for sludge disposal, so that sludge stays in lagoons or similar 'temporary' dumpsites for years. The option of sludge incineration was mentioned only in one plant (Belišće). Furthermore, seepage water from curing fields and lagoons is rarely collected and returned to the plant for treatment.
- It was also noticed that some of the proposals for reconstruction or extension of the outdated or overloaded plants is done in the way that does not consider the oncoming requirements of EU to which Croatia intends to accede, particularly regarding the removal of nitrogen and phosphorus, which is a pity because, with relatively small additional investments in the present time, quite a lot of funds can be saved in the near future.
- From 26 observed plants, only one (Koprivnica) has the tertiary level of treatment (removal of both, nitrogen, and phosphorus), which is a good beginning for wider introduction of adequate technologies for removal of the same in Croatia. The strategy which includes raising the level of the efficiency of most of the larger plants in Croatia is definitely the path to be followed, particularly in sensitive areas which are abundant in Croatia.
- At all the visited locations, the plant personnel was very open, ready for cooperation and it seems that the consultants were given all the possible support in the domain of their competences;
- The plant personnel demonstrated a relatively high level of understanding of operation of the plants in which they are employed, however, it would be very desirable to organize meetings of plant managers at the level of the Inland Water Project and of the Adriatic Project, at which they would share their experience and findings with colleagues from the trade, and expand their knowledge and interests to the processes and systems of wastewater treatment in Croatia, and even wider. Organization of continuous additional training, including the most modern methods and technologies in the field of water and sludge treatment would be very desirable. Maybe, a kind of a "Wastewater Treatment Summer School" would meet that purpose, since there already is a general interest in annual organizing of such a symposium / training for the area of the Black Sea Basin and for the Adriatic Basin.

1.9 Conclusions

In comparison with EU member states, and especially with West European and Scandinavian countries, Croatia has significantly lower connection rate of population to public sewerage systems and, consequently, of treatment of collected wastewater.

Since the beginning of accession negotiations in 2004, significant efforts have been made in terms of overview and analysis of the current state in collection and treatment of wastewater in Croatia. The Water Management Strategy and the Water Act and Act on Financing Water Management were adopted, and harmonization of the national legislation with the EU *acquis communautaire* is finished as well. Implementation of two strategic national projects, the Adriatic Project and the Inland Waters Project, is well under way. Phase two of the Adriatic Project has begun, and up to now significant investments were made into the construction of urban wastewater collection and treatment systems and wastewater treatment plants, in both Adriatic basin and Black Sea basin. This development reflects the determination of Croatia to invest into environmental protection and preserve the quality of inland and coastal waters which is basis for sustainable development of the whole country and has also an impact on the wider region due to cross-boundary character of some large rivers and their basins.

Stocktaking and the analysis of 26 WWTP in the Black Sea Basin provided new information about the present state of wastewater treatment in the Croatian part of the Black Sea Basin and showed a realistic presentation of the sector. For the purpose of completing the survey of the state in the wastewater sector, it is necessary to elaborate a similar study for the Adriatic Basin as well.

Faster introduction of innovative approaches is necessary (such as the implementation of models) in the wastewater sector in the Republic of Croatia. The modelling approach enables the development of considerably more options and variants in relation to the present traditional designing practice in Croatia, and it gives an incomparably greater certainty regarding the reliability of the project results. It is to be expected that this approach will widely open the door towards the implementation of modern approaches in the wastewater treatment sector in Croatia, and that modelling of the processes in the sense of improvement of work and upgrading and expanding of the existing WWTP will come to be a standard in the process of wastewater management in Croatia, whereby Croatia would approach the practice implemented in the most developed member countries of EU in this way too, by which it would have an even greater effect on the actual rationalization of capital and other operating expenses of WWTP.

Chapter 2

The role of modeling in modern wastewater management

2.1 Why modeling?

The most important advantages of the use of models in wastewater treatment are:

- Getting insight into plant performance;
- Evaluating possible scenarios for upgrading;
- Evaluating new plant design;
- Supporting management decisions;
- Developing new control schemes;
- Providing operator training.

Modeling forces the modeller to make his or her work explicit. Qualitative comparisons are often found in the literature such as 'better', 'larger', 'smaller', 'higher', etc. Such comparisons are not very useful and are of a subjective nature, as, for instance, the perception of 'large' or 'small' by a researcher in the laboratory or by a person operating a wastewater treatment plant is not necessarily the same. When it comes to modeling it is not possible to use descriptive elements, but it is necessary to use quantitative inputs for sizes, rates, and conversions as the model requires numbers as input. This also forces modellers to become quantitative and objective in their approach and thus the process knowledge gets a better definition. Of course, one can do without modeling, but very often by making a model one makes a framework that takes into account everything which is considered relevant. It furthermore forces structured and more extensive data collection, and encourages the modeller to be organized. It often exposes knowledge and data limitations and/or incorrect data (such as SRT or flows), supports efforts to improve quality of data, and enhances good plant monitoring practices. Therefore it is not surprising that getting insight into plant performance (quantification of information, mass balances and data reconciliation) and learning about the wastewater treatment plant in question can be even more important than the modeling itself.

The second main reason for using models is the possibility to save time and money whilst selecting the technology and process. A quantative rather than qualitative comparison of the system performances allows in many cases for easier decision-making and a rapid comparison of options. In comparison with the qualitative description such as 'one system is more efficient than the other', model results showing that 'one system is 2% (or 20%) more efficient than the other' is much more informative and useful. If important information or selection criteria are quantified (such as purification efficiency, effluent quality, sludge production, oxygen requirements, etc.), the application of modeling for evaluating possible scenarios for upgrading

will make the comparison more effective and faster than a discussion on such issues that are usually empirical, intuitive, long and often cumbersome. For the purpose of evaluating upgrade scenarios, it is not necessary to make a very accurate model by performing an extensive calibration procedure, as the real uncertainty is associated with the model inputs and not with the model parameters. It is considered much more useful to use trends for comparison as small differences are not relevant in the context of the usual design horizon used in wastewater treatment engineering. In the case of evaluating new plant design, again it is not necessary to have a fine-tuned model due to uncertainty in process conditions in the coming 10 or 20 years. For primary plant design usually static (steady state) models are used while dynamic modeling is applied for sensitivity analysis and optimizing the design. An additional challenge is the fact that wastewater has an extremely complex and uncertain composition. Wastewater flow rate and concentrations are of a highly dynamic nature and are very difficult to control, despite certain limited possibilities to influence the wastewater composition (see Chapter 9). Many processes take place within the wastewater treatment plant, some of them are relevant to the treatment and many of them are not. However, many of them occur simultaneously, even in a single process unit. In order to handle such a complex situation, there is a need for a model to support the understanding of these relevant processes. Despite the fact that from the design perspective, modeling as such is usually not strictly necessary, it is becoming increasingly used as a part of the design process. By applying statistical methods to the occurrence of worst-case scenarios, significant savings can be made and the plant can still achieve its effluent quality standards for about 95% of the time. Often in traditional design all the worst case scenarios are assumed to occur simultaneously leading to a highly unlikely scenario.

Another good reason for using models is the possibility of decreasing or minimizing risks. By using models, 'what if' scenarios can be examined in a quantitative way in respect of what the effects of potential risks are. Such a glass-box type (as opposite to black-box type) quantification is invaluable in the evaluation and selection of acceptable risk, rejecting risks that cannot be taken, and in identification of upfront measures that can be taken to mitigate or control such risks. For example, questions such as 'what will happen if the flow rate doubles?' and 'what is the effect of such an increase on effluent quality?' can be properly addressed by using models. Furthermore, models allow for minimization of risks that are related to scaling-up of the systems (lab-scale vs. pilot-scale vs. large-scale). Related risks originate from the fact that, for example, mixing conditions, load variations etc., are different for full-scale and lab-scale installations. From the perspective of process control, pilot-scale gives a much faster response in comparison with full-scale plants with greater inertia.

Furthermore, the application of models improves knowledge transfer and decision-making. Wastewater treatment engineering and environmental engineering in general are multidisciplinary fields requiring knowledge of different disciplines, such as microbiology, biochemistry, and physical, biological and mechanical engineering. In addition, each expert group involved, be they operators, engineers or scientists, usually has its own perspective of the same subject. By phrasing the subject into a mathematical context the same communication tool (language) is used. Such a multidisciplinary approach allows for a better description of the reality, each discipline delivering its own input for a better understanding of the reality that can be incorporated into the model in a structured, organized and quantitative way. Model-related communication was greatly improved after the introduction of ASM1 in 1987. Prior to the introduction of ASM1, at least five or six different ways of modeling wastewater treatment plants were described. Each model had its different approaches in writing-up, in notation, and in implementation of equations, which made it extremely difficult to understand the models and

their results. The uniform context and standardization introduced by ASM1, in terms of notations, symbols and structure, made comparison of results and knowledge transfer much easier and further encouraged modeling applications.

Models are nowadays invaluable tools for training. For example, the plant operator can safely investigate by means of modeling what can happen if one takes certain action at a treatment plant without risking eventually upsetting the operation of the plant. Moreover, models can be used to transfer knowledge from design engineers to operators and, of course, in academia, where modeling is increasingly becoming a part of the curricula for engineers and scientists worldwide. From the perspective of process control, in practice there are no direct model-based controllers functional yet, as their application still remains only of scientific interest. In practice simple controllers are tuned based on the model, which allows for much quicker optimization of the control strategy at the full-scale installations. In the framework of integrated urban water system modeling, wastewater treatment modeling is an important component and it is necessary to link up wastewater treatment with the sewer system (to take into account effects of, for example, combined sewer overflows or processes taking place in the sewerage system) on the one side, or the receiving water's quality and quantity, on the other. Integrated modeling is becoming an increasingly popular tool to support decision-making at the level of urban water system management as it brings objectivity and gives quantitative insight into relevant differences between options.

2.2 Modeling and WWTP lifecycle and costs

In fact, the application of models has a place during the entire lifecycle of a wastewater treatment plant including the design, construction, and operation and evaluation stages (Figure 2.1). The spectrum of possible use of models during the WWTP lifecycle is shown in Figure 2.2. The practice revealed that the most cost saving is possible when models are used at the early stage of the plant lifecycle (Figure 2.3). As is elaborated on in Chapter 3, the modeling goal determines the type and complexity of the model to be applied. Figure 2.4 depicts how model complexity increases as the plant lifecycle matures.

It can be concluded that the largest savings are possible at the plant design phase because:

- Modeling helps to quantify scenarios at the early design stage;
- Quantification helps speed up decision making;
- There is considerable uncertainty in the early design phase (e.g. influent composition);
- Large safety margins are needed (usual 150 to 200%);
- Models can be simple (no calibration needed!);
- During the design phase investment in models and modeling work is usually paid back.

Furthermore, practice shows that the highest operational risks are at the operational stage because:

- Modeling helps reduce operational risks;
- Operational problems are often complex;
- More accurate models are required (e.g. ASM models);
- Possible financial savings are often less;
- Not only reduction in costs will show if modeling is useful.

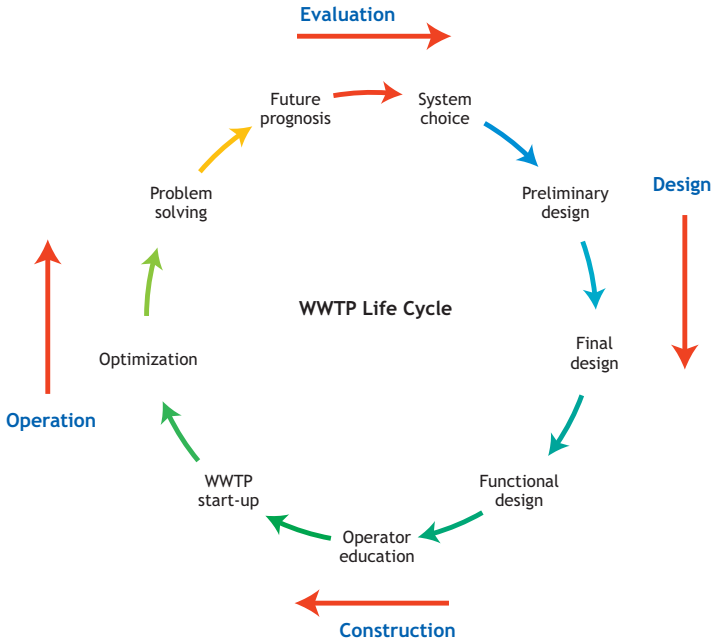

Figure 2.1 Wastewater treatment plant lifecycle

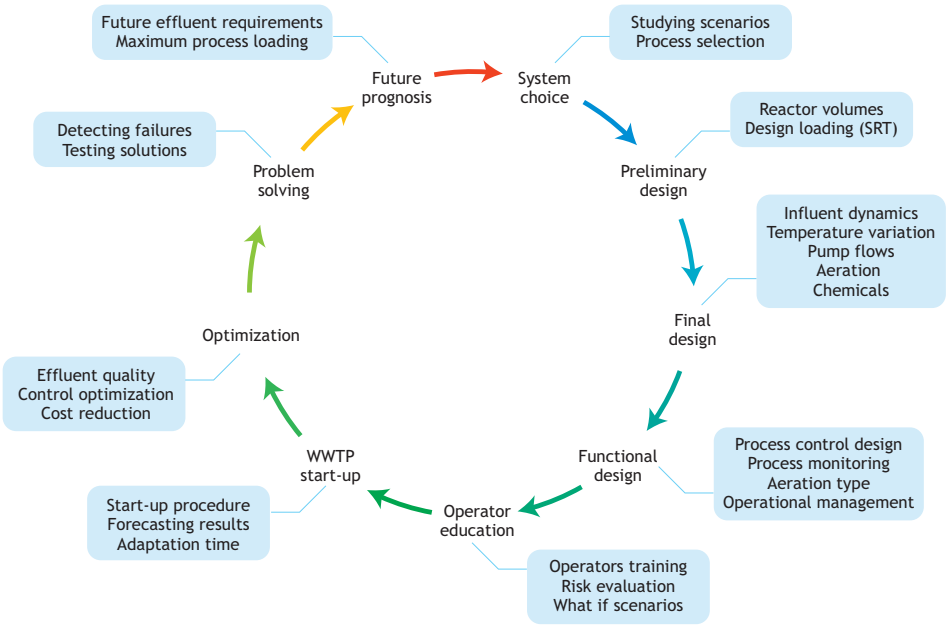

Figure 2.2 Modeling application at different stages of the plant lifecycle

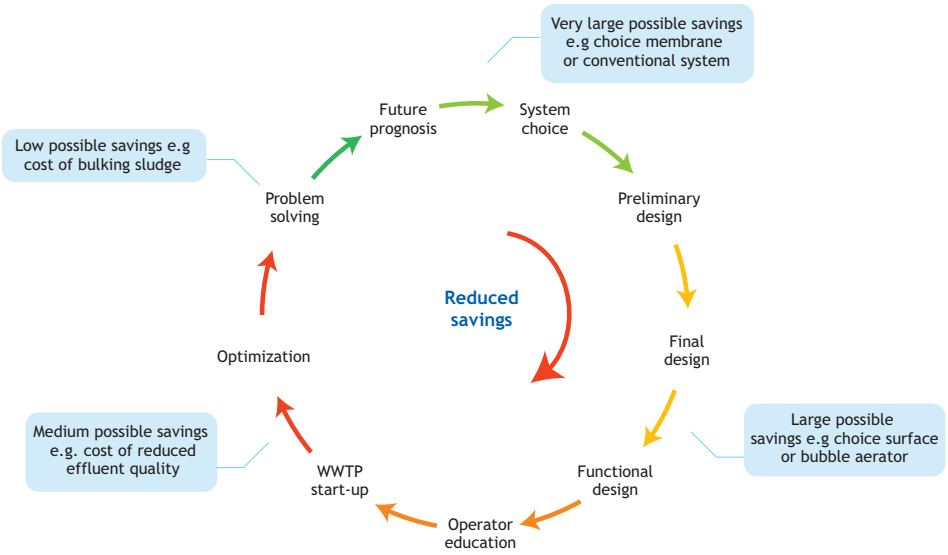

Figure 2.3 Possibility for costs saving during the WWTP lifecycle

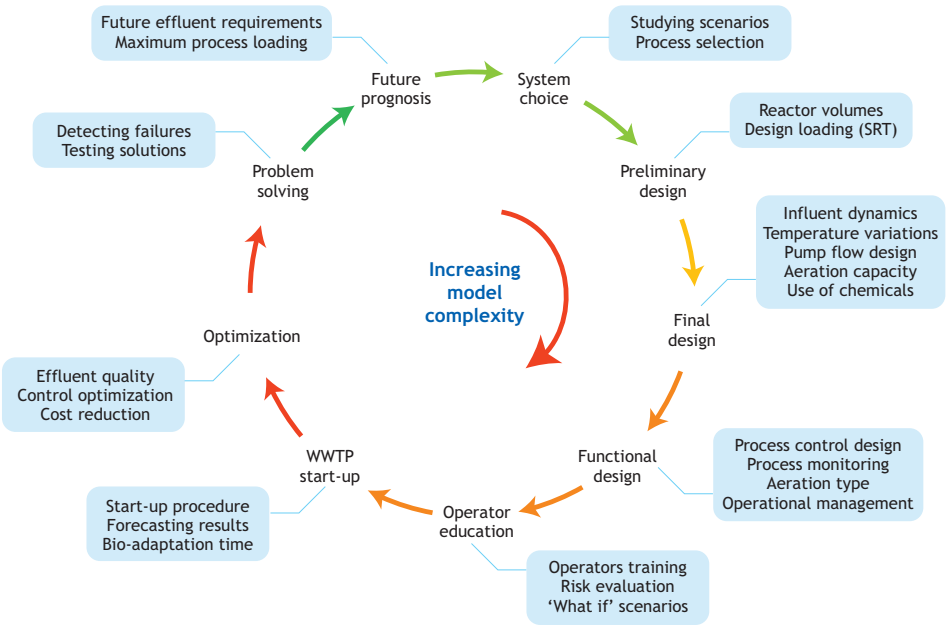

Figure 2.4 Model complexity and increasing operational risks during the plant lifecycle

Reasons to introduce standards and models in wastewater management at institutional level are (the top-down approach):

- To standardize WWTP operation and management;
- For quality assurance and control;
- To improve efficiency and reduce costs;
- As a knowledge base (organize process documentation);
- To improve internal and external communication (standardization of information);
- For planning and decision-making.

Reasons for engineers to use standards and models can be to (the bottom-up approach):

- Do the work better by having better insight in the process and to make better and more effective plant designs;
- Improve personal skills by working with 'state of the art' technology and keeping their knowledge up to date;
- Make the work dynamic by having a low-cost and safe platform to test ideas for better WWTP operation;
- Make the work more easy by creating a reusable model (knowledge base);
- Increase reliability of their work by supporting decisions in communication with higher management;
- Have the most effective tools for process optimization and process control.

Success factors for using modeling in design can be summarized as:

- Working according to a protocol;
- Making a realistic project plan;
- Being practical;
- Getting a 'helicopter' view of the modeling project;
- Defining clear modeling goals;
- Keeping the model as simple as possible;
- Using a standard calibration method.

However, one should be aware that by doing modeling several bottlenecks may be identified:

- Choice of methods and software is important - one standard is required;
- Different approach towards WWTP information is needed;
- There is a continuous need to invest in education (life-long learning);
- Sharing of knowledge through a modeling platform, meetings, internet forum, specialist groups etc., is crucial;
- Knowledge sinks when not used, thus investing in successive demo projects is necessary;
- Consensus on these statements should be reached: "the use of models….
    - saves money,
    - improves quality,
    - is effective for management and decision making,
    - is the technology of the future".

Finally, the use of models in wastewater treatment has the following main advantages:

- Cost reduction, especially at the design phase;
- Improved management and quality control;
- The optimal technological/process design is obtained;
- Works with modern 'state of the art' technology;
- Application of innovative approaches;
- Development of designs at low cost, fast and with confidence.

Chapter 3

Introduction to activated sludge models

This chapter has been modified by D. Brdjanovic and S.C.F. Meijer and is based on the chapter 'Modeling Activated Sludge Processes' by Mark C.M. van Loosdrecht, George A. Ekama, Mark C. Wentzel, Damir Brdjanovic and Christine M. Hooijmans in 'Biological Wastewater Treatment: Principles, Modeling and Design' edited by M. Henze, M.C.M. van Loosdrecht, G.A. Ekama and D. Brdjanovic. ISBN: 9781843391883. Published by IWA Publishing, London, UK. (2008)

3.1 What is a model?

A model can be defined as a purposeful representation or description (often simplified) of a system of interest (Wentzel and Ekama, 1997). This consequently means that the model will never exactly reflect the reality. So, the question 'does this model describe a wastewater treatment plant?' is meaningless, unless it defines what part(s) of the treatment plant the model should describe in the first place. One never develops a model that describes every single organism, every molecule of water or every detail of the process. Models are used as a simplification of reality in such a way that they describe that part of reality that is relevant to understand and to deal with. It is also important to note that a mathematical model can only be successful if it fulfils people's expectations. There are two aspects that are extremely relevant in modeling: the aspect of time and that of scale. In general, processes can be separated into three groups from the perspective of time. Processes can be in a so-called frozen state, dynamic state, steady state or equilibrium. Models are usually made to describe the dynamic state, the state in which variations occur as a function of time. When a process is in a frozen state it means that the process will change over time, but not in the time interval that one is interested in. For example, if the daily dynamics of the wastewater treatment plant is of concern, the concentration of ammonia in the effluent will vary over time; the concentration of nitrate will vary in the reactors etc. However, within one day, the ammonia and nitrate concentrations in the sludge digester, which can be part of the total activated sludge model, will not change. Usually the hydraulic retention time is 30 days, resulting in a characteristic time of change in this digester being in the order of two to three weeks. Consequently, one can consider the processes taking place in the digester as being in a kind of frozen state. There are hardly any daily variations in the processes in comparison with those taking place in the wastewater treatment line. On the other hand, there are processes that occur so fast that they are in steady state or in an equilibrium condition. These processes occur so rapidly that the speed of change greatly exceeds the dynamics that one is interested in. The changes that are of usual concern in wastewater treatment are, for example, changes in the ammonia concentration, which have a rate value in the order of magnitude of hours. The changes that are relevant to process control have a rate value in the order of magnitude of minutes. However, if one considers the chemical precipitation processes, these will occur more or less instantaneously (in a few seconds). These rapid processes do not have to be described in a dynamic way because they happen so fast that one can assume they are in equilibrium condition or completed. Therefore, one of the first considerations when making a model is to consider which processes are of interest, followed by

the determination of the relevant timeframe, an assessment of the dynamics within the process, and finally an accurate description of those processes that are time-varying. The other processes, that are in a frozen or steady state, are not of primary importance as they can be introduced in a much quicker, simpler way within the model, or even omitted. This is because they can be considered as continuous processes with stable concentrations under certain conditions (as in digesters). So, the aspect of time is the first major issue in trying to simplify the reality. The recommended approach is to consider the time constants and select those processes that have the dynamics in order of time constants one is interested in. For wastewater treatment this usually means hourly or daily dynamics, sometimes yearly dynamics. In the latter example, of course, digestion will become important as over the year the performance of a digester will change because the sludge production will vary during the year.

The second relevant issue for modeling is space resolution. One can theoretically make a model that describes every square inch of an activated sludge plant. But the question is whether one is interested in such a detailed description in the first place. The answer depends once again on the purpose of the model. In general, in wastewater treatment practice, the reactor size is in the order of tens of meters. In order to describe the concentration gradients of relevant components in the reactor, of which oxygen is the most sensitive one, usually a scale size of a few meters is needed. On a different scale, there is a gradient of concentrations inside the activated sludge floc that theoretically can be described by a model. However, in standard activated sludge modeling it is neglected, as not being relevant enough to be taken into account. This consequently means that activated sludge models are usually not designed to describe the system at the length scale of an activated sludge floc but at the length scale of a reactor.

The next step in modeling is to look at the relevant level of detail of a microbial model. Typical traditional wastewater treatment design methods are based on the 'black-box approach' focusing on plant influent and effluent characteristics, while nothing or very little is known about what is happening inside the wastewater treatment plant. Traditional design parameters, such as the F/M ratio (sludge loading rate) are not based on the understanding of the processes within a wastewater treatment plant. However, one can design a plant reasonably well by applying a proper loading rate, without really knowing what processes are taking place in the plant. So the black-box model can work well in reality (Figure 3.1).

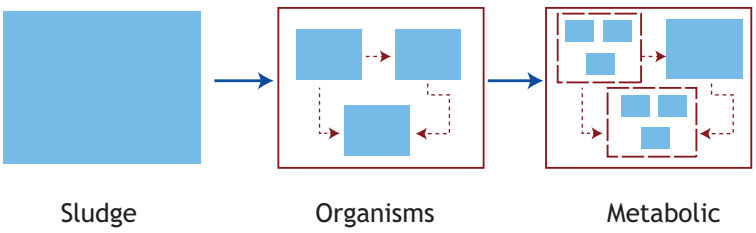

Figure 3.1 Schematic representation of the step-wise refinement of a model (Smolders *et al.,* 1995)

The black-box model is not by definition wrong or unscientific, but the application of the black-box approach very much depends on the purpose of the model. If the purpose is to design a wastewater treatment plant, experience has shown that the F/M ratio can be a very good basic

approach for the design, despite the fact that it does not give information about the sludge composition. One can refine this approach and move towards grey-box models, as was the case in the Activated Sludge Model No.1 (ASM1, Henze *et al*., 1987), No. 2 (ASM2, Henze *et al*., 1995), and No. 2d (ASM2d, Henze *et al*., 1999). Here the sludge was split up into relevant fractions: an inert organic matter fraction, a fraction of nitrifying bacteria, heterotrophic bacteria, denitrifying bacteria, and a fraction of phosphate-removing bacteria. Different functional aspects of the sludge were specified for a population-based model where selected microbial communities are defined inside the activated sludge and as such incorporated in the model.

Furthermore, the metabolism of the organisms and the metabolic routes inside the organisms can also be described. With such an increase in information, the approach becomes close to glass-box modeling (such as the Activated Sludge Model No.3: ASM3, Gujer *et al*., 1999, and the TU Delft EBPR model: TUDP model, Van Veldhuizen *et al.,* 1999). This results in a larger and more complex model. The challenge here is to figure out for each process what the adequate level of description is. The question is: does the increase in complexity also increase the quality of the model (outputs), in other words, does it provide a better description of the wastewater treatment plant? For example, it has been shown that with an increased level of detail in the description of nitrification, a marginal improvement of model performance can be obtained while in the case of phosphate removal, significant improvement can be gained by including a metabolic description. Therefore, the preference for black-, grey-, or glass-box modeling depends largely on the purpose and application of the model. This is the point where modeling often goes wrong as modellers neglect the purpose and make the model itself the purpose of modeling.

Of course, one can attempt to proceed to the next level of complexity by including fundamentals of microbial genetics and genetic change. Technically and in principle this is possible, but again, this must happen depending on what the purpose and use of the model are. If the model is made too complex and with too many parameters in relation to what one ultimately wants to describe, then such an approach can generally be considered a waste of time and effort. There is also no absolute need for a model that describes the exact reality. How far reality should be matched depends again on the purpose. For example, if one wants to get an idea about N_2O emission from wastewater treatment plants, maybe three of four theories can be created and incorporated into the model. At this point it is of prime interest to look at the simulation results of different models in terms of trends and to what extent these trends reflect reality. At this stage one only has to focus on trends; good calibration, exact fitting and accurate knowledge of parameter values are not necessary. On the contrary, as an example, if one needs to predict the performance of a plant to satisfy legislation that requires every sample taken from the effluent to be below 1 mgNH_4/L, then the accuracy of the parameters put in the model must be much higher. One has to guarantee that the model prediction must be below 1 mgNH_4/L exactly. These two examples once again show that the model should always be judged in relation to the purpose of its use.

Two extremes in types of mathematical models can be identified: empirical and mechanistic models. An empirical model is based on recognition of the parameters that seem to be essential to describe the behavioural pattern of interest, and linking these through empirical relationships established by observation. The mechanisms and/or processes operating in the system are not known or are often ignored: a classical black-box approach. In contrast, a mechanistic model is based on some conceptualization of the biological/physical mechanisms operating in the system, i.e. is based on a conceptual idea (or model). The complexity of this mechanistic model will depend on the degree of understanding of the biological and chemical processes occurring in the

system. Because mechanistic models have a conceptual basis, they are often more reliable than empirically-based models. Because of their black-box approach, the empirical models have an application strictly limited by boundaries (e.g. wastewater characteristics, system parameters) within which the model was developed; only interpolation is possible. Being conceptually based, the mechanistic models have greater sureness in application outside the boundaries within which the model was developed; both interpolation and extrapolation are possible. However, ultimately all models are only our rationalization of behavioural patterns or processes we conceive to be of interest. Because of this rationalization, any model needs to be rigorously calibrated and adequately verified by appropriate tests. Also, the conditions within which the model is expected to operate successfully need to be firmly delineated. For the empirical models these are strict conditions within which the model was developed, and for the mechanistic models these are the conditions under which the conceptualized behaviour is expected to remain valid. It is evident from the discussion above that the mechanistically based models have greater potential for application to wastewater treatment plants, and attention will be focused on these models.

Processes operating in a system and the compounds on which these act must be identified to set up a conceptual model on which the mechanistic mathematical model is based. The various interactions between the processes, and between the processes and compounds, are delineated descriptively. In order to develop the mechanistic model from the conceptual model, the process rates and stoichiometric interactions with the compounds are formulated mathematically. The mathematical equivalent of the mechanistic model will very probably not include all the processes and compounds that are present in the system; only those perceived to be of significance for fulfilling the objectives set for the model need be included. The art of constructing conceptual and mechanistic models is in eliminating those processes and compounds that contribute little or nothing to fulfilling the objectives set for the model. It is a waste of time and effort to develop a complicated model when it is unlikely that a model can be developed that describes a phenomenon completely. Theoretically a complete description should include aspects down to the most fundamental level. The level of organization is usually set by the objectives for the model. For example, in modeling biological behaviour in wastewater treatment systems, we cannot directly implicate biochemical control mechanisms (such as ADP/ATP and NAD/NADH ratios), or even the behaviour of specific microorganism species. The mixed liquor in the activated sludge system contains a wide diversity of different microorganism species for which identification and enumeration techniques have recently become available. These techniques, however, are time and labour consuming. Instead, microorganisms that fulfil a particular function in the activated sludge system (e.g. aerobic degradation of organics or nitrification) are grouped together as a single entity, which is called a 'surrogate' organism. This surrogate organism is assigned a set of unique characteristics that reflect the behaviour of the group, but might not reflect the characteristics of any individual organism or species of organisms in the group. To illustrate, this approach is equivalent to modeling the 'macroscopic' behaviour of a forest of trees as opposed to the 'microscopic' behaviour of each individual tree or species of trees that makes up the forest. In considering the behaviour of the forest, a parameter that could be modelled, for example, is carbon dioxide (CO_2) production. The forest as an entity will have defined CO_2 production and consumption rates. Individual tree species within the forest, or even every single tree, might have specific CO_2 production and consumption rates that deviate significantly from those of the forest entity. However, the effect achieved by modeling the forest as an entity will closely equal the net effect of modeling the cumulative contribution of each individual tree or tree species. The great advantage in modeling the forest as an entity over modeling the individuals is that considerably less information is required to develop the model and to calibrate it. In modeling biological wastewater treatment systems, the

utilization of substrate by organisms is a typical example: Monod's Equation (Monod, 1949) is used to relate the specific growth rate of the surrogate organism to the surrounding substrate concentration, whereas the organisms making up the surrogate group might have different specific growth rates or might respond differently to the various substrates present in the influent wastewater. Thus, for modeling wastewater treatment systems the organizational level that is modelled is the mass behaviour of a population or group of selected microorganisms. In the models developed for activated sludge systems, this is the principle organism groups, their functions and the zones in which these functions are performed.

The parameters at that level that need to be included in the mathematical model depend greatly on the objectives for the model accepting the level of organization described above. For mathematical modeling of wastewater treatment systems two different kinds of mathematical models are generally developed: steady state and dynamic models. Steady state models have constant flows and loads and tend to be relatively simple. This simplicity makes these models useful for design. In these models complete descriptions of system parameters are not required. They are oriented towards determining the more important system design parameters. The dynamic models have varying flows and loads and accordingly include time as a parameter. Dynamic models are more complex than the steady state ones. The dynamic models are useful in predicting the time-dependent system response of an existing or proposed system. Their complexity means that the system parameters have to be completely defined for its application. For this reason the use of dynamic models for design is restricted. Often the steady state design and dynamic kinetic models evolve interactively. The dynamic kinetic models can provide guidance for the development of the steady state design models; they help identify the design parameters that have a major influence on the system response and help eliminate those processes that are not of major importance at steady state. For the dynamic models, with their greater complexity, only those parameters that seem to be of importance are considered for inclusion in the model. For activated sludge systems, when selecting the level of organization at the surrogate organism or mass behaviour of populations, until recently the dynamic models have been structured to consider only the net effects as present in the bulk liquid. For example, when using Monod's Equation the kinetic rate is determined by the bulk liquid soluble COD and surrogate organism concentrations.

However, with the extensions of the models to include enhanced biological phosphorus removal (EBPR), parameters internal to the surrogate biomass have had to be included, e.g. poly-β-hydroxyalkanoate (PHA), glycogen and polyphosphate. With this development, although the model might be at a selected level of organization, information on processes and behaviour from lower levels of organization is often essential, particularly to identify the key processes that control the response of the system. Usually information from lower levels of organization is of a microbiological and/or biochemical nature, and the more complete this information is the more reliable the model. To make use of this information, 'model' organisms that are part of the 'surrogate' are identified and the known microbiological and biochemical characteristics of the organism are used to obtain a greater understanding of the surrogate. More recently the surrogate organism approach to modeling has been found to be inadequate to completely describe some behavioural patterns observed in activated sludge systems; for example the selector effect (Gabb *et al*., 1991), substrate utilization inhibition on transfer from anoxic to aerobic zones (Casey *et al*., 1994), and generation of nitrogen intermediates in denitrification (Casey *et al*., 1994). To describe these and similar observations, it has been found that a lower level of organization needs to be selected: the synthesis and activity of certain key enzymes and the processes they mediate need to be modelled (Wild *et al*., 1994). Modeling at this level of

organization has been termed modeling with structured biomass. Detailed microbiological and biochemical information is required for this modeling approach (Wentzel and Ekama, 1997).

It should be noted that there is an essential difference between an activated sludge model and a wastewater treatment (plant) model. The latter term is used to indicate the ensemble of activated sludge model, hydraulic model, oxygen transfer model and sedimentation tank model, all needed to describe an actual wastewater treatment at a full-scale installation (Gernaey *et al.,* 2004). The wastewater treatment plant model should be furthermore distinguished from a plant-wide model, which combines wastewater treatment models and sludge treatment models.

3.2 Modeling basics

3.2.1 Model building

Many different types of models exist; these can be broadly categorized into (*i*) physical, (*ii*) verbal or conceptual and (*iii*) mathematical models. The physical model is a spatially-scaled representation of the system. For example, the laboratory- and pilot-scale experiments used by scientists and engineers to investigate system response and behaviour are physical models. The verbal or conceptual model provides a qualitative description of the system and is usually developed from detailed observations; these models can be presented as schematic diagrams (e.g. flow diagrams) or as a series of narrative statements. Preparation of a mechanistic (verbal) model is the most important but also the most complex part of model building. The mathematical model provides a quantitative description of the system. With mathematical models the rates of the processes acting in the system and their stoichiometric interaction with the compounds are formulated mathematically. The mathematical formulations need to be incorporated in a solving procedure that takes account of the physical constraints and characteristics imposed by the system in which the processes take place, e.g. temperature and mixing conditions. Mathematical models are seldom developed in isolation, but usually evolve interactively from a conceptual model that might to some degree be based on observations made in a physical model, e.g. laboratory- or pilot-scale experiments (Wentzel and Ekama, 1997).

Methodology in research which combines verbal, mathematical and physical models (Figure 3.2) is very helpful to rapidly progress and evaluate new systems. A number of factors should be considered regarding activated sludge modeling and simulation, and a step-wise approach is needed to progress from the model purpose definition to the point where a wastewater treatment plant model is available for simulations. The following main steps can be distinguished in this process (Coen *et al.,* 1996; Petersen *et al.,* 2002; Hulsbeek *et al.,* 2002):

- Definition of the model purpose or the objectives of the simulation study;
- Model selection: choice of the models needed to describe the different plant units to be considered in the simulation, i.e. selection of the activated sludge model, the sedimentation model, etc.;
- Hydraulics, i.e. determination of the hydraulic models for the plant or plant tanks;
- Wastewater and biomass characterization, including biomass sedimentation characteristics;
- Calibration of the activated sludge model parameters;
- Model verification;
- Scenario evaluations.

The methodology is illustrated in detail by Petersen *et al.* (2002).

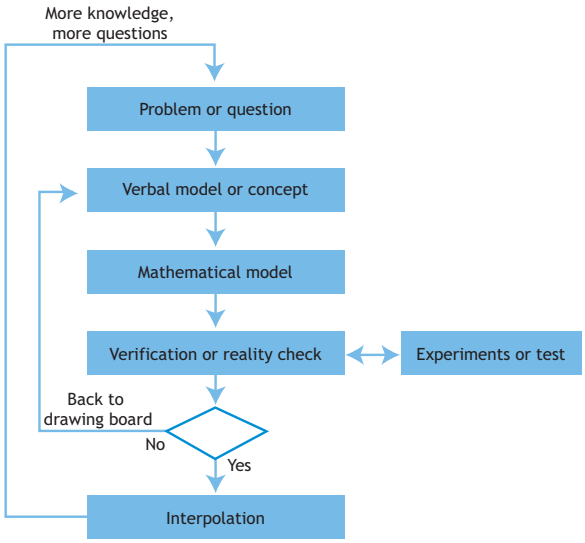

Figure 3.2 Model-building process

3.2.2 General model set-up

Balance equations form the basis of any model description. These equations describe the change in concentration in a reactor over time as the result of chemical and biological conversions and of transport processes. In steady state the change of the concentration as a function of time becomes zero. Transport and conversion processes are two different parts of the model (of a physical and chemical-biological nature, respectively). The biological processes are only dependent on the concentration in a reactor at the place where the conversion takes place. In essence the conversion processes are therefore independent of the type of reactor or the size of the reactor (microorganisms do not know in which type of reactor they are: concrete or steel, plug-flow or fully-mixed, activated sludge or biofilm etc.). Therefore the biological and chemical conversions are called micro-kinetics and can be easily studied in the laboratory and will not change at a full-scale installation. This part of a process model is therefore universal and can be formulated as a general activated sludge model, like the ASM model family. The local concentrations in a reactor depend on the transport of the reacting compounds in the reactor system or treatment plant. When comparing full-scale systems, the difference lies effectively in these transport processes. The advantage of transport processes (such as convective flow, mixing, aeration) is that they have already been thoroughly studied and described by general rules. They can therefore be predicted relatively accurately for different types and scales of processes. One can study biology and chemistry in the laboratory (for instance the effect of temperature, concentration and pressure on microorganisms) and then use physical transport models to predict what is going to happen at full-scale. Acknowledging the fact that microbes will not undergo changes between laboratory and full-scale conditions, as opposed to transport processes, helps to understand the processes and their integration in the mathematical model. Such integration allows the models to be used in the process design (selection of bio-reactors, types, stability, optimization, automation and control, scale up etc.).

The components of a full wastewater treatment model are given schematically in Figure 3.3.

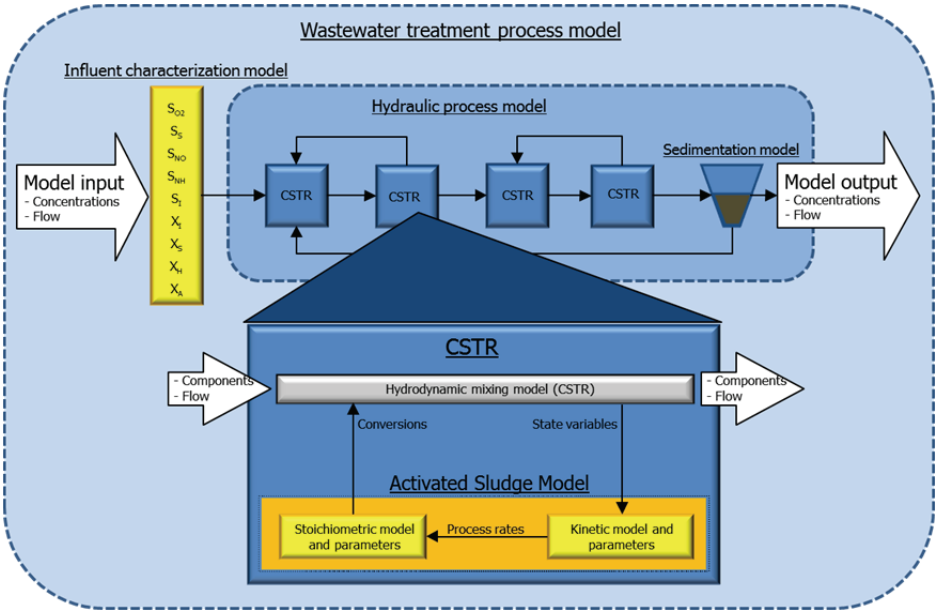

Figure 3.3 Schematic representation of a complete wastewater treatment plant model (Meijer, 2004)

First, measurable wastewater parameters have to be transformed to an influent vector with the concentrations of the different model compounds. The wastewater treatment plant is modelled hydraulically, describing the different zones/reactor compartments of the plant, including the settler. Each reactor compartment is modelled individually for its mixing and mass transfer (e.g. aeration) characteristics. Usually a completely mixed tank reactor is used. A mass balance equation is applied to each reactor. In such a mass balance equation the bioconversion model is included. In the overall model all the compartments are coupled by the state vector including the concentrations and flow rates of the links between the compartments. This overall model is usually solved numerically to show the concentrations of all compounds as a function of time for each compound included in the model. Therefore effectively we can speak of four models: the process model, the hydraulic model, the reactor/compartment model and finally the activated sludge model. Only this last model is general. The mass balance equation in steady state in mathematical terms reads:

$$\frac{\delta(S_{in}\cdot Q_{in})}{\delta t}=\frac{\delta(S_{out}\cdot Q_{out})}{\delta t}+(\alpha\cdot q\cdot X\cdot V)+(k_l A\cdot(S_{max}-S)) \qquad (3.1)$$

In which:

α	stoichiometry
A	surface area (m^2)
k_l	external mass transfer coefficient (m/h)
q	specific conversion rate (1/h)
Q_{in}	influent flow (m^3/h)
Q_{out}	effluent flow (m^3/h
S_{max}	saturation concentration (gCOD/m^3)
S	concentration in liquid (gCOD/m^3)
S_{in}	concentration in influent (gCOD/m^3)
S_{out}	concentration in effluent (gCOD/m^3)
t	time (h)
V	volume (m^3)
X	biomass concentration (gCOD/m^3)

Effectively it states that a compound entering a reactor either leaves with the effluent is converted in the reactor or is exchanged with the gas phase in the compartment. Each term on balance has the dimension of mass over time. It is useful to realize that in order to analyze a complex system it is better to work in these dimensions than with concentration terms.

3.2.3 Stoichiometry

From the system definition one takes only those compounds of the system that are considered important and/or make a significant part of the total system mass (being at least a small percentage of it). For example, in the case of nitrification the nitrite concentration will remain very low or close to the detection limit in most plants, so from the perspective of mass balance there is no need to take nitrite into account. In anaerobic digestion, similarly, there is no need to take hydrogen into account, as the hydrogen content of gas is very low, as almost everything ends up as methane. Such intermediates will only be specified if considered important, for example, when there is nitrite or hydrogen accumulation. Nitrite is not included in the nitrification process in ASM1, while in the Anaerobic Digestion Model (ADM1, Batstone *et al.*, 2000) hydrogen is included as it plays an important role in the stability of the anaerobic system. ASM models are specifically designed for applications at lower temperatures (5 to 20°C), under which no significant accumulation of nitrite is expected to take place. Nitrite will only accumulate at higher temperatures or in the case of unusual toxic events. Thus nitrite is left out of the model. Similarly, in denitrification, only a small amount of nitrate turnover is in the form of N_2O, so from the perspective of describing the N removal it is not relevant to include the contribution of N_2O. However, if the plant has to fulfil N_2O gas limits, then it becomes important to take it into account. Again, it depends on the purpose of the application of the model. Besides the determination of relevant compounds and processes, defining relevant balances is essential. For each conserved balance the number of atoms of a compound entering the plant is equal to those leaving. Examples of conserved balances are nitrogen, phosphorus, COD or alkalinity conversion. Using balance equations, unknown stoichiometry coefficients can be calculated. This substantially reduces the information required for modeling as the approach allows a number of unknown values to be calculated. The use of BOD measurement as the characteristic of wastewater is declining and, instead, modern approaches rely on COD. BOD-based design is associated with a black-box approach, which cannot be used for balancing as it is not conserved, and depends on many factors (e.g. reaction time, temperature). In reality, it is still mostly used to link the output of ASM regarding the effluent impact on the receiving waters (where BOD is still a relevant indicator of water quality). In contrast, the COD balance is conserved because

COD is by definition the amount of electrons which are transferred to oxygen in order to oxidize all the organic matter in the system to CO_2 and water. That is why the modeling is based on COD rather than on BOD.

Stoichiometry can be determined based on relevant compounds involved in the reaction and use of balances to calculate those relevant coefficients. For example, in the reaction for heterotrophic growth, the relevant compounds are organic matter, oxygen, ammonia, alkalinity, biomass, carbon dioxide and water. At this stage of equation development it is not necessary to determine which compound is utilized and which produced as it will simply receive a negative or positive sign, in other words, it is not important on which side of equation the parameter is. The next step is to choose one coefficient as 1 and to use balances to calculate all the other (relevant) coefficients. In the example below, 5 balance equations can be made (for carbon, oxygen, hydrogen, nitrogen and charge) and there are 7 unknown coefficients. One of these coefficients can be made equal to 1 and there is only one coefficient that we need to know, for example the amount of oxygen consumed per COD converted or the amount of biomass produced per substrate COD mass utilized (yield coefficients).

This is a general approach for setting up the stoichiometry of the reaction for any biological process e.g. if organics (or COD) are utilized aerobically (with O_2).

$$?COD + ?O_2 + ?NH_4^+ + ?HCO_3^- \rightarrow ?Biomass\text{-}COD + ?CO_2 + ?H_2O \quad (3.2)$$

In wastewater treatment systems we are usually not interested in the CO_2 and H_2O conversion and the COD balance is used to replace one of the elemental balances. If one sets the coefficient for substrate-COD as 1, and if the yield coefficient for the biomass is known, Eq.3.2 becomes:

$$1COD + (1-Y_H)O_2 + f_N Y_H NH_4^+ + f_N Y_H HCO_3^- \rightarrow Y_H\ Biomass\text{-}COD \quad (3.3)$$

where:

Y_H	heterotrophic yield (gBiomass-COD /gSubstrate COD), and
f_N	fraction of nitrogen in biomass (gN/gBiomass-COD)

To derive this equation we effectively used the COD, nitrogen and charge balance. The COD balance states that oxygen consumption and biomass production are always linked; it is impossible to save oxygen and produce less sludge as the substrate (COD) is either oxidized by oxygen or becomes sludge. From the N balance the amount of ammonia needed can be calculated, and from the charge balance the amount of bicarbonate (alkalinity) can be determined, etc. The stoichiometric reaction can be written down as a function of the yield coefficient and, in this particular example, the amount of nitrogen inside the biomass. The stoichiometric coefficients for each compound are included in the model matrix (Table 3.1).

3.2.4 Kinetics

Each reaction has its own rate equation. The rate equation specifies the rate of conversion of the compound with the stoichiometric yield coefficient of 1. The conversion rate of the other compounds follows from multiplying each yield coefficient with the rate equation. The model can be based either on substrate-based kinetics (the substrate stoichiometric coefficient is equal to 1) or growth-based kinetics (the biomass stoichiometric coefficient is equal to 1). It is not advisable to use both at the same time in one model. In ASM1 the rates are described based on the growth rate; the biomass coefficient is therefore set as 1. In ASM, a saturation equation is

used as the standard rate equation. Saturation (Monod) kinetics includes two main parameters, the maximum rate parameter and affinity or saturation constant (K value, defined as the concentration at half the maximum rate). The saturation term S/(K+S) can have a value between 0 and 1, and can have a different function in the model. Several affinity terms reflect a real value, e.g. the oxygen affinity term is an observed parameter. However, in some cases the saturation term is only a switching term. For example, a switching function is used in the model to stop the growth process when there is no ammonia present (Eq. 3.4). The affinity constant for ammonia is effectively very low and hardly measurable, so the sole purpose of the coefficient placed in the equation is to guarantee there is no growth anymore if the ammonia is fully consumed. This consequently means that one does not need to calibrate this value. How to distinguish between real measurable parameters and switching functions is a bit vague and inexplicit in activated sludge modeling. Therefore it is important to realize whether the K values are there as real model parameters or as a switching function to stop the process when the relevant compound is not present.

$$\mu = \mu^{max} \frac{S}{K_S + S} \cdot \frac{S_O}{K_O + S_O} \cdot \frac{S_N}{K_N + S_N} \cdots \tag{3.4}$$

To describe inhibition kinetics, a similar approach is applied, but now the affinity constant is called the inhibition constant, and consequently it is possible to define an inhibition term (Eq. 3.5), which again has a value between 0 and 1. The inhibition constant is equal to the substrate concentration at which a 50% decrease in the rate is observed. There are also much more complex inhibition terms, but in ASM this is the usually applied term, especially for the substrate inhibition.

$$1 - \frac{S}{K_S + S} = \frac{K_S}{K_S + S} \tag{3.5}$$

It is important to note that multiplying so many factors causes deviation because these factors are never exactly 1. If one multiplies the two factors with the value of 0.9 with the value of the third factor that is 0.5, the result will be 0.4, while the real value should be 0.5 because this is the limiting factor. This consequently means a 20% lower rate value. Therefore it is better to use a logical operator in the model and choose the minimum factor among the terms (Eq. 3.7) instead of multiplying these factors (Eq. 3.6) as it seems that it gives a better approximation of the reality.

$$\mu = \mu^{max} \cdot \frac{S}{K_S + S} \cdot \frac{S_O}{K_O + S_O} \cdot \frac{S_{NH}}{K_{NH} + S_{NH}} \cdot \frac{S_{KI}}{K_I + S_{KI}} \tag{3.6}$$

$$\mu = \mu^{max} \cdot MIN\left(\frac{S}{K_S + S}; \frac{S_O}{K_O + S_O}; \frac{S_{NH}}{K_{NH} + S_{NH}}; \frac{S_{KI}}{K_I + S_{KI}} \right) \tag{3.7}$$

The reason that Eq. 3.6 is used is partly an inherited habit (at the time of early model development in the 1970s, computing logical operators by integral differential equations was difficult and extremely time-consuming and thus not applied). It does not matter so much which equation is used for activated sludge modeling; the point is to understand the reasons for the choices at different stages of the model development.

3.2.5 Transport

A typical wastewater treatment model has several transport terms, often being time-dependent (Figure 3.3). The model input is the time variable flow and composition of the wastewater. The process is described in a hydraulic model, representing the hydraulics of the full-scale plant. An example is given in Figure 3.4.

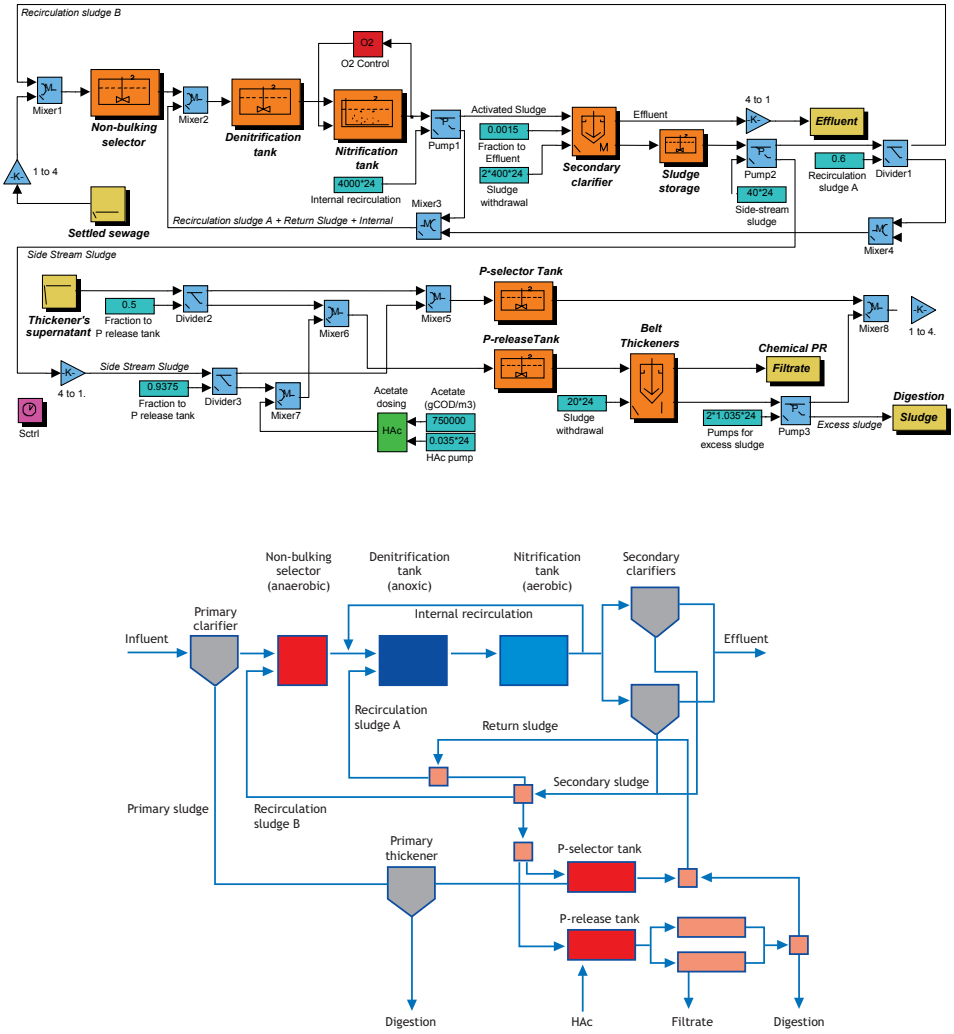

Figure 3.4 Hydraulic process scheme of the PhoStrip® process at WWTP Haarlem Waarderpolder in the Netherlands and its representation in the modeling simulator SIMBA (Brdjanovic *et al.*, 2000)

Within each zone one then has to observe whether e.g. in an aerated tank a gradient exists in the oxygen concentration. As long as the oxygen concentration is always well above the saturation coefficient for oxygen used in the kinetic equations, there is no direct need to describe the changes in concentration in the aerobic zone, and the tank can be considered fully mixed. If the concentration of the reacting compounds becomes close to or lower than the saturation constants, the hydraulic model should be such that the change in concentrations is well described. In general, this means using a plug flow model or describing the system as a series of tanks. In the case that the observed concentration of e.g. ammonia in the aeration tank is around 4 mg/L across the tank, it can be considered as a fully mixed tank and be represented in the hydraulic scheme of the plant as a single reactor. However, if the observed ammonia concentration changes from 4 mg/L at the inlet to the aeration tank to 0 mg/L at the outlet, this indicates a strong concentration gradient within the tank, and consequently it is much better to model it as a multi-compartmental tank with a number of fully-mixed smaller reactors in series. A second aspect taken into consideration is the transfer of compounds between gas and liquid (e.g. oxygen) in aerated reactors or between biofilm and liquid.

The main problem is associated with making the hydraulic model of a wastewater treatment plant. A rigorous solution would be to make a full computational fluid dynamics model of the plant, which exactly describes the flow in the reactors. However, in general the details obtained in this way for the flow are far too large for most conversion models. Since we are mainly interested in the bioconversion we need to adequately describe changes in concentrations in the treatment plant. Measuring several relevant compounds can help to define the hydraulic model. For activated sludge models these compounds are in general oxygen, ammonium, and nitrate and, for phosphate-removing systems, phosphate. As a first step a clear division can be made between aerobic and anoxic or anaerobic zones in a treatment plant.

3.2.6 The matrix notation

The balance equation (3.1) can be described for each individual compound. The large number of relevant compounds and conversions make activated sludge modeling complex. A large number of balance equations need to be formulated resulting in a loss of overview. The IAWQ Task Group (Henze *et al.*, 1987) on 'Mathematical Modeling of Wastewater Treatment' has recommended the matrix method for model presentation.

This format facilitates a clear and unambiguous presentation of the compounds and processes and their interaction on a single page. In addition, the matrix format allows an easy comparison of different models, and facilitates transforming the model into a computer program. The matrix is represented by a number of columns and rows; one column for each compound and one row for each process. A simplified example is given in Table 3.1.

The first step in setting up the matrix is to identify the compounds that are of relevance to the model. The compounds are presented as symbols listed at the head of the appropriate column including a row with the dimensions.

The second step in setting up a matrix is to identify the biological processes occurring in the system. These are conversions or transformations that affect the compounds considered in the model and are itemized one below the other down the left-hand side of the matrix. The process rates are formulated mathematically and are listed on the right-hand side of the stoichiometry matrix in line with the respective process. The stoichiometric coefficient for conversion from one compound to another is inserted along each process row so that each compound column lists the stoichiometric coefficients for the processes that influence that compound. If the

stoichiometric coefficient equals zero it is generally not shown in the printed matrix to maintain clarity. The sign convention used in the matrix for each compound *per se* is 'negative for consumption' and 'positive for production'. In this convention the process rates always have a positive sign. Note that oxygen is negative COD because oxygen accepts electrons: electrons are passed from the substrate to oxygen to form water. Care must be taken of the units used in the rate equations for the processes. The stoichiometric coefficients are greatly simplified by working in consistent units.

Table 3.1 Example of a simple stoichiometric matrix for activated sludge modeling (Henze *et al.*, 1987)

Components i	**1: S_O**	**2: S_S**	**3: X_H**	**Process rate equation ρ_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	$r_i = \sum_j v_{j,i} \cdot \rho_j \ [M_i L^{-3} T^{-1}]$			
Definition of stoichiometric parameters: Y_H Heterotrophic yield coefficient [$M_H\ M_S^{-1}$]	Dissolved oxygen (O_2)	Dissolved organic substrate (COD)	Heterotrophic biomass (COD)	Definition of kinetic parameters: μ_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}]

In the example presented in Table 3.1 the compounds are expressed as COD equivalents. Provided that consistent units are used, continuity can be checked from the stoichiometric parameters by moving across any row of the matrix; the sum of the stoichiometric coefficients must be zero.

This matrix forms a succinct summary of the complex interactions between compounds and processes. It allows alterations in processes, compounds, stoichiometry and kinetics to be readily incorporated. The matrix shows two important process aspects: the reaction equation for each process is represented in the different rows. In the columns of each compound one directly observes in which conversions the compound is involved. By multiplying the stoichiometric factors with their respective rate equations one gets the total conversion equation for each compound. For convenience two extra aspects can be added to the matrix description (Table 3.2). The first aspect is a matrix with the composition in terms of conserved balances, in this case the COD, N and charge balance. Biomass is expressed in the stoichiometric matrix in terms of COD, but it also contains nitrogen. In the composition matrix this is included. Since the composition matrix and the stoichiometry matrix contain all the conserved balances, therefore multiplication of the two matrices leads to zero. Secondly we are generally interested not only in the compounds expressed in the dimensions as used in the model, but also in their measured or observed units. For instance the amount of sludge is usually measured as gTSS and not gCOD. The observed matrix contains these conversion numbers between e.g. gCOD and gTSS. Other potentially interesting observed quantities are Kjeldahl nitrogen, VSS or BOD.

Table 3.2 Example of a stoichiometric matrix for activated sludge modeling (adapted from Gujer and Larsen, 1995)

Component	Oxygen	Inert	Substrate	Ammonia	Alkalinity	Biomass	Inert	Substrate	TSS	Rate
Symbol	S_O	S_I	S_S	S_{NH}	S_{HCO}	X_H	X_I	X_S	X_{TSS}	
Unit	gO_2	gCOD	gCOD	gN	mole	gCOD	gCOD	gCOD	gTSS	
Process	STOCIHIOMETRY MATRIX									
Hydrolysis			1					-1	-0.75	r_1
Aerobic growth	-0.5		-1.5	-0.08	-0.005714	1			0.9	r_2
Lysis				0.07	0.005	-1	0.2	0.8	-0.12	r_3
Conservatives	COMPOSITION MATRIX									
ThOD-COD	-1	1	1	0		1	1	1		
N		0.02		1		0.08	0.05	0		
Charge				0.071429	-1					
Observables										
TSS						0.9	0.9	0.75		

3.3 The stepwise development of biokinetic model: ASM 1

Model development is a step-wise, bottom-up process in which only the strictly necessary processes for the pre-defined purpose of modeling are included. Starting simple and increasing complexity when needed is the general governing principle in model development. In general, activated sludge models from the ASM family are developed to describe the oxygen uptake rate and sludge production (coupled with COD balance), and N and P conversions at domestic wastewater treatment plants. However, despite the fact that they are designed for practical (and therefore not academic) purposes, they are not sanitation models as they do not describe the removal of pathogens. Probably the best way to describe the stepwise activated sludge model development is the original approach of Ekama and Marais (1978), later depicted by Dold *et al*. (1980), and further elaborated on in Gujer and Henze (1991). The outcome of this approach is the model which comes close to ASM1 and as such is briefly described here. The experimental system used in this approach comprised a completely mixed activated sludge system using settled domestic wastewater, and basic influent and sludge characterization and operational conditions are listed in Table 3.3.

Table 3.3 Experimental system summary data according to Ekama and Marais (1978)

Parameter	Value
Feeding regime	12 h/d on between 02 and 14 hrs
Flow	18 L/d
Reactor volume	6.73 L
Processes	COD removal and nitrification (fully aerobic)
Biomass content in the reactor	1,375 mgVSS/L or 2,090 mgCOD/L
Sludge retention time (SRT)	2.5 d
Operating temperature	20.4°C
Influent COD concentration	570 mgCOD/L
Influent TKN concentration	46.8 mgN/L

The objective of the study was to use the model to correctly describe the biomass content in the system, oxygen uptake by the biomass and nitrogen conversion. To begin with, one can use a very simple model consisting of only three relevant components (dissolved oxygen S_O, dissolved organic substrate S_S, and heterotrophic biomass X_H) and two relevant conversion processes (aerobic biomass growth and lysis). With an increase in SRT, the biomass (live organisms) as a fraction of the sludge mass (VSS) in the system decreases. To describe this, lysis process or death regeneration was used i.e. disintegration of death cells resulting in generation of soluble biodegradable substrate available for generation of new biomass. Lysis of heterotrophic biomass here summarizes all the processes which lead to a loss of biomass (decay, lysis, endogenous respiration, predation etc). Maintenance or endogenous decay could have also been used here to describe the reduction in biomass. For the aerobic growth process all three components are relevant; dissolved oxygen and organic substrate are utilized by the biomass under aerobic conditions (thus negative coefficient) to produce biomass (positive coefficient). In general, a matrix can be simplified if one can choose to freely assign one stoichiometric coefficient to each process with a value of +1 or -1. The choice of the yield coefficient Y_H (0.67 gCOD/gCOD) together with the COD conservation equation is sufficient to determine all the stoichiometric coefficients for aerobic growth (Figure 3.5).

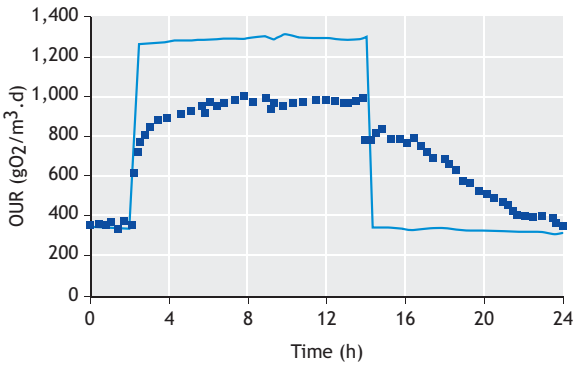

Figure 3.5 Model A: comparison of experimentally observed values (data points) with theoretically predicted oxygen uptake rate (continuous line) (adapted from Ekama and Marais, 1978; Gujer and Henze, 1991)

For both processes one can define the rate; for aerobic growth it is a product of maximum specific growth rate, the affinity of substrate and the biomass concentration (assuming that the oxygen is not limiting the growth). For lysis, it is a 1st order process in which biomass falls apart in proportion to the biomass concentration present and the constant of proportionality is called the rate constant for decay. Substituting the coefficients in the biokinetic model gives the matrix for model A (Table 3.4).

Table 3.4 Matrix presentation of the model A

Component	S_O	S_S	X_H	Rate
Growth	-0.5	-1.5	1.0	$\mu_H^{max} \cdot \frac{S_S}{K_S+S_S} \cdot \frac{S_O}{K_{O,H}+S_O} \cdot X_H$
Lysis		+1.0	-1.0	$b_H \cdot X_H$

If this model is used to compare the experimentally observed oxygen uptake rate (OUR), it can be seen that experimental observations strongly deviate from the model predictions, except for the period between 0 and 2 hours and at the very end of the experiment (endogenous respiration). In general, if the model predictions are deviating considering the levels of parameter of interest, it can be relatively straightforward to adjust by changing the value of the selected model parameter(s). However, if the model predictions in terms of trends and shapes are wrong, very likely the relevant process or processes are overlooked. In this particular experiment, the difference between the total oxygen consumption observed over 24 hrs and the one predicted by the model seems to be quite close. However, it was the deviation between the model prediction and experimental results that led Dold *et al*. (1980) to suggest splitting the degradation of organic matter in wastewater into two processes (fractions): the relatively quick process of biodegradation of part of COD comprising of organics such as VFAs and glucose, and the relatively slow process of COD degradation (cellulose, starch, proteins etc). This fractionation of biodegradable COD to readily and slowly biodegradable COD (RBCOD and SBCOD, respectively) was triggered by the experimental observation of the OUR profile which showed a very sharp drop almost immediately after feeding stopped (14 hrs), followed by a slow decrease observed until the end of the experiment where it reached the steady value observed during the first two hours of the test. Therefore, the lysis process was reasonably well described by the model. It was furthermore concluded that the SBCOD is converted into RBCOD by the relatively slow process of hydrolysis (Figure 3.5). This implies that there is a need to extend model A by introducing two types of substrate (RBCOD and SBCOD) and one additional process (hydrolysis). In ASM1 it was also assumed that slowly biodegradable substrate consists fully of particulate substrate (X_S), which is not necessarily true, but in ASM1 it is accepted as such. A distinction between soluble (S) and particulate (X) material is necessary in order to determine which compound will settle in the clarifier and which will leave the system with the effluent. The introduction of a SBCOD fraction (X_S) did not affect the heterotrophic growth process as it is assumed that growth is not directly based on SBCOD. Also, the lysis processes were adjusted by the assumption that the lysis products are slowly biodegradable and, as such, are added to the pool of X_S. These products are made available for aerobic heterotrophic growth by hydrolysis. This means that there are two types of particulate substrate: one derived from the influent wastewater, and the second generated by the biomass decay. In some cases these are lumped together (as in this case), while in some models they are taken into account separately. However, both options result in practically no net difference. Furthermore, hydrolysable material X_S is assumed to adsorb onto heterotrophic biomass X_H resulting into a kind of Lagrangian kinetic expression as the rate equation for hydrolysis. So it is the amount of substrate per biomass which is important here (rate limiting) and not the substrate concentration with respect to the bulk liquid as in the case for the RBCOD. By implementing this, Model B was formed (Table 3.5).

Table 3.5 Matrix presentation of Model B

Component	S_O	S_S	X_H	X_S	Rate
Growth	-0.5	-1.5	1.0		$\mu_H^{max} \cdot \frac{S_S}{K_S+S_S} \cdot \frac{S_O}{K_{O,H}+S_O} \cdot X_H$
Lysis			-1.0	+1.0	$b_H \cdot X_H$
Hydrolysis		+1.0		-1.0	$k_H \cdot \frac{(X_S/X_H)}{K_x+(X_S/X_H)} \cdot X_H$

Model B describes the OUR competently (Figure 3.6). However, the predicted activated sludge concentration was 22% less than the measured concentration. This indicated the need to increase the sludge production by introduction of an influent non-biodegradable fraction of COD (being inert and also particulate organic matter which accumulates in the reactor: X_I). The term 'non-biodegradable' is in the context of wastewater treatment used for the compound which is not degraded by microorganisms during its retention in the treatment system.

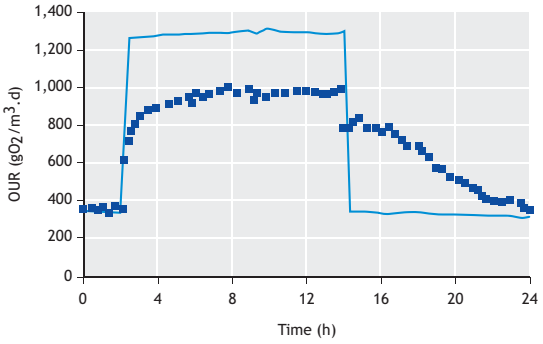

Figure 3.6 Model B: comparison of experimentally observed values (data points) with theoretically predicted oxygen uptake rate (continuous line) (adapted from Ekama and Marais 1978; Gujer and Henze 1991)

Materials such as plastics, wood-based and fibrous materials, nails, and hair are all organic and strictly considered biodegradable, but not in wastewater treatment systems. Even a compound such as cellulose is considered non-biodegradable in high-loaded wastewater treatment plants but biodegradable in low-loaded systems. Besides inert particulate material derived from the influent wastewater, there is also the second component generated by the biomass decay. The latter arises from the fact that cell walls are very slowly biodegradable COD which is considered non-biodegradable, resulting in the experimentally-determined division of lysis process products of 92% being X_s (biodegradable) and 8% X_I (non-biodegradable). Consequently the rates in Model B are not changed given the fact that the OUR profile was described well. Inclusion of X_I resulted in the new Model C shown in Table 3.6.

Table 3.6 Matrix presentation of Model C

Component	S_O	S_S	X_H	X_S	X_I	Rate
Growth	-0.5	-1.5	1.0			$\mu_H^{max} \cdot \frac{S_S}{K_S+S_S} \cdot \frac{S_O}{K_{O,H}+S_O} \cdot X_H$
Lysis			-1.0	+0.92	+0.8	$b_H \cdot X_H$
Hydrolysis		+1.0		-1.0		$k_H \cdot \frac{(X_S/X_H)}{K_x+(X_S/X_H)} \cdot X_H$

The observed biomass concentration in the reactor was predicted well by Model C, however, due to higher sludge (COD) production/removal from the system, oxygen consumption was significantly underestimated by the model (despite the fact that the general OUR profile was well matched, Figure 3.7).

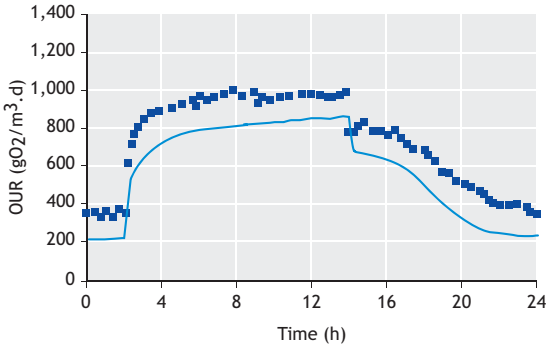

Figure 3.7 Model C: comparison of experimentally observed values (data points) with theoretically predicted oxygen uptake rate (continuous line) (adapted from Ekama and Marais, 1978; Gujer and Henze, 1991)

This is to be expected as oxygen consumption and sludge production are linked through the COD balance, and consequently, an increase in sludge production will cause a decrease in oxygen demand. It was not possible to correctly predict both oxygen consumption and sludge production using Model C. From the experimental observations (results not shown) it was clear that the effluent from the treatment plant contained nitrate which implied that nitrification should be included. By inclusion of nitrification the model had to be extended by addition of three materials and two processes, namely ammonium (NH_4^+, S_{NH}), nitrate (NO_3^-, S_{NO}) and nitrifying autotrophic biomass (X_A), and aerobic (nitrifier) growth and nitrifier lysis. Again, it was necessary to evaluate the influence of each of the additional materials on the existing reactions. Ammonia is not only used in nitrification, but also for cell growth, therefore it was necessary to add a stoichiometric factor for ammonia in the growth relation. If the biomass contains 8% nitrogen (0.08 mgN/mgCOD), then the factor becomes 0.08. Furthermore, it was assumed that in the lysis process nitrogen remains within the biomass. However, in the hydrolysis process the biomass SBCOD is degraded (e.g. proteins into amino-acids) resulting in a release of ammonia. This has as a consequence that, besides one unit of substrate, also 0.08 units of ammonia is produced. This also happens with the influent SBCOD – as the protein part of it is hydrolysed, ammonia is released – the proteins are measured in the influent as organic N, i.e. the difference between the TKN and FSA. So for nitrification, a certain amount of oxygen and ammonia are consumed, and nitrate and biomass are produced. The amount of ammonia consumed is not exactly the same as the amount of nitrate produced, due to dual use of ammonia, namely (*i*) for energy generation in the nitrification process, and (*ii*) as nitrogen source for the heterotrophic biomass growth. The difference between the ammonia consumed and the nitrate formed is 0.08, representing the nitrogen content of the heterotrophic biomass. The overall N balance in this case will fit (4.25 = 4.17 + 0.08 x 1.0). Furthermore, the lysis process for the autotrophs was assumed to be the same as for the heterotrophs, by which particular substrate and a small amount of inert substrate is produced. The process rate for lysis for autotrophs is generated analogously to heterotrophs with saturation terms for ammonia and oxygen. By the inclusion of an additional three materials and two processes, Model D was created which satisfied both the COD and N balance. By having such a model it was possible to split the total oxygen consumption into oxygen consumption for ammonium oxidation and oxygen consumption for COD degradation. This shows the added value of using the model by providing insight into where the oxygen is used and for which processes (so-called process analysis). The results of simulation of the OUR by Model D are presented in Figure 3.8.

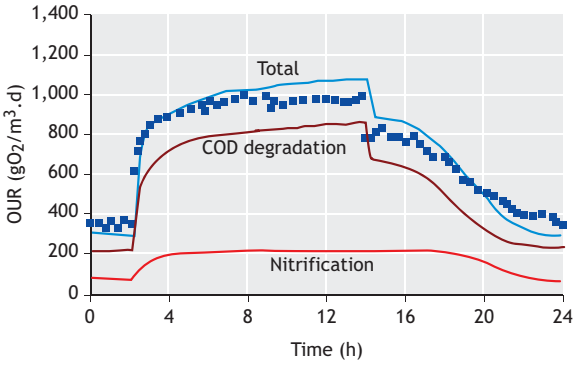

Figure 3.8 Model D: comparison of experimentally observed values (data points) with theoretically predicted total oxygen uptake rate (continuous line). Oxygen uptake for COD degradation and nitrification are separated out (adapted from Ekama and Marais, 1978; Gujer and Henze, 1991)

There were a few dilemmas at this stage of the model development such as 'is the match sufficiently accurate?' or 'is deviation, for example, in OUR of 5 to 10% at 14.00h acceptable?'. The answer to this depends entirely on the quality of the experimental data. If the COD and N balances of the data are exactly 100%, then it may be worth pursuing refinements in the model to get a better prediction because the data is reliable and accurate. If the COD and N balances are not 100% accurate but in the 95-105% range, then there is not much sense in making the model much more accurate. Making models is relatively easy – getting reliable and accurate mass balanced experimental data is the most difficult part of developing models for wastewater treatment systems. If the equipment results in an inaccuracy of 5-10%, it makes no sense to make the model more accurate. One should not forget that a model can only be successful if it fulfils the expectations the modeller has of it. If the purpose of the model is to describe correctly general trends, there is no need for further refinements. Of course, fine-tuning and calibration for a more accurate fit are possible, but this increases the model's complexity. Here, it was decided not to add extra compounds or processes, as the last part of calibration can be done by the straightforward shift of some model parameters. In general, the model simulations showed that the measured line and model line are fitting rather well: OUR, sludge production and nitrification (data not shown, see Dold *et al.*, 1980; Gujer and Henze, 1991) are predicted correctly. For the pre-defined goal, the model is considered correct, but it does not necessarily mean that the assumptions used are correct. Indeed, by using those assumptions, a mathematical description sufficiently appropriate for the purpose of its use is obtained. However, Model D omitted some processes which in reality play an important role, such as protozoa activity (Table 3.7). In this case, it was evaluated that inclusion of protozoa in the model would not increase its descriptive power and consequently this process was not included. The next step in model development was the inclusion of the denitrification process. In general there are two approaches possible that ultimately lead to the same end results. It can be assumed that there is either a special group of bacteria which perform denitrification or that all heterotrophic microorganisms can denitrify, but at a fraction of their rate under aerobic conditions. In other words, there is either a specialized population that can utilize both oxygen and nitrate, and another part of the population that can only use oxygen, or all heterotrophs can denitrify but at a reduced rate, corrected by the η factor (reduction factor for growth rate under anoxic conditions).

Table 3.7 Matrix presentation of Model D

Component	So	S_S	S_{NH}	S_{NO}	X_H	X_S	X_I	X_A	Rate
Growth	-0.5	-1.5	-0.08		+1.0				$\mu_H^{max} \cdot \frac{S_S}{K_S+S_S} \cdot \frac{S_O}{K_{O,H}+S_O} \cdot \frac{S_{NH}}{K_{N,H}+S_{NH}} \cdot X_H$
Lysis					-1.0	+0.92	+0.08		$b_H \cdot X_H$
Hydrolysis		+1.0	+0.08			-1.0			$k_H \cdot \frac{(X_S/X_H)}{K_x+(X_S/X_H)} \cdot X_H$
Autotrophic growth	-18.0		-4.25	+4.17				+1.0	$\mu_A^{max} \cdot \frac{S_O}{K_{O,A}+S_O} \cdot \frac{S_{NH}}{K_{N,A}+S_{NH}} \cdot X_A$
Autotrophic lysis						+0.92	+0.08	-1.0	$b_A \cdot X_A$

Conceptually, these are different assumptions but mathematically they come down to the same equation. Since the last assumption simplifies the model, it has been chosen to use the stoichiometry for denitrification and the bacteria that are ordinary heterotrophic organisms. Although the reality is probably much more complex, it was demonstrated that in reality this simplified approach works satisfactorily. Another important aspect concerns differentiation between fractions of inert material and nitrogen which is one of the items that differs from one commercial model to another. As mentioned earlier, inert material can originate from influent or from degradation of biomass, and the end content of the degraded biomass, inert material, might be different in composition to that of inert material in the influent. This can also be taken into account in the model; the inert material can be either separated or lumped together. In principle, it is not strictly necessary to define these fractions separately, but sometimes it is done based on aesthetic reasons or specific purpose of the model application. Similar reasoning applies for nitrogen fractionation. The model development as described by Ekama and Marais (1978) is still considered valid and Model D extended with denitrification becomes close to the ASM1 (Henze *et al*., 1987). For further details on ASM1 the reader is referred to Dold *et al.,* 1980; van Haandel *et al.,* 1981; Alexander *et al.,* 1983; Warner *et al.,* 1986; Henze *et al.,* 1987, 2000. One of the most important limitations of the ASM is that it does not describe the sludge bulking phenomenon. Therefore, if the ASM is used, for example to improve the nitrification process, it is necessary to check if the proposed changes create bulking sludge. Limited aeration, which is beneficial for nitrogen removal, will almost inevitably induce bulking sludge. Sludge bulking itself cannot be modelled in a way that is sufficiently reliable for implementation in commercial software packages despite some attempts described in the literature (Krebs, 1995). This consequently means that the model cannot be accurately applied to predict very low effluent concentrations when highly efficient processes are implemented. Furthermore, the analogous consideration is valid for denitrification process as well. On the other hand, even if the model is able to predict the concentration of ammonia at 0.5 mg/l, there are always some inaccuracies and imperfections in the analytical procedures for determining ammonia concentration, as well as in the sampling procedure and sample handling. Another ASM limitation is that it does not take into account removal of micro-pollutants such as metals, xenobiotics or oestrogenic endocrine disrupting compounds. This is partially due to the required increase in the complexity of the model and partially due to lack of knowledge on microorganisms and biochemical reactions involved in the conversion of these compounds. In some cases, as in modeling of the wastewater treatment at oil refineries, it is necessary to predict the phenol reduction. To support denitrification, methanol is often added under anoxic

conditions and its conversion needs to be included. And there are cases when, for example, one is interested in sulphite reduction. In all these cases a new specialized organism must be included in the model, as the biomass included in the ASM1 does not convert these micro-pollutants. Examples of such extensions can be found in the literature, and nowadays some commercial software packages include methanol utilization, for example. In the case of other COD compounds such as volatile fatty acids (VFAs), the ordinary organisms which remove COD from the wastewater will convert these and therefore the model does not have to be extended for that. Beyond the ASM1 level, the model can be extended to take into account oxygen transfer, pH and alkalinity, anaerobic digestion, chemical phosphorus removal and precipitation, additional units (like settlers etc), side stream processes, gas phase etc. Again, whether the model needs to be extended depends on the purpose of the model.

3.4 ASM3

In essence, ASM3 describes the same processes as ASM1, although ASM3 was introduced to correct the deficiencies of ASM1. This is partly based on the observations from OUR tests with activated sludge which revealed the fact that bacteria rapidly take-up readily biodegradable COD and store it as internal substrate which will then be converted slowly (conversion of readily biodegradable COD into slowly biodegradable COD). When the acetate (defined substrate) is added to the activated sludge the observed OUR suggests the presence of two substrates; a rapid and slow degradation of substrate associated with OUR can be observed (Figure 3.9).

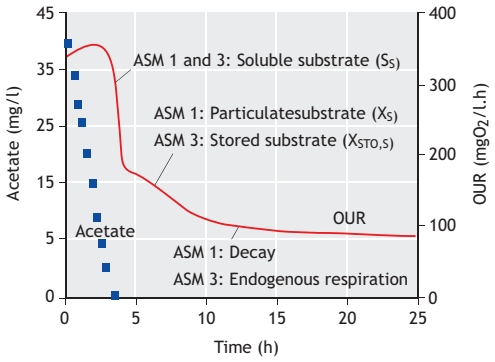

Figure 3.9 Differences between ASM1 and ASM3

In ASM1 it appears as if two substrates are present (S_S and X_S) while in reality only acetate (S_S) is dosed. In order to describe the observed OUR by ASM1 in this case, it is necessary to define that the acetate is partly soluble and partly particulate, which is not recommended. This deficiency is solved by the introduction of a storage compound, $X_{STO,S}$, in ASM3. This means that substrate is taken up rapidly and stored, while growth occurs within the stored substrate. Both models will describe the observed OUR, but only ASM3 will accurately describe the uptake. However, there is no problem in using ASM1 for simulation of nitrogen removal systems because nitrification is a slow process, and thus enough time is available for biodegradation of slowly biodegradable COD.

The second reason to introduce ASM3 was that ASM1 proved to be rather successful for simulation of wastewater treatment plants and consequently too many started to believe that

what was in ASM1 was 100% true and the reality. However, the storage mechanisms exhibited by the biomass show that what is in ASM1 is not all true, but close enough to reality to serve its purpose. Therefore, ASM3 has an added educational value because it demonstrates that there are different (but not necessarily better) ways to model the same treatment plant.

However, the most important reason to introduce ASM3 was the recognition of the importance of three rates of oxygen consumption in the process, namely: the rapid rate of oxygen consumption for degradation of RBCOD, the slow rate associated with degradation of SBCOD, and the even slower endogenous OUR (Figure 3.9). In contrast, in ASM1 there is only one oxygen-consuming process, so it is very difficult to perform calibration as one needs to calibrate other processes that indirectly influence the processes that consume oxygen.

The other problem is the cycling of the COD in the process, as in the decay process particulate COD is produced, hydrolyzed, and used for growth again. It means that if in the process one parameter is changed, it influences all the other processes due to the cycling, and it is difficult to use automated calibration as every parameter has influences on every process. In ASM3 this issue has been solved as the decay process has been replaced by endogenous respiration which eliminates the COD cycle (Figure 3.10).

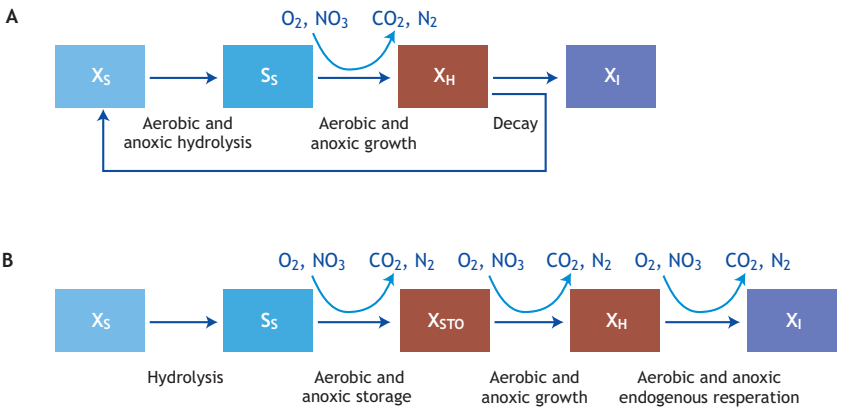

Figure 3.10 Degradation of COD in (A) ASM1 and (B) ASM3

In other words, once the cells are produced, they start to oxidize themselves and by this means the biomass is reduced by the aerobic mineralization process (the classical endogenous respiration). While this has some conceptual controversy, e.g. why would an organism oxidize itself (i.e. go on a diet) when there is food around, it is useful to eliminate the bioprocess interaction from the substrate recycling of the death–regeneration model.

In addition, in ASM3 the oxygen consumption is divided into three processes (storage, growth and endogenous respiration) instead of having only one as in ASM1. ASM3 allows one of these three rates to be fitted if one knows which process to target, which directly links the measurements and calibration parameter. The fact that the $RBCOD_i$ is taken up and stored is irrelevant for most plants (and therefore also the choice between ASM1 and ASM3).

However, the only place where it really makes sense to use ASM3 is in plug flow reactors, such as selectors. If, for example, acetate must be removed in the aerobic selector to prevent sludge bulking, the design of the selector is governed by the time needed to take up the acetate and by the amount of oxygen needed for it. If ASM1 is used instead the oxygen requirements in the selector will be significantly overestimated. In reality a large proportion of acetate is stored inside the biomass, and once it is stored, there is no longer a problem with bulking sludge. If one wants to design the aerobic selector and include it in the model, then ASM3 is the best model to use.

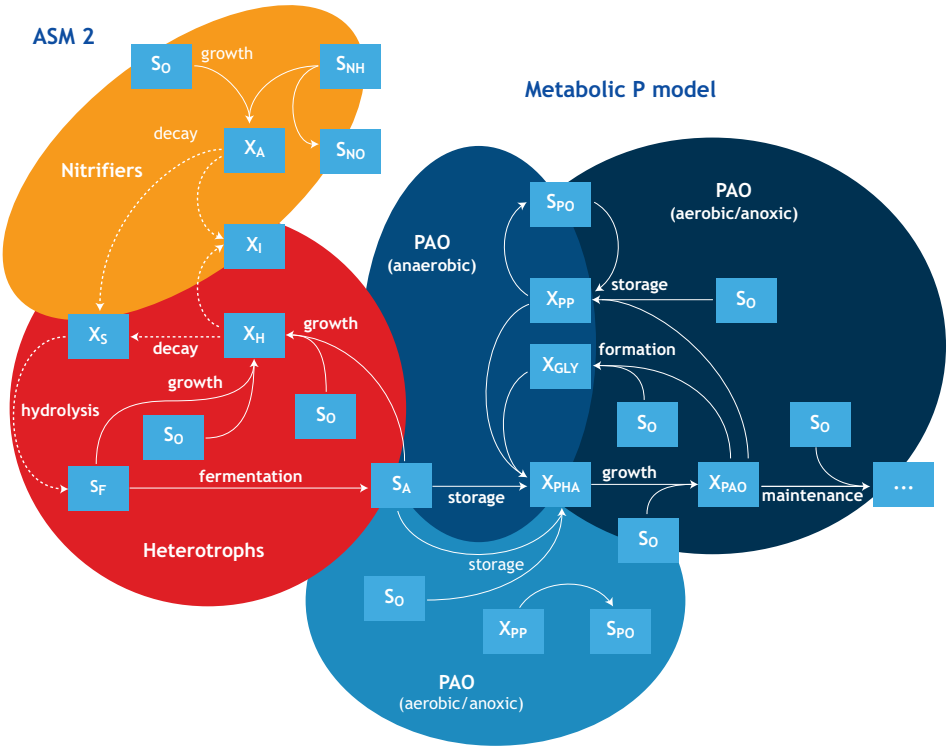

Figure 3.11 Interactions in the integrated TUDP model. The anaerobic conversions are represented by bold lines, and the anoxic and aerobic conversions by thin lines. Conversions independent of oxygen and nitrate are represented by dotted lines (Meijer, 2004)

The other preferred application of ASM3 is for the description of a pre-denitrifying nitrogen removal plant operating at a short SRT. Here, it makes a substantial difference whether or not readily or slowly biodegradable COD is present or whether COD is stored or not. In systems with a long SRT (10-20 days depending on temperature, which are more common in practice), a large part of the nitrate removal is effectively associated with the slowly biodegradable COD from the influent and death-regeneration in the pre-denitrification reactor and from death–regeneration only in the post denitrification reactor, so the sensitivity to the exact ratio between readily and slowly biodegradable COD is much less. The same applies for the differentiation between ASM1

and 3. In highly loaded systems endogenous respiration is less important and accumulation of COD in the form of storage polymers and the carry over in the aerated phase of a treatment plant might be significant.

In conclusion, ASM3 is recommended to be used for (*i*) simulation of highly loaded nitrification-denitrification systems with short anoxic retention times (volumes), (*ii*) selector modeling, and (*iii*) automatic calibration. Otherwise ASM1 should be equally successful in describing the activated sludge plant.

The consequence of introducing EBPR and Phosphorus Accumulating Organisms (PAO) into ASM is that the model becomes quite complex, as illustrated in Figure 3.11. The left side of the figure depicts the part of conversions carried out by nitrifiers and ordinary heterotrophs, while the right side shows the extension needed for the description of the complex physiology of PAO. The nitrifiers and ordinary heterotrophs use oxygen to oxidize their substrate to form CO_2 or nitrate and biomass. They have a rather simple physiology resulting in simple processes. PAO's physiology includes internal storage polymers (PHA, glycogen and polyP) and their behaviour under anaerobic, anoxic and aerobic conditions is different. They also behave differently under aerobic conditions depending on whether the substrate is present or not. Obviously, there are lots of possible variations and inclusion of EBPR in the model substantially increases its complexity (the number of processes in ASM increases from 11 to 22). The situation becomes even more complex when Glycogen Accumulating Organisms (GAO) are also included. ASM2 and ASM2d are similar to ASM1 in assuming the cell to be a black box as opposed to using the metabolic approach to modeling which takes into account what is happening inside the cell.

3.5 The metabolic model

Why is it useful to use a metabolic model? In the standard model for heterotrophic growth there are seven relevant compounds (substrate, oxygen, charge, carbon dioxide, water, ammonia and biomass), five independent balances (carbon, hydrogen, oxygen, nitrogen and charge), and two degrees of freedom. If one knows one yield and one rate coefficient, it is possible to describe the whole system with one model. If one was to describe COD removal and nitrification at a metabolic level, it would not bring any advantage as the yield and rate coefficients would still be needed. Although the metabolic stoichiometry allows tracking, the C, H, O, N, P and charge flows through a system give more information from a modeling point of view, which makes the model more complex but not more accurate. All the rates are linked through conservation relations (stoichiometry) and, therefore, the choices between the process rate or growth rate, and substrate uptake rate or oxygen utilization rate, is not important (Figure 3.12).

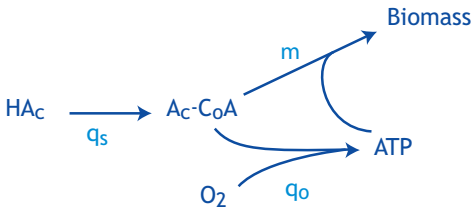

Figure 3.12 Linking of rates through conservation relations in the metabolic model for heterotrophic growth

Thus the black box approach can be used as has been in the case with ASM1. So for the activated sludge system itself, C, H, O and charge tracking is not required - COD and N is enough, but when the ASMs are integrated with anaerobic digestion models to form plant-wide models, it becomes important because AD modeling requires C, H, O and charge tracking to predict gas production and composition and alkalinity generation (Brink *et al.*, 2007)

However, if one needs to describe the situation with heterotrophic growth and product (storage polymers) formation as for PAO, the number of relevant compounds increases; each additional storage polymer brings an extra compound, but the amount of balances does not increase, which means that the degrees of freedom (unknown values) increase as a consequence of the increased number of unknown compounds. In this case one needs to know at least one yield and rate coefficient, and the choice of the process rate becomes important. For example, during aerobic conditions PAO use internally-stored PHA to produce intermediate compound Acetyl-CoA, that is used further for biomass growth, glycogen formation and creation of energy required for these processes, and polyP formation (Figure 3.13).

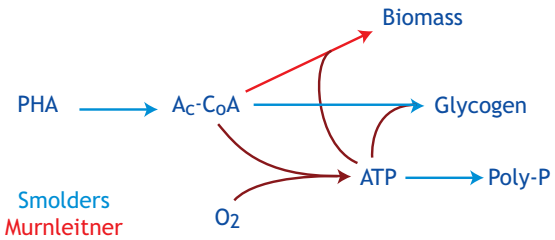

Figure 3.13 Simplified conversions of intercellular storage materials in PAO under aerobic conditions: the approach of Smolders *et al*., 1994, modified by Murnleitner *et al*., 1997.

Obviously, the introduction of storage compounds creates a more complicated network of processes. In this case one needs to choose three process rates. Originally, the metabolic phosphate removal model was created with assumed rates for biomass growth, glycogen and polyP formation (Smolders *et al*., 1994). Later it was observed that this was not the correct assumption and that PAO are regulated in such a way that they consume storage polymers (PHA) at a certain rate and produce glycogen and polyP. The surplus conversion of PHA will be used for growth (Figure 3.13). Thus the more correct approach was proposed to define PHA consumption rate and glycogen and polyP formation rates and to use stoichiometric coupling to calculate how much biomass is formed (Murnleitner *et al.,* 1997).

In the processes with extra storage polymers, extra yield coefficients will also be introduced. The efficiency of the conversion processes would however be the same for all the yields. Within a metabolic description one can link the macroscopic yields to the metabolic yield, which is the efficiency of energy (ATP) generation per unit of substrate oxidized. The substrate oxidation is related to electron transfer to oxygen or nitrate consumption. The yield coefficients are therefore all a function of this basic parameter (ATP produced per pair of electrons transferred) and the number of independent yield parameters is less in a metabolic description for these complex microorganisms.

It is clear that when using metabolic information the degrees of freedom in the model can be reduced. Better understanding of the metabolic processes of the organism will close the gap to a fully glass box situation. The increased complexity of processes is consequently reflected in the models. However, improved understanding of the complex interactions within the cell and the introduction of the metabolic approach gives more confidence and consistency in the application of models to describe activated sludge processes. It is in effect gathering information from a lower level of organization to help understand and model the processes at a higher level of organization. For further details on ASM2, ASM2d, ASM3 and metabolic models the reader is referred to Henze *et al.,* 2000.

3.6 Activated sludge model development history

In this section, the most frequently used activated sludge models are considered to support the modeller in the model selection phase. The focus is on the recent developments of activated sludge models, mainly the family of activated sludge models developed by the International Water Association (IWA) and the metabolic model developed at the Delft University of Technology (the TUDP model). Table 3.8 summarizes the essential features of these and several other activated sludge models.

Table 3.8 Overview of selected activated sludge models (based on Gernaey *et al.,* 2004)

Model	Nitrification	Denitrification	Heterotrophic / autotrophic decay	Hydrolysis	EBPR	Denitrifying PAO	Lysis of PAO / PHA	Fermentation	Chemical P removal	Reactions	State variables	Reference
UCTOLD	●	●	DR, Cst	EA						8	13	Dold *et al*., 1980, 1991
ASM1	●	●	DR, Cst	EA						8	13	Henze *et al*., 1987
ASM3	●	●	ER, EA	Cst						12	13	Gujer *et al*., 1999
UCTPHO	●	●	DR, Cst	EA	●		Cst	●		19	19	Wentzel, 1988, 1989a,b
ASM2	●	●	DR, Cst	EA	●		Cst	●	●	19	19	Henze *et al*., 1995
ASM2d	●	●	DR, Cst	EA	●	●	Cst	●	●	21	19	Henze *et al*., 1999
B&D	●	●	DR, Cst	EA	●	●	EA	●		36	19	Barker and Dold, 1997
TUDP	●	●	DR, Cst	EA	●	●	EA	●		21	17	Meijer, 2004
ASM3-bioP	●	●	ER, EA	Cst	●	●	EA			23	17	Rieger *et al*., 2001

Den. PAO, Denitrifying PAO activity included in the model; DR, death regeneration concept; EA, electron acceptor depending; ER, endogenous respiration concept; Cst= not electron acceptor depending

The ASM1 can be considered as the reference model, since this model triggered the general acceptance of wastewater treatment modeling, first in the research community and later on also in practice. This evolution was undoubtedly supported by the availability of more powerful computers. ASM1 is in essence a consensus model-compromising result of discussions at the time between different modeling groups, most prominently from South Africa, U.S.A., Switzerland, Japan and Denmark. Many of the basic concepts of ASM1 were adapted from the activated sludge model defined by Dold *et al.*, 1980. A summary of the research developments that resulted in ASM1 was given by Jeppsson (1996). Even today, the ASM1 model is still in many cases the state of the art for modeling activated sludge systems (Roeleveld and van Loosdrecht, 2002).

ASM1 has become a reference for many scientific and practical projects, and has been implemented (in some cases with modifications) in most of the commercial software available for modeling and simulation of plants for N removal. Copp (2002) reports on experiences with ASM1 implementations on different software platforms. ASM1 was primarily developed for municipal activated sludge plants, to describe the removal of organic carbon compounds and N, with simultaneous consumption of oxygen and nitrate as electron acceptors. The model furthermore aims at yielding a good description of the sludge production. Chemical oxygen demand (COD) was adopted as the measure of the concentration of organic matter. In the model, the wide variety of organic carbon compounds and nitrogenous compounds are subdivided into a limited number of fractions based on biodegradability and solubility considerations. The ASM3 model was also developed for biological N removal plants, with basically the same goals as ASM1. The ASM3 model is intended to become the new standard model, correcting a number of defects that have appeared during the usage of the ASM1 model (Gujer *et al*., 1999). The major difference between the ASM1 and ASM3 model is that the latter recognizes the importance of storage polymers in the heterotrophic activated sludge conversions. Biomass growth directly on external substrate as described in ASM1 is not considered in ASM3. A second difference between ASM1 and ASM3 is that the ASM3 model should be easier to calibrate than the ASM1 model. This is mainly achieved by converting the circular growth-decay-growth (death-regeneration) model by a growth-endogenous respiration model (Figure 3.11). Whereas in ASM1 effectively all state variables are directly influenced by a change in a parameter value, in ASM3 the direct influence is considerably lower allowing a better identification. Koch *et al*., (2000) concluded that ASM1 and ASM3 are both capable of describing the dynamic behaviour in common municipal plants, whereas ASM3 performs better in situations where the storage of readily biodegradable substrate is significant (industrial wastewater) or for plants with substantial non-aerated zones. The ASM3 model can be extended with a EBPR removal module similar to ASM2 (Ky *et al*., 2001; Rieger *et al*., 2001).

The overview of models including EBPR may start with the ASM2 model, which extends the capabilities of ASM1 to the description of EBPR. Chemical P removal (CPR) via precipitation was also included. The ASM2 publication mentions explicitly that this model allows description of EBPR processes, but does not yet include all observed phenomena; most importantly it is based exclusively on aerobic P uptake EBPR behaviour. The ASM2d model builds on the ASM2 model, adding the denitrifying activity of PAO which should allow a better description of the dynamics of phosphate and nitrate. However, it merely allows P uptake to commence in the anoxic reactor with the same kinetics as in the aerobic reactor – it does not take into account the observed reduction in P removal when significant P uptake takes place in the anoxic reactor (Ekama and Wentzel, 1999; Hu *et al*., 2002). Later EBPR models have sought to address this (Hu *et al*., 2007a,b). EBPR modeling in ASM2 is illustrated in Figure 3.14.

The PAO are modelled with internal cellular structure, where all organic storage products are lumped into 1 model component (X_{PHA}). PAO can only grow on cell internal organic storage material; storage is not dependent on the electron acceptor conditions, but is only possible when fermentation products such as acetate are available. In practice, it means that storage will usually only be observed in the anaerobic activated sludge tanks. The TUDP model combines the metabolic model for denitrifying and non-denitrifying EBPR of Murnleitner *et al*. (1997) with the ASM1 model (autotrophic and heterotrophic reactions). Contrary to ASM2/ASM2d, the TUDP model fully considers the metabolism of PAO, modeling all organic storage components explicitly (X_{PHA} and X_{GLY}), as shown in Figure 3.14. A full description of the TUDP model is given by Meijer (2004) and de Kreuk *et al.* (2007).

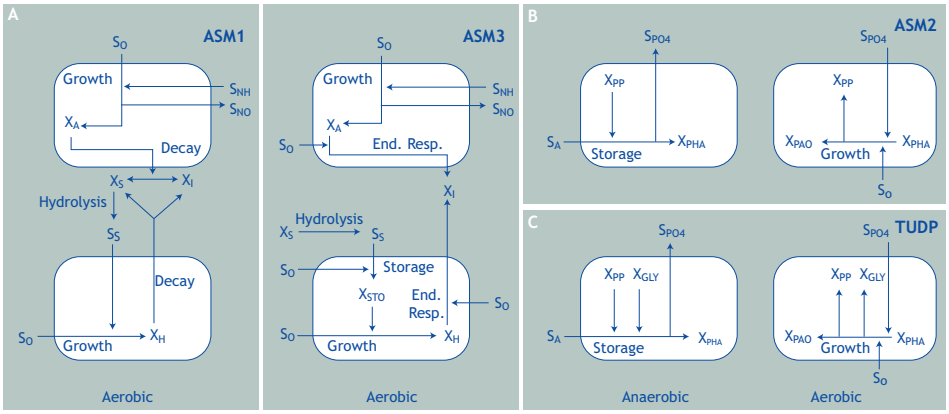

Figure 3.14 Simplified schemes of substrate flows for (A) autotrophic and heterotrophic biomass in the ASM1 and ASM3 models (modified from Gujer *et al*., 1999), (B) storage and growth of PAO in the ASM2 model (Henze *et al*. 1995), and (C) storage and aerobic growth of PAO in the TUDP model (van Veldhuizen *et al*., 1999; Brdjanovic *et al*., 2000). Adapted from Gernaey *et al*., 2004.

In some cases, such as high pH (>7.5) and high Ca^{++} concentrations, it can be necessary to add biologically induced P precipitation to the EBPR model (Maurer *et al*., 1999; Maurer and Boller, 1999a). Indeed, under certain conditions the EBPR reactions coincide with a natural precipitation that can account for an important P removal effect that is not related to the EBPR reactions included in the models described thus far. The formation of these precipitates, mostly consisting of calcium phosphates, is promoted by the high P concentration and increased ionic strength during the anaerobic P release by the PAO. Model equations and components necessary to describe this precipitation process were given by Maurer and Boller (1999b). In general it can be said that the introduction of the ASM model family by the IWA task group was of great importance in this field, providing researchers and practitioners with a standardized set of basic models mainly applicable to municipal wastewater systems, but rather easily adaptable to specific situations such as the presence of industrial wastewater (e.g. Pinzón *et al*., 2007).

3.7 Simulator environments

A wastewater treatment simulator can be described as software that allows the modeller to simulate a wastewater treatment plant configuration. A detailed overview of simulators for wastewater treatment models can be found in Olsson and Newell (1999) and Copp (2002). The Consortium Project Leader has recently co-written the chapter on the choice of models and simulators for WWTP modeling in an educational book which is currently one of the bestselling books of IWA Publishing – the publishing agency of the world's largest International Water Association.

General purpose simulators can be distinguished from specific wastewater treatment simulators. General purpose simulators normally have a high flexibility in their application, but the modeller has to supply (by programming) the models that are to be used to model a specific plant configuration. The latter task can be very time consuming although necessary, as it is needed to debug the implemented model, to avoid running lots of simulations with a model that afterwards is shown to be erroneous for the specific task. As a consequence, general purpose simulators require a skilled user (with programming skills) who fully understands the

implications of each line of code in the models. A popular example of a general purpose simulator is MATLAB™/SIMULINK™ (developed by www.mathworks.com). In contrast to general purpose simulators, specific wastewater treatment simulators usually contain an extended library of predefined process unit models, for example a perfectly mixed ASM1 or ASM2d bioreactor, and a 1-dimensional 10-layer settler model. The process configuration to be simulated can easily be constructed by connecting process unit blocks. Pop-up windows allow the modification of the model parameters.

Examples of specific commercial wastewater treatment simulators are (in alphabetic order, and some of which are presented in Figure 3.15):

- AQUASIM (www.aquasim.eawag.ch)
- BioWin (www.envirosim.com)
- EFOR (www.dhisoftware.com/efor)
- GPS-X (www.hydromantis.com)
- SIMBA (www.ifak-system.com)
- STOAT (www.wrcplc.co.uk/software)
- WEST (www.hemmis.com)

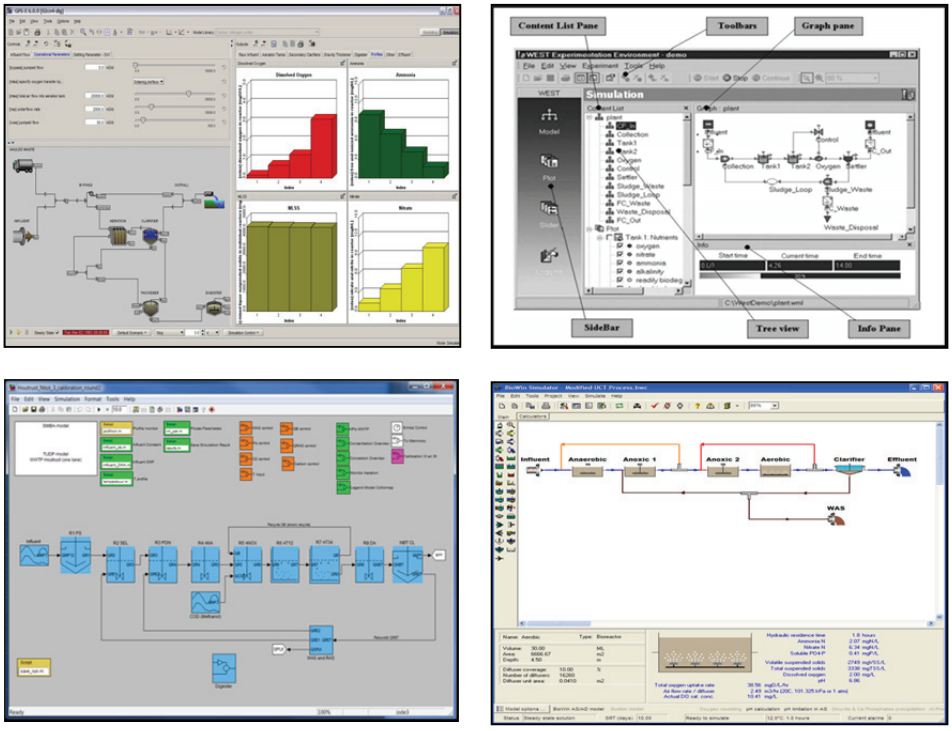

Figure 3.15 WWTP simulators, clockwise from top left; GPS-X, WEST, SIMBA and BioWin

More information about these specific WWTP simulators can be found on their respective websites. On these websites it is often possible to download a demo version of the simulators for evaluation purposes. Specific wastewater treatment simulators allow the modeller to easily produce the desired plant configuration by connecting predefined model blocks (the process flow diagram or PFD). As such, this involves the danger that the user is simulating process configurations without fully understanding the model structure, with the implication that model assumptions and limitations can also easily be overlooked. In that respect, the BioWin simulation software, selected to be used in this project, excels in user-friendliness. For each of the process units introduced in the model, the BioWin software presents practical restraints for the model input data thereby confining the input to realistic proportions and also helping the modeller to understand the function and limitations of the modelled process units.

The selection and justification of the WWTP simulation software used in this project was determined based on the project's objectives and consultant's expertise in accordance to the requirements stated in the terms of reference (ToR). It was stated that in the project, state-of-the-art internationally proffered and worldwide applied family of Activated Sludge Models (ASM) should be used. According to the most recent developments ASM3 is used for WWTPs with secondary level of treatment. This model is extended with ASM2d (or the TU Delft bio-P model) for plants with tertiary treatment including biological phosphorus removal (Table 3.8).

The major objective of the project is to demonstrate the use of mathematical modeling and to present the modeling method. This is demonstrated using steady state WWTP modeling (steady-state retrofit design). This means daily average concentrations and flows are used, rather than dynamic (time dependent) data including variable flows and concentrations. From a technical point of view, steady-state modeling, as required in the ToR, is both the most reliable and most simple design approach. From model design experience in the Netherlands, where the design conditions are very similar to the conditions in Croatia (flat sewers → long hydraulic retention time → low variability → daily averages sufficient), it is concluded that steady-state design simulations are sufficient in most practical cases. In the terms of reference, it is recognized that dynamic modeling can be very useful under specific design conditions. This can be the case with steep sewers (e.g. Swiss conditions), process control design projects, operational troubleshooting, specific effluent discharge criteria, etc. To maintain this possibility for future projects, the selected simulation software should also provide this option. However, the situation in the Black Sea Basin belonging to Croatia and indicated as the project area, is very similar to the Dutch circumstances and therefore, the steady state approach is technically fully justified. Moreover, from the point of view of demonstrating the model-based design methodology, dynamic modeling would make the example not only unnecessarily complicated but also would not add any additional value to the project (all the presented techniques apply both to steady-state and dynamic modeling). For the selection of the commercial modeling software in this project, the possibility of dynamic simulation was indicated as necessary. Dynamic modeling could also be used in Step 4 of the project for the final design of wastewater treatment facilities WWTP Varaždin.

3.8 BioWin WWTP simulation software

For the design and scenario simulations, BioWin (3.1) WWTP simulation software was used. This software is a commercial package developed by Envirosim Ltd. (at www.envirosim.com). BioWin (Figure 3.15) is state-of-the-art and an especially user-friendly (intuitively operated), MS Windows-based simulation software package founded on generally accepted modeling principles according to internationally-accepted IAWQ model standards. The BioWin calculation

model integrates the activated sludge models ASM1, ASM2d and ASM3 and combines these with an anaerobic digestion model (ADM) and several other (specialized) models. Moreover, BioWin includes several sub-models for different process units including primary sedimentation, sludge dewatering, biofilm systems, membrane systems, pH calculation, lime dosage and (metal) precipitation reactions (and more).

Of all the previously presented dedicated wastewater treatment software packages, BioWin is the only software that has been developed specifically to be used by engineers rather than by scientists. Therefore, the BioWin software uses a unique interface which translates the activated sludge model results into practical operational measurements and vice versa. This means the modeller (in this case the wastewater treatment engineer) can use actual measured data directly as input parameters (e.g. TSS, COD. BOD_5, TN, TP) and BioWin will accept this data and translate it to ASM model parameters (e.g. X_I, X_S, S_S, S_I, S_F, etc.). In this process, the modeller will not be experiencing the calculations in the background of the activated sludge model.. Therefore, BioWin makes it possible to use an activated sludge model without having extensive knowledge of activated sludge modeling. For this reason, the BioWin simulator, made by Envirosim Company, is the most widely used software package by operators and process engineers (while other packages may be used more often by scientists or control engineers).

BioWin was selected to act as the preferred software in this project (and for future use in the Croatian context of wastewater treatment design). The BioWin simulator software has been selected because it is widely used by engineers, designers but also by academics and researchers. It has all the features necessary for this modeling assignment and also for future WWTP design projects in Croatia. The BioWin simulator includes an exceptionally quick and stable (user friendly) steady-state model solver and thereby provides all the required options for both steady-state and dynamic simulations. The software is very reasonably priced compared to other commercial packages and can be leased on a use-only basis. The Consortium has extensive experience in using BioWin (and also in the other previously mentioned software packages). No less important is that BioWin is being constantly developed and improved and has an international (English-spoken) professional support service. A team of worldwide professional teachers is available to train engineers and designers in the use of the software under practical (case-related) conditions. BioWin arguably has the most user-friendly interface of all the commercial packages (Figure 3.15) and has been developed for relatively inexperienced users and under practical conditions.

3.9 Conclusions

Models can serve as extremely useful tools in the design and operation of wastewater treatment plants, and in research into the behaviour of these plants. For design, models can provide guidance in identifying the key design parameters and can quantify system parameters to ensure optimal performance. For operation, models can provide quantitative predictions as to the effluent quality to be expected from a design or existing system, and allow the effect of system or operational modifications to be assessed theoretically. For research, models allow testing of hypotheses in a consistent and integrated fashion, to direct attention to issues not obvious from the physical system and thus lead to a deeper understanding of the fundamental behavioural patterns controlling the system response. In this manner models provide a defined framework that can give a direction for further investigations. However, this framework does have disadvantages; it can restrict innovative new developments that do not fall within the boundaries of the framework. Also, in modeling and using models, it should be remembered that models are only our rationalization of behavioural patterns of parameters that we conceive

to be of importance when describing a particular system. Due to this rationalization, the models need to be rigorously verified by conforming to internal mass balances and adequately validated against appropriate experimental tests, and the conditions within which the model is expected to operate successfully need to be clearly defined. A model can be deemed successful if it fulfils the expectations that people have of it.

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NOMENCLATURE

Symbol	Description	Unit
b_A	Specific rate of endogenous mass loss of nitrifying organisms	1/d
b_H	Specific rate of endogenous mass loss of ordinary heterotrophic organisms (OHO)	1/d
F/M	Food to microorganism ratio or load factor (LF)	gCOD/gVSS.d
f_H	Unbiodegradable fraction of the OHO	mgCOD/mgCOD
f_N	Nitrogen content of VSS	mgN/mgVSS
FSA	Free and saline ammonia	mgN/L
K	Half saturation constant	
k_H	Maximum specific hydrolysis rate of SBCOD by OHO under aerobic conditions	mgCOD/mgCOD.d
K_I	Half saturation constant for inhibition compound	mg/L
K_l	External transfer coefficient	m/h
K_N	Half saturation constant for growth of organisms with nitrogen (FSA)	mgN/L
$K_{N,A}$	Half saturation constant for growth of nitrifiers with nitrogen (FSA)	mgN/L
$K_{N,H}$	Half saturation constant for growth of OHOs with nitrogen (FSA)	mgN/L
K_O	Half saturation constant for dissolved oxygen	mgO_2/L
$K_{O,A}$	Half saturation constant for nitrifiers for dissolved oxygen	mgO_2/L
$K_{O,H}$	Half saturation constant for OHOs for dissolved oxygen	mgO_2/L
K_S	Half saturation concentration for soluble organics utilization	mgCOD/L
K_x	Half saturation concentration for utilization SBCOD by OHO	mgCOD/mgCOD.d
q	Specific conversion rate	L/h
Q_{in}	Influent flow rate	m^3/h
Q_{out}	Effluent flow rate	m^3/h
r_i	Observed transformation rate for process i	$ML^{-3}T^{-1}$
S	Soluble concentration in bulk liquid	mgCOD/L
S_{HCO}	Bicarbonate concentration	mg/L
S_I	Soluble unbiodegradable COD concentration	mgCOD/L
S_{in}	Influent substrate concentration	mgCOD/L
S_{KI}	Inhibition compound concentration	mg/L
S_{max}	Saturation concentration	$gCOD/m^3$
S_N	Nitrogen concentration (ammonia or nitrate)	mgN/l
S_{NH}	Free and saline ammonia concentration	mgFSA-N/L
S_{NO}	Nitrate concentration	$mgNO_3$-N/L
S_O	Dissolved oxygen concentration	mgO_2/L
S_{out}	Effluent substrate concentration	mgCOD/L
S_S	Soluble readily biodegradable (RB)COD concentration	mgCOD/L
t	Time	h
V	Reactor volume	m^3
$v_{j,i}$	General stoichiometry term in model matrix for component i in process j	
X	Biomass concentration	$gCOD/m^3$
X_A	Nitrifier biomass concentration	mgCOD/L
X_H	Ordinary heterotrophic (OHO) biomass concentration	mgCOD/L
X_I	Unbiodegradable particulate organics from influent wastewater	mgCOD/L
X_S/X_H	SBCOD/OHO concentration ratio	mgCOD/mgCOD
X_S	Slowly biodegradable (SB)COD concentration	mgCOD/L
$X_{STO,S}$	Intra-cellularly stored organic concentration	mgCOD/L
X_{TSS}	TSS concentration in reactor	mgTSS/L
Y_H	Yield of OHOs	mgCOD/mgCOD

Abbreviation	Description
ADM	Anaerobic digestion model
ASM	Activated sludge model
BOD	Biological oxygen demand
COD	Chemical oxygen demand
CSTR	Complete stirred tank reactor
DO	Dissolved oxygen
DR	Death regeneration
EA	Electron acceptor
EBPR	Enhanced biological phosphorus removal
ER	Endogenous respiration
GAO	Glycogen accumulating organisms
IWA	International Water Association
OUR	Oxygen utilization rate
OHO	Ordinary heterotrophic organisms
PAO	Phosphorus accumulating organisms
PHA	Polyhydroxyalkanoates
RBCOD	Readily biodegradable COD
SBCOD	Slowly biodegradable COD
SRT	Sludge retention time
TKN	Total Kjeldahl nitrogen
TSS	Total suspended solids
TUDP	Delft University of Technology EBPR model
VFA	Volatile fatty acid

Greek symbol	Description	
A	Symbol representing a stoichiometric formula	
H	Reduction factor for utilization of SBCOD under anoxic conditions	
M	Specific growth rate of organisms	1/d
μ_A^{max}	Maximum specific growth rate of nitrifiers	1/d
μ_H	Specific growth rate of OHOs	1/d
μ_H^{max}	Maximum specific growth rate of OHOs	1/d
μ^{max}	Maximum specific growth rate of organisms	1/d
ρ_j	Kinetic rate of process j	$ML^{-3}T^{-1}$



The International Short Course on Modeling of Activated Sludge Wastewater Treatment has been offered in Delft jointly by UNESCO-IHE and Delft University of Technology (TUD) for more than 15 years. From 1993-1996 the course was devised and carried out by the activated sludge modeling pioneer, the late Professor G.v.R. Marais of the University of Cape Town, South Africa. The photo features Prof. Mark C.M. van Loosdrecht of TUD delivering a lecture on modeling protocols as a part of the Masters Programme at UNESCO-IHE to the Sanitary Engineering class 2007/09 (photo: V. Becker).

Chapter 4

Introduction to general modeling protocols

4.1 Introduction

The key factor for the successful introduction of model-based design of wastewater treatment in the Netherlands was standardization of the methods, models and the simulation software. At an early stage it was decided to use one type of commercial software based on the ASM family of activated sludge models. Thus, protocols and guidelines were developed for the practical use (such as presented here). This approach has proven to be effective for the introduction of this technology in the Netherlands where nowadays application of activated sludge modeling is standard practice.

Different protocols were developed by the Dutch water boards, including the protocol for influent characterization and the protocol for model-based design. The usefulness of the guidelines is determined by the simplicity and practical applicability of the proposed methods. Therefore, simple methods have been developed allowing small inaccuracies in the calculations rather than complex methods that may make preforming slightly more accurate but less understandable. The choice was made to provide individual engineers with tools they would also use in practice. Therefore, self-help groups were encouraged and also organized by the Dutch water boards during the first years of introduction of the new model-based design technology. Finally, the support from the senior management for the introduction and use of the technology and the facilitation of extra time for training and education was crucial for the successful introduction in the Netherlands. By doing so, the Dutch water boards have been the main investors in the introduction and standardization of this new design technology in the Netherlands. Currently, most model-based design studies are performed by engineering firms; the application of models has become standard practice and is often required in the terms of reference for WWTP upgrading and design projects. One of the factors determining the successful introduction was the choice of standard simulation software. At that time, SIMBA/MATLAB/SIMULINK was chosen as the preferred simulator, being one of the most flexible and best developed commercial software packages. The criteria on which SIMBA was selected were user-friendliness, price and independency of the developer (Ifak, Germany). The last was seen as very important, for the activated sludge models were not fully developed at that time. Currently, activated sludge models are at the end of the developing stage and widely applied in practice. Therefore this argument no longer holds; most commercial software packages nowadays are based on very similar model (IAWQ ASM type). The choice of BioWin software in this study is based on its competitive price and because BioWin (Figure 4.1) is currently arguably the most user-friendly and widely-applied commercial software package worldwide. It was also selected as the best choice in this capacity building and model-based design project because of the consultants' previous practical experience. The objective of this project is to demonstrate the use of modeling in practice at full scale and to present a methodology (a general protocol) based on our local (Croatian) experience that can be used for WWTP modeling and the application of wastewater treatment assessment, upgrade and design.

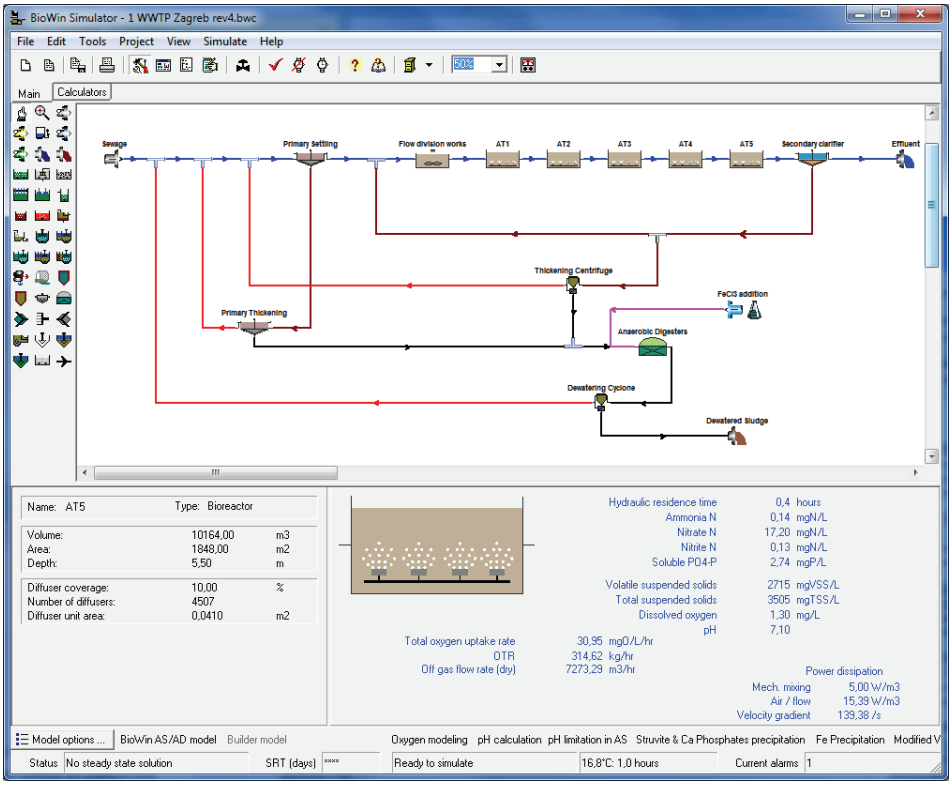

Figure 4.1 Example of a whole plant model of Zagreb WWTP using the BioWin simulation platform

The protocol developed here originates from the STOWA protocol for model design in activated sludge modeling (generally referred to as *Model Based Design* or MBD). This protocol was based on published work by Hulsbeek *et al.* (2002) and Meijer *et al.* (2001a). After its first publication, this work was further applied and studied in practice by Meijer *et al.* (2001b), and based on extensive practical experience with full-scale and plant-wide MBD studies, adjustments in the protocol were proposed. While the STOWA protocol has its focus on activated sludge modeling only, for plant-wide modeling all the relevant WWTP process units need to be included in the model. This requires a different (more general) approach that is covered by the guidelines here.

The general aim of the guidelines (the model protocol) is to reduce the complexity of a model-based design project. Therefore, the design tasks are divided into several sub-tasks. Seven different phases in wastewater treatment plant modeling are distinguished according to the general scheme in Figure 4.2. In the protocol each step is addressed in a standard, practical, feasible and (time) realistic manner. The method was demonstrated in this project in a practical, Croatian context for five WWTPs, namely: Varaždin, Čakovec, Vinkovci, Zagreb and Belišće. In the guidelines the practical and applied methodology is presented in a generic and explanatory way. The aim is for the guidelines to be used as a practical manual and checklist for new design projects, thereby providing a quality standard for modeling design studies in the future development of wastewater treatment in Croatia.

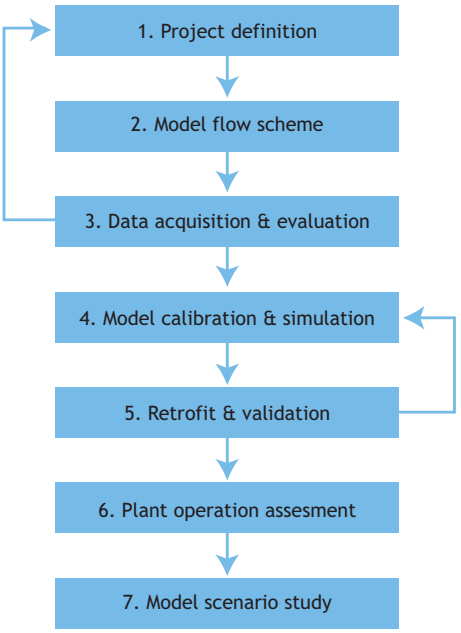

Figure 4.2 General structure of the guidelines showing the 7 phases of activated sludge plant modeling

4.2 The stepwise design approach

A (model-based) design project generally consists of several successive design calculation steps. The order of these calculations is typically determined by the type of information that is needed to perform these calculations; e.g. first the influent quality and quantity is assessed, then the sludge loading is determined and only after that, the oxygen requirement for aeration can be calculated. In the protocol, the design tasks are all ordered in logical successive steps that lead towards the end results of the design study: capacity and volumes of the wastewater treatment plant units. Firstly, in the design protocol, the more traditional performance indicators are calculated e.g. the sludge loading and SRT (sludge retention time or sludge age). Following on from this step, (static) empirical design models, typically in MS Excel, are calculated. Finally, and only if this is required by the nature of the study and allowed by the accuracy of the information, a detailed (and also more complex) process model can be calculated. Therefore, the successive steps in the protocol provide all the necessary design information needed to finally preform, calibrate (correct) and validate (check) the model simulations. Models are typically first simulated in steady state (static simulation) and, but only if needed, dynamic (time variant) simulations are also performed. However, for most model design studies the static approach will be sufficient. Working towards a more detailed plant design (typically in the final stage), a more detailed process model can be developed giving more detailed results on the plant performance.

In the five previously performed case studies (Varaždin, Čakovec, Vinkovci, Zagreb and Belišće), all the steps in the protocol are demonstrated. In the guidelines presented here, the design steps were made largely generic for general model-based design studies, disregarding the type of modeling software used. As a result of the selected modeling software, some of the

presented methods and calculations are specific to the use of the BioWin simulation software. BioWin was used in the five case studies that were presented in this project. The BioWin software is a typical full plant simulator providing the possibility to combine the most common WWTP process units into one holistic model, thereby modeling the complete wastewater treatment plant rather than only the activated sludge reactors. Constructing a full plant model requires more effort and time for analysis and data processing and also a larger financial investment, mainly for additional analytical measurements. Generally, a full plant model (plant-wide model) provides much more valuable information for decision-making. The plant-wide model approach allows for more precise (and more efficient) design that reduces the risks from design shortcomings, possibly resulting in costly operational failures. In the MBD protocol empirical design rules (e.g. the German ATV design method and STOWA design methods) and BioWin simulations are used together and are fully complementary to assure the quality of the plant design under all conditions.

4.3 Structure of the guidelines

The guidelines describe a model design project from the start-up phase up to the conclusive model scenario study. Thereby seven phases can be distinguished in the project.

4.3.1 The inception phase

In this first phase, the main goal is to get an accurate and complete project overview. The main goal of this phase is a definition of realistic objectives and to make a realistic project plan for the given time and project budget. Because the model-based design project relies on actual measured data, it needs to be established whether this data is readily available from historical sources and what the quality of such data is. Often additional measurements will be required, and this will affect the planning and project budget. At this stage the problem and the research objectives are defined. This directly determines the physical boundaries of the process that will be studied. To do this efficiently, a process flow diagram (PFD) of the WWTP is made. Also an inventory is carried out of all the available plant information. Therefore, a plant visit is always recommended. Probably not all the required data can be gathered at this stage; however indicative samples of the data can be acquired to get a first impression on what can one expect of the data collection phase. All the information that is needed to be collected during the inception phase is described in the guidelines (templates/checklists). The result of the inception phase is a complete documentation of the plant inventory including the most important design information, a list of the historical operational data and indicative (average) operational plant information, a process flow diagram and all other collected information that could be of relevance for the rest of the project (e.g. contact data, photographs, guidelines, design reports etc.). Therefore, the inception phase should be seen as a smaller project on its own within the larger modeling project. The time invested here is approximately 4 to 5 days including (at least) one day for visiting the plant. In short, the inception phase covers the following activities:

- Definition of the project objectives;
- Definition of the process lay-out in the process boundaries;
- Collection of initial (estimated) process information;
- Documentation of the project starting points.

4.3.2 The initial model construction

The information collected on the plant in question can be used to construct a preliminary plant model. Based on the definition of the objectives, it will be clear what the specific goals of the project are and, thereby, also the preferred simulation platform and activated sludge and/or

anaerobic digestion models. From the process diagram, a (typically simplified) model flow diagram is constructed in the simulation platform. At this stage no calibration is necessary. After 'feeding' the model with the plant design data and preliminary flow data, it will be possible to do an initial simulation of the plant. It is very unlikely that the first simulation will lead to reliable results, however the goal of this step in the protocol is to get an early indication of whether the assumed flow scheme is correct, and also whether the initial provided operational data (at this stage mainly the flow balances) can be fitted in the model. If this is not possible, this can indicate that the preliminary data is incorrect and/or unreliable. In that case, action should be undertaken to acquire better data and information. Testing the data in an early project phase provides the possibility for early feedback to the plant operators and the possibility to resolve this potentially critical problem at an early stage. The model construction phase is a desk study that can be performed within 1 to 2 days. In short, the model construction phase covers the following activities:

- Choosing the simulation platform;
- Constructing the model flow schemes;
- Constructing a non-calibrated model based on non-verified data;
- Checking information on major errors by running a preliminary (quick scan) simulation.

4.3.3 Data acquisition and evaluation

The main goal of this phase is to acquire reliable plant information. Therefore, the historical operational data is collected and checked using mass balance calculations. Also additional measurements are planned with the goal of checking the main operational parameters, mainly flows and the sludge production. Moreover, during this phase, plant data is acquired to be used for the design calculations and in the simulation study as model input, and to validate the model simulation results. Therefore, a comprehensive measurement plan should be made which includes the minimum required information on sampling and analytical measurements to be able to proceed to the next project activities. This plan is based on an initial mass balance analysis; all the data is measured within a preselected (sub) system of mass balances, determined by the initial definition of the process boundaries, as stated in the report objectives of the inception phase. In the data acquisition phase, the actual plant sampling will be executed. It should be taken into account that this is a critical part of the study and also that it will take approximately 2 to 3 weeks from the start of sampling to the moment the final results can be presented for further processing. Again, the additional measured information will need to be checked for major errors and also, mass balance calculations will be used for evaluating the data. The results of the final mass balance calculations will be reported and will serve as input for the already constructed model. An initial plant assessment is also possible, based on the checked information. Therefore, multiple empirical design calculations, as well as calculations of traditional key design parameters such as the SRT, sludge loading, oxygen consumption and also more complicated static empirical design methods (e.g. published by STOWA, WEF or ATV) are applied here. The retrofit design phase can also be performed only using the historical data, e.g., without performing a model-based simulation. Depending on the amount of data that needs to be collected, which largely depends on the definition of the project objectives, the data acquisition and evaluation phase will take 2 to 4 weeks including all the practical work related to the sampling and laboratory analysis. In short, this phase covers the following activities:

- Flow balance analysis of the plant;
- Designing a sampling campaign;
- Executing the sampling and lab analysis;

- Collecting detailed historical operational plant data;
- Performing mass balance calculations;
- Detecting major errors in the measured and historical plant data;
- Reporting a checked and balanced data set;
- Calculating an empirical design using the measured data;
- Conducting an initial plant assessment.

4.3.4 The model simulation and calibration phase

At the beginning of this phase a non-calibrated model is already available. A data report is also available which contains the checked and balanced input data that is ready to be used for the model input and calibration. When the model is calibrated and preformed as expected (according to the checked and balanced data), a model calculation report is made in the form of overview/summary tables. This phase of the modeling project is described by the STOWA protocol as presented in these guidelines. The STOWA protocol handles in detail all the steps needed for the construction and calibration of the activated sludge system. From a mathematical modeling perspective, the activated sludge model is the most complicated part of the total (plant-wide) model, and therefore the additional attention when calibrating the activated sludge system is justified. In the chapter dealing with the STOWA protocol (Chapter 7), advice is also given on the steps necessary to fit the model to the measured plant data. The time needed to invest in this phase heavily depends on the experience of the modeller. An experienced modeller will generally be able to produce a calibrated model and report the results within 5 days. If an inexperienced modeller is working on the project, 2 to 3 additional weeks should be reserved for the modeller to get acquainted with the use of the software, the background of ASM models and model calibration, and probably follow a training course on this topic. In summary, this phase covers the following activities:

- Loading the model with checked and balanced input data;
- Performing steady-state simulations;
- Calibrating the model;
- Reporting the calibrated model results.

4.3.5 The model retrofit and validation

In this phase of the project, the calibrated simulation results are subject to multiple empirical design calculations (as was the measured data in the previous stage), and calculations of traditional key design parameters such as the SRT, sludge loading, oxygen consumption and also more complicated empirical design methods (e.g. published by STOWA, WEF or ATV) are also applied. This serves three main goals, namely: (i) a plant operational assessment, (ii) checking the information for accuracy and detecting inconsistencies, and (iii) determining whether the model simulation results are within acceptable design boundaries. For the model validation, the checked and balanced data, including key operational and design parameters, is compared to the results of the calibrated model. The design data is typically presented in a double column (side-by-side) table for easy comparison. In principle, the retrofit design can be performed only on historical data without performing model-based simulations. For the majority of (simple) studies the plant assessment can be based on the traditional retrofit design calculations which are sufficient for the defined project objectives. Performing a full model-based study is only advisable in cases where the traditional approach is not sufficient. On the other hand, it should be taken into consideration that when an extensive measurement campaign is preformed, execution of the model simulations is only a little extra work that may provide a lot more detailed and accurate information on the plant operation. In conclusion, it is advisable to decide

at an early project phase (before commencing with plant measurements) whether modeling is necessary (preferably at the first phase of this protocol). Depending on the experience of the designer and availability of design tools (e.g. a template MS Excel spreadsheet with all the required calculations), the retrofit design, which is a desk study, will take 2 to 5 days. In short, this phase covers the following activities:

- Preliminary retrofit design calculations based on historical plant data;
- Preliminary plant operational assessment based on historical plant data;
- Empirical design calculations based on the results of the calibrated model;
- Presentation of the historical data and model results together;
- Validation of the model results.

4.3.6 The operational plant assessment

During this stage, the major errors are filtered out of the measurements. However, smaller model calculation errors and measurement errors will still be present, probably resulting in small differences between the checked and balanced data and the data coming from the calibrated model. These (smaller or sometimes larger) differences are a useful indication of the accuracy of the design study; in the final design proposal the appropriate safety factors will be determined based on these results. A safety factor is an introduced margin of uncertainty on e.g. proposed volumes and plant capacities, ensuring that the design takes into account the model uncertainties. Therefore it is advisable to indicate precisely where the model does not fit the data as this information is valuable for the final phase of the design. In the process of comparing the historical data and the model results, firstly the model is validated (or not) and after that the final operational plant assessment is made. The results of all the design calculations are carefully evaluated and conclusions are drawn on the plant operation. Interpreting the design data requires expert judgment. Assessing the plant performance will take 1 to 2 days depending on the experience of the designer. In short, this phase covers the following activities:

- Making an operational plant assessment;
- Reporting the plant assessment.

4.3.7 The model scenarios

In the preceding steps, only the current operation of the wastewater treatment plant was evaluated. In the scenario study, the validated model can be used to predict the plant operation under different conditions (e.g. increased load, additional volume, different temperature, etc). This phase is fully dependent on the definition of the objectives and will be different for each new study. The general advice is to strictly define the amount and type of simulation runs (scenarios) that will be studied; typically 5 to 8 scenarios is the maximum. Above this number, the amount of presented data will make it difficult to distinguish between the different scenario calculations and the whole study becomes meaningless. Depending on the type of scenario study, a typical study presenting 5-8 scenarios can take up to 5 to 8 days.

The 7 modeling steps are summarized in Figure 4.3.

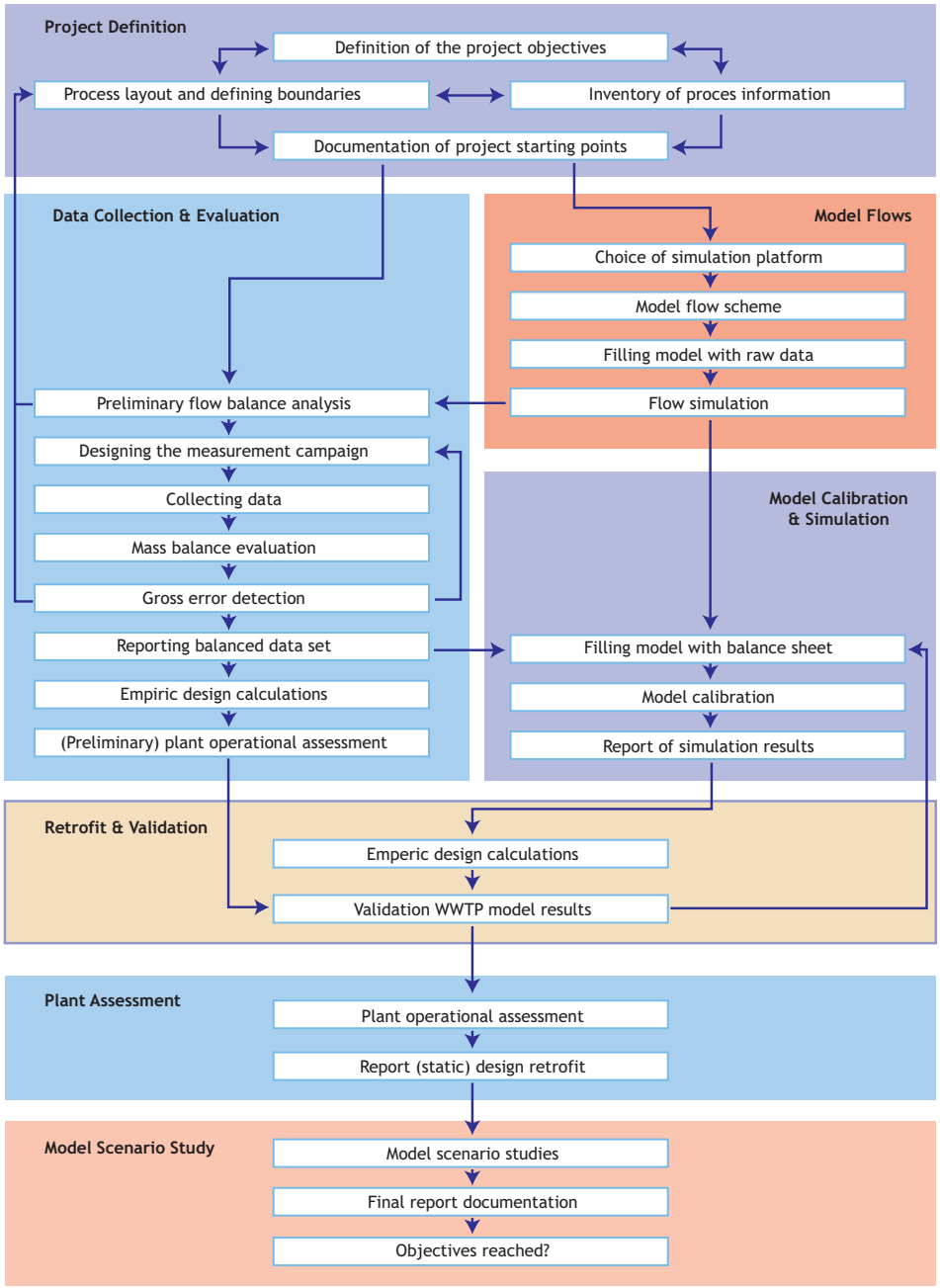

Figure 4.3 The general but more detailed structure of the guidelines showing the 7 phases of activated sludge plant modeling.

4.4 Introducing the STOWA protocol for activated sludge modeling

The original STOWA protocol for modeling activated sludge systems was published by Hulsebeek *et al.* (2002) and is also based on the work on influent characterization by Roeleveld *et al.* (1999) and Meijer *et al.* (2001). The protocol is generally applied in practice as depicted by Figure 4.4. The protocol gives a logical sequence of activities needed to model a wastewater treatment plant, such as formulation of modeling objective(s), plant description, data collection, construction of the model, flow characterization (using specific protocols also presented in this report), model fine-tuning (calibration), model verification using other sets of data (validation), and reporting. This approach has been applied and verified in many modeling studies and adopted as good modeling practice.

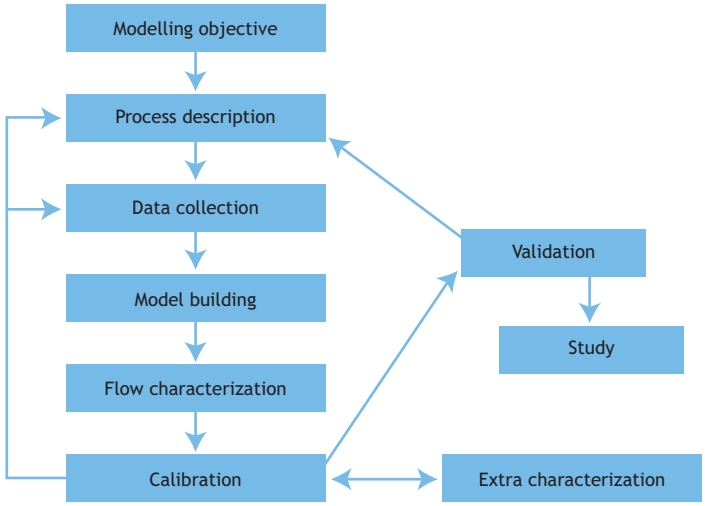

Figure 4.4 The STOWA modeling approach for constructing a calibrated model describing the activated sludge system. In the plant-wide model, this scheme can be applied to model the water line only.

At first glance the STOWA protocol looks different from the approach proposed in these guidelines. However, it should be taken in account that the guidelines cover a much broader scope than only activated sludge modeling. The STOWA protocol is mainly focused on modeling the water line (activated sludge system). The guidelines also cover organizational and logistical matters on data collecting, planning time and resources, presenting information, methods for data evaluation, etc. Also from the technical perspective, the guidelines have a broader scope than only describing the water line; they focus on describing the full plant model (WWTP model or plant-wide model) which also often includes anaerobic digestion, primary settling, thickeners, secondary settlers, internal process water flows, etc. Thereby the STOWA protocol supports the guidelines with practical, detailed, structured and stepwise applicable advice on modeling the activated sludge plant (water line). Practical use of activated sludge models for wastewater treatment plant design and upgrade was encouraged in the Netherlands in three ways; first of all, in 1995, the simulation software SIMBA® was chosen by the Dutch Foundation of Applied Water Research (STOWA) as the preferred software. This made it possible for a country-wide

community of users to form and the opportunity to build national experience with the use of (dynamic) simulation software. Following this success, in 1998 STOWA recommended the use of a standard methodology for characterizing the influent (published by Roeleveld *et al*. in 1998). In 2002 STOWA introduced a protocol for activated sludge modeling. Since 1995 almost all wastewater treatment plants in the Netherlands have been modelled by staff from different water boards, with or without the support of consultants. With wider use of the technology, the need for a better standardization and some form of quality control also became apparent. This led to the development of a standard methodology for characterizing the influent and for developing a calibrated model. In both cases, easy-to-use practical methods were preferred to scientific exactness. Originally, the simulation studies started with a long (1 to 4 week) intensive measurement campaign collecting and analyzing hundreds of samples, and even potentially checking many of the model parameters by respirometry or other tests. This is an expensive and lengthy procedure. It can be shown that many parameters in full-scale systems are not very sensitive (although they might be so in e.g. laboratory batch experiments) and therefore an experimental evaluation of these parameters in practice is often not useful. The same is true for some of the measured concentrations. Here it should be noted that under some conditions (e.g. industrial wastewater), biological limitations can occur and these can be useful specifically for troubleshooting. For most wastewater treatment plants however, it is fully justifiable to develop the model based on historical data and correctly executed measurements, thereby saving much time and cost and making the use of modeling in everyday practice more feasible. When an initial model is developed (preliminary simulations), it can be used to evaluate which parameters should be measured more, based on the sensitivity to model parameters or variation over the day. During this stage the readily available flow variations are already giving sufficient dynamic conditions for simulation.

In these guidelines, the STOWA guidelines for dynamic modeling of activated sludge systems are introduced with the objective of providing an univocal protocol for modeling of activated sludge systems (being an important and technically difficult part of the full modeling protocol), that will give a standardized and easier transfer of knowledge. The main purpose of the protocol was to reflect the present experience and knowledge in a structured way so that the simulation models are used in such a manner that quality control becomes possible. A more standardized use of dynamic simulations is essential because it is increasingly being used for optimization studies and for studying different aspects in the design phase of WWTP. Not only the quality control but also the planning should therefore be improved. The amount of sampling and testing should be kept at a satisfactory but minimal level to allow for a cost-effective use of models. An exemplary use of the proposed scheme can be found in scientific literature (e.g. Brdjanovic *et al*. 2000, Meijer *et al.* 2001a, b, etc.).

4.5 Introduction to the influent characterization guidelines

Influent characterization is an important step in the effective application of the ASM models. The characteristics were commonly determined according to the STOWA protocol that are also presented and explained in these guidelines (Roeleveld, 2001; STOWA, 1996; STOWA, 1999). The influent characterization is mainly based on applying filter methods to raw measured influent, thereby determining the slowly degradable COD. A BOD measurement of the influent is carried out to determine the inert (non-biodegradable) COD. In practice, the influent characterization is not a difficult analysis to make, and therefore is suitable to be taken up in the daily (routine) practice of operational data acquisition of wastewater treatment plants. Although one can argue whether the filter methods proposed by the STOWA method are accurate, in practice the full-scale simulation models are not too sensitive for the division between slowly and readily

degradable COD. Only the estimation of the inert COD from a BOD test poses problems. By measuring the $BOD_{1,2,3,4,5,7,10}$ (Roeleveld *et al*. 1998) and essentially using the model to describe the BOD as a function of the incubation time, the method becomes more reliable. Nevertheless it also becomes laborious and is still vulnerable to analytical errors. It also seems from detailed calibrations that the fraction of inert COD in the influent depends on the sludge age of the WWTP. Because of all the uncertainties, we propose using the fraction of inert particulate COD in the influent as a main calibration factor. It is concluded that the wastewater characterization methods are not difficult to apply. Valuable information from these methods can be obtained not only on the influent but also on the expected plant performance. Therefore, it is recommended to incorporate such influent characterization measurements into the standard practice of operational data acquisition on a regular basis, e.g. every two weeks.

4.6 Introduction to model calibration

In the modeling protocol all the successive steps lead to the calibration procedure. The detailed procedure to calibrate the activated sludge system is described in the calibration protocol included in the guidelines. To fit the model results to the reality, the protocol indicates specific model parameters to be adjusted within an acceptable range. With the protocol at hand, the often complex task of model calibration becomes a controlled and reproducible exercise. In general, it is unlikely that the first simulation will match the observed values; usually a model will need to be calibrated. Based on practical experience it is concluded that nowadays the ASM models are advanced to the level that any 'normal', well described, domestic wastewater treatment plant can be successfully modelled by changing (calibrating) only a few preselected model parameters. If the required changes are very different from the default parameter settings, it is highly likely that the model description or input is not correct. In that case, the input data has to be fixed first before proceeding with calibration. The calibration protocol itself provides a base for problem solving. When the model cannot be fitted to the measured data within a reasonable parameter range, this indicates the actual plant is working differently than the collected information is suggesting. This will require looking back and evaluating the processed data and finding the error. Here, the plant model (that cannot be calibrated in a reasonable range) gives the direction for where to find the problem in the measurement data (e.g. incorrect analytical measurement) or in the actual process (e.g. incorrect process description). This needs to be addressed before the model is used for further calculations. Thereby, the calibration protocol provides a useful tool that can be applied to increase the accuracy of the model study and to obtain reliable results within a controlled time frame. When the model is calibrated, the operation of the WWTP can be simulated thereby matching the measured data to the level of a satisfactory description of the existing conditions. However, these are not the final results of the model study. From this point onwards the model can prove its value by being used for all kinds of extrapolation; e.g. simulating not (yet) existing conditions. These types of 'what if' studies are typically referred to as scenario studies (or scenario analyses). The model is used to check how the plant will perform under full future capacity, in other words whether the plant is indeed capable of reaching the designed effluent standards. If the design is already fully loaded, the model can be used to alter the design and extend the plant to test the effect on the effluent quality. Simulations may also reveal a possibility for operational improvements, savings (energy, chemicals), and an improvement in efficiency in terms of effluent quality, biogas production, sludge minimization, and aeration requirements. The power of preforming realistic (dynamic and static) scenario studies is unique for modeling; there is no other technique by which these type of predictions can be made within the same margin of accuracy. On the other hand, these results can only be achieved if all the recommended steps are precisely followed according to the guidelines including the STOWA protocol. According to

the standard protocol, full-scale practice does not include (laboratory) experimental work, as it is considered too complicated to be performed in wastewater treatment practice. However, when building a knowledge base on the use and application of activated sludge modeling, as is currently being done in the Croatian context, this should also be done in cooperation with training and educational institutions (universities and institutes), and also by stimulating national research that is adopting this kind of experimental work. The introduction of modeling and experimental methods will further support the development of the Croatian wastewater sector. Modern measurement equipment is available nowadays to considerably speed up laboratory batch reactor experiments (batch tests) and greatly facilitate their use in practice. This was (and is) also done in the Netherlands by universities (e.g. TU Delft) and research institutes (e.g. UNESCO-IHE), often stimulated by the Dutch water boards, and has been one of the success factors for the broader introduction of this technology and putting these state-of-the-art modeling approaches into practical use. The following lab-scale batch tests are typically performed during research into full-scale activated sludge systems (Brdjanovic *et al.* 2000):

- Respirometry test;
- Nitrification test;
- Denitrification test;
- Anaerobic P-release;
- Aerobic P-uptake;
- Anoxic P-uptake;
- Endogenous P-release;
- PAO denitrifying activity.

4.7 The stepwise data approach to data acquisition

In the model design protocol that is presented here, a methodology is proposed for structured data acquisition. Information for plant design is acquired in several steps, each with increasing detail and accuracy of the collected data. The first step is the inventory phase where all the information possible is collected ranging from photos, plant visits, lay-outs, operational data, construction data, analytical measurements, aerial photographs, etc. The inventory report (in this study referred to as the Inception Phase*)* typically presents all the collected information in tabular overviews. For this purpose, a generic plant inventory checklist was developed (as presented in the appendix). The second step of the data acquisition involves the collection of more detailed (historical operational) information typically obtained from operational databases (e.g. data stored from SCADA systems). Typical historical data collected (time ranges) is flow measurements, energy consumptions, temperature measurements, control set-points, concentration measurements, waste sludge production etc. In the third step of the data acquisition, additional analytical measurements can be performed (typically grab samples) specifically for the assessment of the plant operation and also to be used in the model study. This data is mainly: the composition of the influent (influent characterization), effluent fractions and the activated sludge composition (activated sludge characterization). In the protocol there is a description of how these additional analytical measurements should be collected. The results are summarized in a practical measurement plan that is ready to be applied.

4.8 Introduction to measurement planning

In general, design projects that involve measuring actual plant data are known to be difficult to plan. Besides all the practical and logistical problems of gathering information on site, weather conditions should also be taken in account. In general dry weather conditions are preferred for

sampling and therefore at least 3 days of dry weather should precede a sampling campaign. On top of that, it will take 1 to 2 weeks before the analytical data has been measured. Therefore in general it will take 3 to 4 weeks from the start of the sampling campaign before the raw measured data is processed and presentable, and before it is known if the sampling and measurements have all been carried out with the required accuracy, and if the campaign is considered useful for the model design study. The main risk in the design project is getting all the required (sometimes not routinely measured) information on time and with satisfactory accuracy to be used for the plant assessment. For the model-based study, extra attention should be given to the project planning and specifically to the planning of the acquisition via sampling. Because measuring and processing plant data is time consuming and also costly, it is worthwhile investing time in thorough preparation thereby also reducing the risk of exceeding the budget and the available time. One of the difficulties of executing sampling campaigns on site is effective communication with the staff (laboratory and operators) on what, where, when and how should samples be taken and analyzed. To facilitate this communication, the protocol includes a sampling program plan together with information on measurement types, handling, sampling location in reactors and flows, sampling frequency and the analytical method and preparation. For budget management, a cost calculation is also included for the analytical work.

An efficient plan for the data acquisition only should include the minimal required amount of information needed for the design study, and at the same time this data should be available with the highest possible accuracy. In practice, the accuracy of the data will determine how easy it is to construct a model and preform the study. Typically, the model designer is confronted with conflicting (operational and plant design) information, which usually causes the less experienced designer to struggle. This was the main reason for the development of the modeling protocol: to familiarize inexperienced designers with a practical stepwise protocol simplifying the design tasks. In the protocol it is emphasized how project information can be collected, also focusing on data accuracy and thereby providing not only a reliable model study, but also making the study easier to perform (which is generally the case when the data quality is better). The modeling protocol presented here provides all the necessary information to plan the project regarding the time and resources, including standards for the MS project planning schemes, table layouts and checklists. Thereby the protocol supports the completion of a model design project in time and within the budget and also provides a basis for a qualitatively well preformed design study.

4.9 Standards for project presentations

A practical and time-saving feature of the design protocol presented here are standards for processing and presenting the often large amounts of data. After gathering the information in several steps, the data is processed and presented in templates - standard tabular overviews. A standard method for presenting the information is proposed for several reasons as the tables serve as a checklist for the required plant information. Not having to come up with a new table format every time saves time. Standardized presentation of data also makes it easier to compare simulation studies, and to use and judge the results. Several sample tables are presented in this project, such as:

- Initial plant inventory data: contact information;
- Initial plant inventory data;
- Reactor volumes and dimensions;
- Hydraulic design (flows);
- Influent concentrations and flow;
- Influent loads;

- Evaluation of the primary settler operation;
- BioWin influent characterization (concentrations of calculated fractions);
- Activated sludge characterization (concentrations of sludge fractions);
- Measurement data overview (concentrations and loads of raw and corrected data);
- Mass balance tables;
- Sludge retention time (design calculation);
- Retrofit presentation table (including the retrofit and model results);
- Secondary clarifier design (inputs and design results of 5 calculation methods).

4.10 Standards for indicating errors and inconsistent information

Inconsistencies in actual plant data will regularly be found and in order to establish realistic safety factors it is essential to keep note of all the (mostly expert-based) data corrections. It should be realized that the introduction of safety factors, however necessary, is also the main factor determining the cost of a plant design or upgrade; there is a direct relation between the implemented safety factor and the volumes and capacity of the process units and technical equipment, and thereby the investment and running cost of a project. For this reason it is proposed in the protocol, after checking the data for consistency, to present the corrected data set in a tabular overview indicating all the adapted measurement errors, estimations and corrected data (in the tables e.g. printed bold or underlined). This final and corrected data set is used to validate and calibrate the model and directly leads to the design results (e.g. reactor volumes or effluent quality). It is important to keep track of all the data adjustments because this information is used to determine the required safety factor used for the model-based design.

4.11 Requirements for the model accuracy

Within the framework of each design project, information is gathered according to the available possibilities within a given timeframe and budget. A range of information sources can be used such as interviews with plant operators and plant management, historical measured operational plant data, additional analytical measurements, original design documents and reports on plant operation. Most of the time, some of the data will be unavailable or will be available but inaccurate. These gaps in information will need to be filled in based on expert judgment and sound reasoning, supported by literature references. While doing so, it is possible that new errors are introduced. In general, plant information is likely to contain errors and therefore model design calculations will not always (fully) represent the reality. On the other hand, the model design method presented here in itself serves as a check on the plant information; the model relies on general rules of mass conservation (mass balances) and therefore, all the input and output data needs to closely correspond. Therefore, when the model cannot be validated by the plant measurements, this indicates model or measurement errors. When this occurs, it should be realized that the model results do not necessarily represent reality and the appropriate safety factors need to be applied when interpreting a (model) design. When evaluating a model design study, the possibility of errors should always be taken in account (the designer must use common sense), and the results of a (model or classical) design study also need to be seen in that context. Therefore it is good practice to present the results of a model design study in a way that it is clear to the reader where data adjustments and corrections were necessary to fit the model to the measurements.

To decide on how accurate in practice a model-based design should be depends on the design questions to be answered (the objectives of the study). This can be explained based on the following example where two totally different design questions are compared in relation to the required model accuracy:

- The first question could be: how accurate does a model need to be if one wants to use it to design a new (non-existent) WWTP if there is only influent information available about the general demographic growth of a community over 15 years? Here, probably a simple (and less accurate) model would be acceptable; the error in this study would be largely determined by the error in the estimation of the demographic forecast. To cope with this large uncertainty of the input data (the influent), large safety factors would also be required in the plant design.
- A very different question could be: how accurate does a model need to be to predict ammonium effluent peaks in stormy weather conditions at an existing plant? Here the model error probably needs to be less than 5%. This would require extensive plant measurements of a range of operational conditions and model calibration and validation and maybe even dynamic simulation.

For both the above design questions, model-based design is useful but, however, the required model accuracy will be totally different. To answer the first question, constructing a simple model based on readily available plant and operational information would probably be sufficient. However, to answer the second question, it is probable that detailed (e.g. time dependent) operational information and process measurements are needed with all the costs and time involved. From this example it becomes evident that before making a model, the question for what purpose the model will be used should be answered.

Chapter 5

Methodology for influent assessment

5.1 The wastewater chain

The construction of a new wastewater treatment plant or the upgrade of an existing plant is carried out based on a certain wastewater flow (quantity) and composition (quality), taking into account both present and future situations. When a new wastewater treatment plant is to be built, there are often no flow or concentration measurements available concerning the plant influent. In this case estimations have to be made based on the expected influent quantity and quality including a future forecast (often 10 to 15 years). When upgrading an existing treatment plant the influent flow and composition can often be measured; however, it will then also be necessary to make a future forecast of the influent flow and possibly also the composition, taking into account industrial activity. Usually such predictions of the influent flow and composition are carried out for the typical design parameters (e.g. Figure 5.1). However, this approach does not necessarily result in the most accurate estimation. As a result of the uncertainty involved, large safety factors are often introduced at the early design stage which consequently increases the costs of construction of the new or upgrade of the existing plant.

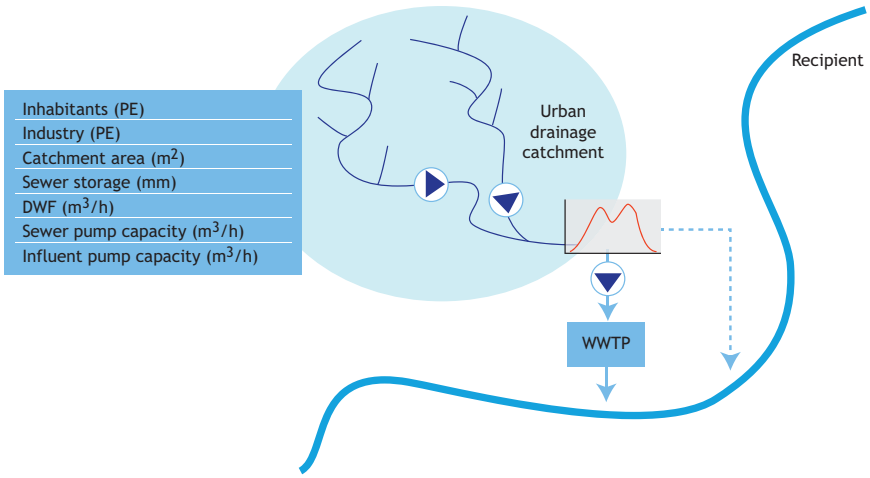

Figure 5.1 Schematic representation of the wastewater chain and key design parameters. Sewage flow is measured at the influent of a wastewater treatment plant with its typical daily pattern.

On the other hand, there is often more information on the origin of the sewage system available, which can help to make better estimates of the influent flow (the flow dynamics) and sewage quality (concentrations and loads). In this chapter, the factors determining the influent flow and concentrations are explained. A methodology is proposed to better establish the key influent design parameters. Rather than a mathematical model prediction of the sewerage quantity and quality, the methodology presented here is a framework supporting the estimations needed for determining the key influent design parameters. The sewer system is presented as a chain of events referred to as the 'wastewater chain' as it describes the origin of sewage (influent) from several sources in the sewage system. In the following sections the wastewater chain is explained step by step and the key influent parameters needed for the design of a wastewater treatment plant are presented as well.

For a design study the influent flow needs to be presented by an average dry weather flow (ADWF), peak dry weather flow (PDWF) and peak wet weather flow (PWWF or rain flow). The influent quality varies depending on the type of influent flow and is typically established at mixed (combined) weather conditions (Q_{365}, yearly average). Typically, this is done based on a pre-set measurement program over a year (disregarding the weather conditions) for the purpose of legislation compliance. Figure 5.1 schematically shows the overall sewage system including the pumping stations and WWTP. The key design parameters based on which the sewage (or influent) composition is often determined are shown in the box in the figure.

Figure 5.2 is the enlargement of the daily dynamic influent flow measurement schematically represented in Figure 5.1. It shows the peak wet weather flow (PWWF), average wet weather flow (AWWF), peak dry weather flow (PDWF), mixed weather flow (Q_{365}) and average dry weather flow (ADWF). In practice PWWF is often identical to the maximum hydraulic sewer capacity determined by the sewage Pump Overflow Capacity (POC). When the sewer has overcapacity (and thus never overflows), the measured PWWF can be lower. PWWF is expressed as a percentage of ADWF (typically 350%).

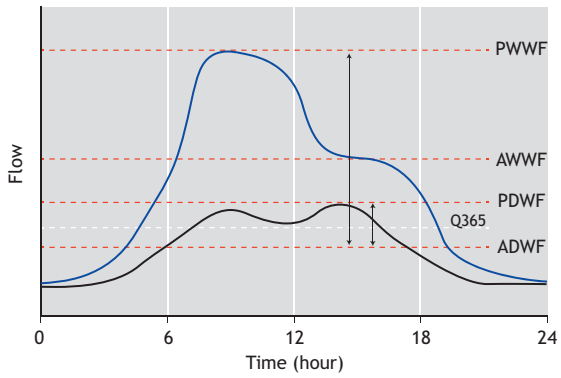

Figure 5.2 Definition of peak dry weather flow (PDWF), peak wet weather flow (PWWF), mixed yearly average flow (Q_{365}), and average wet weather flow (AWWF). All peak factors in this study are expressed as a % of ADWF.

Typical municipal sewage consists of:

- Domestic/household sewage;
- Industrial effluents;
- Rainwater;
- Infiltrate.

Each of the four main municipal sewage components has different qualitative and quantitative characteristics as elaborated on in the following sections.

5.1.1 Domestic sewage

The sewage flow quantity of domestic origin (from person equivalent or PE) has a strong relation with the drinking water consumption per person. Therefore, the dry weather sewage flow (DWF) from a purely domestic origin should in theory correspond with the drinking water consumption per person (capita). Typical drinking water consumption is 120-140 litres per capita per day and the water is available from the local drinking water company. Typically, drinking water is used over 12 to 16 hours per day with peak usage in the morning at around eight o'clock and in the afternoon at around six o'clock. The exact time when these peaks are measured at the location of the WWTP influent is determined by the sewage hydraulic residence time (HRT). For domestic sewage this results in a typical camel back-shaped flow profile during the day (Figure 5.2). However the calculated water use per person (ADWF divided by total PE) will typically be higher as the result of infiltration of foreign water into the sewer (Table 5.1). If the same is calculated for the yearly average flow, the mix of 365 dry and rainy days indicated as Q_{365}, this number will again turn out higher; typically 200 to 240 L/cap.d. More information on population equivalent and person load is given in Chapter 6.

5.1.2 Industrial discharge

Industrial wastewater production heavily depends on the type of industry. Industry working with batch production typically results in sewage peak loadings whereas continuously operated industrial processes will result in a constant increased 24-hour sewer discharge. Again, other types of industry can have peak production during certain periods of the year, for example, industries processing seasonally produced agricultural products. Also the tourist sector (camping sites and temporary residents) can cause these typical seasonal effects on the influent (ADWF) loading. Therefore, for a reliable plant assessment, it is necessary to evaluate the origin of the different industrial and domestic sewage sources over longer periods of time. Industrial activity is measured at the influent of a WWTP as increased ADWF and also accounts for the PE loading. Industrial discharge will also result in an increase of the Q_{365} flow; however, the ratio of Q_{365} over ADWF (typically 1.2) as a result of industrial discharge becomes smaller, because the amount of rain catchment, which is determined by the paved surface and related to inhabitants, will typically not increase (or relatively little) with increasing industrial activity. Therefore, the Q_{365}/ADWF ratio is an indication of the contribution of industry to the sewage flow (and its composition). For example, the measured Q_{365}/ADWF ratio of 1.05 indicates some influence by industry on the flow and thereby probably also on the composition. If the designed Q_{365}/ADWF ratio is, for example 1.22, then the influence of industry should be ignored.

5.1.3 Wet weather flow

Rainwater (wet weather flow or WWF), typically expressed in millimetres of rainfall, is collected on paved catchment surface (m^2) and from there led into the sewer system. These two numbers multiplied together (mm rainfall × paved surface = flow volume) will result in the flow increase

on top of ADWF caused by a rain storm event. Taken on average over a year this results in the mixed flow Q_{365}. The maximum sewage capacity is expressed in millimetres of storage. Above the maximum sewage storage capacity the sewage system will overflow (bypass), usually into nearby surface water. In the sewer system pumping stations are used to transport the sewage to the WWTP. The maximum sewer pump capacity determines the amount of water which can be processed/transported by the sewage system to the WWTP. This maximum pump capacity indicates the pump overflow capacity (POC). Finally, the collected sewage reaches the wastewater treatment plant (and from here is regarded as influent). The maximum hydraulic capacity of the wastewater plant determines how much sewage can be treated. What cannot be treated will be bypassed to the recipient (river). The maximum influent pump capacity is often larger than the hydraulic capacity of the WWTP as the result of installed safety factor and need for maintenance.

5.1.4 Dry weather flow and mixed weather flow

In general 90% of influent pollution load (kgCOD, kgTKN, kgBOD, etc.) has its origin in dry weather flow (ADWF) combined with industrial discharge. This load is indicated in PE, being the average pollution load produced per person per day. PE used to be expressed in 60 (or 54) g of BOD_5 per person per day. However, nitrogen removal is also of importance for modern design. Therefore, for a nutrient removing plant, PE is calculated as 136 g total oxygen demand (TOD) being the combined oxygen requirement of COD (1 gTOD/gCOD) and TKN (4.57 gTOD/gTKN) according to the calculation below.

$$\text{PE at 60 gBOD per person per day} = \frac{\text{kg BOD}_5 \times 1000 \text{ kg/g}}{60 \text{ g BOD}_5} \quad (5.1)$$

$$\text{PE at 136 gTOD per person per day} = \frac{(\text{kg COD} + 4{,}57 \times \text{kg TKN}) * 1000 \text{ g/kg}}{136 \text{ g TOD}} \quad (5.2)$$

The remaining 10% of the measured pollution load is caused by rainwater collecting and transporting dirt from paved catchment areas during storm events. This is expressed in a loading difference between ADWF and Q_{365}, which in theory, if rain was totally clean, would result in a zero loading difference between ADWF and Q_{365}. Thereby, the concentration of ADWF is higher than that of Q_{365} (dilution). However, if a rain event is measured, often a high initial peak load (the first flush) is followed by a slightly increased (COD) load as the result of rainwater. The first flush rapidly pushes out concentrated ADWF sewage in combination with settled solids originating from ADFW. Therefore the first flush can theoretically be attributed to the ADWF loading. However, the increased load following the first flush (ADWF plus mixed with the actual rainwater) is the result of rain-born pollution originating from pavement dirt containing slow and/or non-biodegradable organics and sand (inorganic material or ISS). Also pavement dirt is a source of micro pollutants of oil and heavy metals. The extra sand is largely removed in the grid chambers (when properly designed). The additional load by rain (typically 10%) is mainly COD, and contains a relative high fraction of unbiodegradable organics. Therefore the difference between ADWF and Q_{365} load is often smaller (as low as zero) when only looking at BOD and TKN loads (kg).

5.1.5 Infiltration of foreign water

Besides the sewer inflow, what happens also inside the sewer system during transport determines the influent composition and flow. This is schematically represented in Figure 5.3.

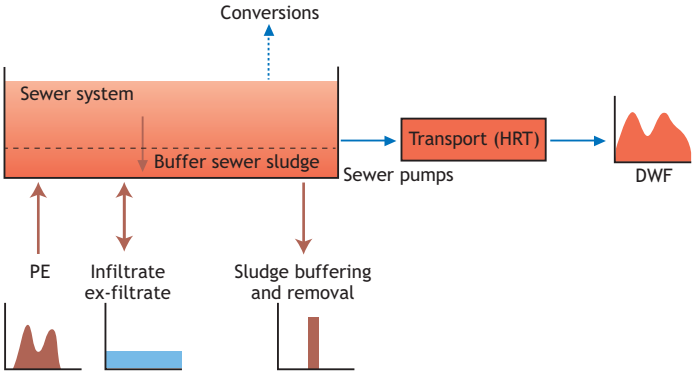

Figure 5.3 Schematic model of a sewer system and DWF composition

Besides PE-based DWF from domestic and industrial water use, sewage can also be diluted by a considerable amount of ground and/or surface water. This foreign sewer water infiltrates into the sewer system via leaks, especially when the system is not well maintained and during periods when groundwater levels are high (aquifer conditions). Surface water can sometimes freely flow back into the sewer system when sewer overflows are not properly designed, overflow weirs are mechanically malfunctioning or when flooding occurs of surface waters above the regular overflow design height. Typically, infiltration results in dilution of DWF. Where typically the COD concentration of domestic origin is around 500 mg/L (TKN 40 and TP 6 mg/L), as a result of infiltration, COD concentrations can drop to 200-300 mg/L (TKN 20-30 mg/L). Therefore, it is important to take into consideration that sewage can be strongly diluted (by up to 50%) with infiltrate. From a design point of view, infiltration will not influence the plant biological loading (BOD and TKN) very much because typically it will be relatively clean water of surface or groundwater origin. However, infiltration will increase the hydraulic plant loading (ADWF, PDWF and Q_{365} and PWWF) and thereby affects the hydraulic design loading of sand catchments, and primary and secondary settling. Occurrence of infiltration can be indicated by diluted influent concentrations and also by the calculated difference between the use of drinking water (typically 140 L/PE.d) and the actual measured influent ADWF flow (typically 200 L/PE.d here is calculated for ADWF). Infiltration can be a constant source (e.g. leakage) but can also vary over time as the result of seasonal groundwater and/or surface water heights, causing sewer back-flow. Infiltration and dilution therefore have to be taken into consideration especially when designing hydraulic limited process units such as settling tanks and influent pumps.

5.2 Conversions in the sewer system

Infiltrate can contain high amounts of nitrate; up to 20 mgN-NO_3/L is possible depending on the ground or surface water source. If nitrate is present in the sewer then readily biodegradable organic substances, typically acetate, are converted by denitrification. Bacteria capable of denitrification grow in biofilms attached to the sewer. Also aerobic processes can occur when sewage is aerated during transport e.g. due to turbulent open flow conditions. This conversion will change the sewage composition. Some activated sludge systems heavily rely on the presence of readily biodegradable organic substrate for good operation. Volatile Fatty Acids (VFA), primarily acetate and propionate, are used by the Bio-P process for growth. Therefore,

possible conversions in the sewer and shortage of influent VFAs need to be taken in account when designing a wastewater treatment plant. The amount of bacteria in raw sewage is very small, a factor of 1,000 times lower, compared to the MLSS concentration in activated sludge. Therefore this active COD concentration can be ignored when modeling a WWTP. When sewage is transported over long distances (a long sewage residence time) without being exposed to oxygen or nitrate, this results in hydrolysis and fermentation of organics (VSS) in the sewage system. This changes the sewage composition and it becomes more soluble (acidic) and readily biodegradable. Typically this can be indicated by influent pH (< 7) measurements and a typical rotting (H_2S) smell of raw sewage. When the hydraulic residence time in the sewer is relative short, less or no hydrolysis and fermentation will occur and as a result relatively more particulate and slowly biodegradable material is expected in the influent. This type of raw sewage will in general smell less and will also have a higher pH value (of around 7). Therefore, also the type and duration of the transport of sewage to the wastewater treatment plant will influence the wastewater treatment design.

5.3 Sludge buffering and removal

Depending on the flow velocity in the sewage system, the heavier particulate material in domestic sewage will settle and accumulate in the sewer system. Especially after a long dry period the first storm event will result in a significant amount of accumulated sewage sludge to be transported to the wastewater treatment plant. This effect is known as the first flush; a high amount of particulate organic (VSS) and inorganic material (ISS) accompanies wet weather flow (WWF). Especially for wastewater treatment plants operating with a short sludge residence time (SRT), the first flush can have a major effect on the plant operation. It causes a sudden increase in organic material in the sludge. Because at WWF MLSS is initially stored in the settlers, it will not be directly measured in the activated sludge tanks. However, sludge production will increase and therefore the VSS will contain more waste sludge, which negatively affects thickening and sludge dewatering. With increasing SRT, this effect is reduced, because the accumulated organics from the first flush have more time to be converted in the activated sludge process (e.g. 20 days instead of 3 days SRT for a BOD-removing plant).

5.4 Transport system

The type of transport system to the wastewater treatment plant (gravitational flow or pumped) and the sewage residence time (storage capacity) will determine the DWF 24-hour influent flow dynamics and also the peak dry weather flow (PDWF). Figure 5.4 shows two examples of typical influent flow dynamics for a short sewer residence time (left) and a longer Hydraulic Residence Time (long HRT, right). Even when the 24-hour average flow (ADWF) is identical, the peak flow (PDWF) can be significantly different as the result of sewer residence time. An increased HRT (right) cause lowers PDWF due to sewer storage and buffering. Also the typical peaks measured in the morning and afternoon will be measured later on in the day due to longer transport time (in this example the morning peak arrives at the WWTP not at 8 but 12 o'clock). As a result of the sewer HRT, PDWF is typically between 125 and 200% of ADWF. The hydraulic sewer residence time is also strongly determined by a sloped surface; for sloped sewer systems peak factors can be expected up to 200% ADWF. For flat systems, typical for Dutch and central Croatian conditions, lower peaks are expected (e.g. 150% ADWF). In this guidelines it was assumed that DWF is produced over 16 hours per day resulting in 150% PDWF (24h/16h×100% = 150%). When designing a WWTP, the peak flows (and corresponding peak loads) determine the maximum required aeration capacity.

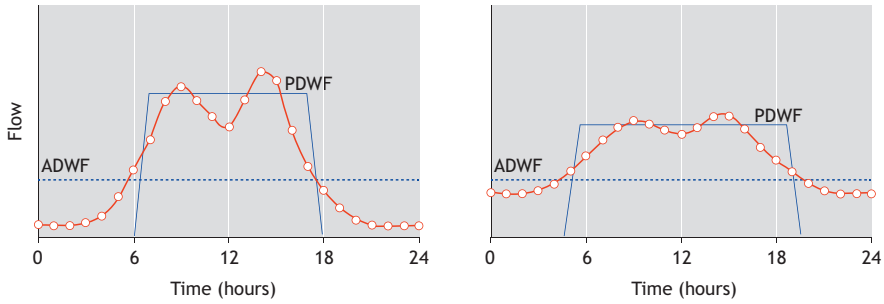

Figure 5.4 Schematic representation of a 24-hour DWF. ADWF is the same for both sets of conditions. PDWF changes as the result of sewer residence time. Increased HRT (right) causes lower PDWF due to sewer buffering. Also the typical increase in morning and afternoon will be measured later on in the day (in this example not at 8 but at 12 o'clock).

5.5 Methodology for ADWF assessment

The wastewater chain model as presented in the previous paragraphs can be used to estimate a realistic influent load and concentrations if no measurements are available or if a future condition needs to be estimated. First the DWF is reconstructed. The per-capita loading is estimated from demographic data (a count of residents from tax information). The drinking water use gives an indication of the DWF flow (L/cap.d). The prime population pollution load is estimated from 136 gTOD/cap.day and 60 gBOD/cap.day. The daily variation in the influent loading can be reconstructed using the wastewater chain assumptions; ADWF carries an average pollution load of population and industry. By estimating a sewage residence time (e.g. 16 hours of water use per day), PDWF can be estimated leading to the typical daily flow and loading pattern (according to Figure 5.4.)

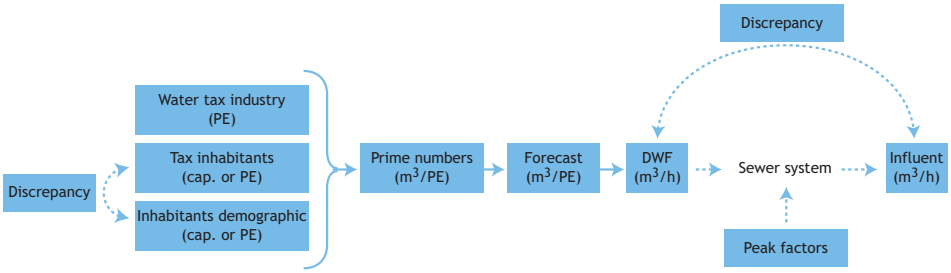

Figure 5.5 Methodology for (future) DWF assessment

A systematic estimation of the pollution loading can be made according to the scheme in Figure 5.5. Administrative information on the amount of pollutants can be obtained from the number of taxed inhabitants and industry PE. This can be a drinking water tax, sewer tax or environmental tax and production numbers on drinking water consumption. The difference between these numbers and actual ADWF is shown as a discrepancy and is typically caused by

differences between administrative and actual population numbers and industrial (or tourism) activity. For a forecast the best guess for future discrepancy is calculating the current discrepancy. Discrepancy is a main indicator for the magnitude of the required safety factor when calculating a future influent forecast for WWTP design. A plant is typically designed for 10 to 15 years, which is also the maximum realistic planning horizon for demographic industrial developments. When designing for the future, realistic assumptions are needed which take all the factors into account. A forecast of expected population growth is typically in the range of 1-2% per year for a slow-growing community. Future industrial activity is often overestimated (by industry) due to progressive planning. However, economic factors usually heavily affect industrial activity and smaller growth factors may often be more realistic (e.g. 3-4% growth per year). Nevertheless, as a result of progressively larger forecast estimates, WWTP are often designed bigger than necessary, which results in not only large financial budgets but also in operational problems. On the other hand, as a result of (administrative) discrepancies, progressively larger sewage forecasting calculations can also be partly cancelled out because more inhabitants will be connected in the future.

5.6 Calculating design loads and concentrations

In design practice, loading rates, dry weather and mixed weather (ADWF and Q_{365}) are used for design purposes, often depending on the choice of the designer. This is allowable but it is important to distinguish between the origin of the measured concentrations and flows when using these concentrations and the flows for design (load) calculations. Typically, Q_{365} based concentration measurements are planned on a pre-set yearly agenda disregarding the weather conditions (often determined by legislative requirements). As a result, these measurements do not always represent actual mixed (dry and wet weather) conditions. In general, wet weather conditions are difficult to define; establishing the starting point of a rain event and when it ends can be problematic.

Therefore, a better option is to design a WWTP using only flow-weighted average ADWF influent loads; ADWF flow and concentration can be measured separately leading to straightforward calculation of an ADWF load. Dividing this average influent load by the average ADWF flow will give the flow-weighted average ADWF concentrations. This is shown to be the most precise estimation of the actual influent loading. If the influent data provided does not clearly distinguish between dry days and wet days, influent data filtering can be used to distinguish the dry days, as was also done in the example presented here.

When calculating design loads, it should be borne in mind that concentrations measured at average Q_{365} flow conditions (a typical method for influent measurements) are usually slightly lower (slightly more diluted) than concentrations measured at ADWF. Therefore, when calculating the ADWF design load by using average Q_{365} concentrations multiplied by ADWF, this results in an underestimation of the design loads. The magnitude of this underestimation depends on the ratio Q_{365}/ADWF, typically in the range of 120%. This error is significant and should be avoided by only working with preselected (statistically filtered) ADWF influent flows and concentration data.

5.7 Influent flow assessment; an example

As an example of the influent flow analysis as previously explained in this chapter, the measured influent flow data for WWTP Varaždin has been analyzed. The yearly average flow data of the wastewater treatment plant was available (the daily average flows are presented in Figure 5.6). Dry and wet weather days are distinguished based on statistical filtering. The filter can be

designed based on different criteria: either detecting the combination of high flow and diluted concentration measurements in the data based on a pre-set threshold or, as was done in this example, detecting rapid flow increase based on the first order derivative of the flow measurements indicating rain events. This option was used in this example because the amount of concentration data was limited.

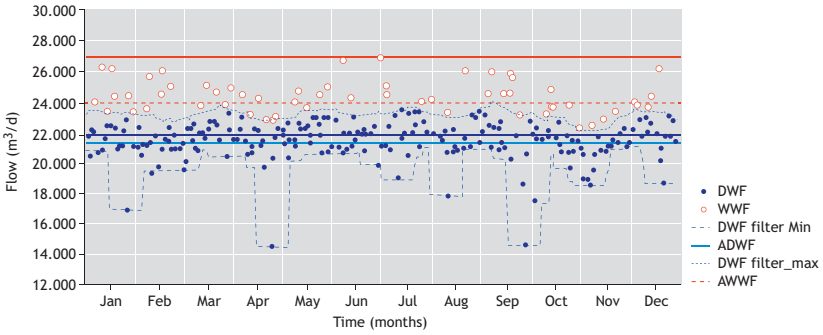

Figure 5.6 Example of influent flow measurements and statistical filtering of DWF and WWF (at WWTP Varaždin in 2010)

In Figure 5.6, all the measurements above the maximum DWF filter level (indicated in the figure by a grey line) are indicated as wet days (WWF, white dots). A general check on the filter calculations is the number of wet weather days (white dots, in this case 90) which should (approximately) correspond with the actual days of rainfall. In this example the filter resulted in 90 wet days over total 485 measurement days in the year 2010 which seems an acceptable number of rainy days.

5.7.1 Average dry weather flow (ADWF)

From the example in Figure 5.6, the yearly average dry weather flow (ADWF) is 21,300 m^3/d which is calculated by averaging only the filtered dry weather days (the black dots in Figure 5.6). As was explained earlier in this chapter, ADWF is related to the population equivalents (PE by inhabitants and industry) and thereby closely related to the concentration loading of the activated sludge system. However, infiltration of foreign water in the sewer system will be also typically included in ADWF. In this case, it will result in diluted influent concentrations.

5.7.2. Peak dry weather flow (PDWF)

Peak dry weather flow (PDWF) represents the (average) maximum flow measured over the day at ADWF (24-hour dynamics). At PDWF typically the maximum aeration capacity in the activated sludge tanks is required. Peak dry weather flow is established by measuring the maximum influent peak over a 24-hour recurring period during several days of dry weather. The height of PDWF is heavily dependent on the sewer residence time (HRT). A typical sewage residence time for communal influent is 16 hours (of 24) based on which peak dry weather flow is calculated (Table 5.1: 25,600 m^3/d). If the influent flow is produced by industry, the daily peak load is very dependent on the industrial plant operation and wastewater discharge. Buffer and equalization tanks are often applied.

5.7.3 Yearly average flow (Q_{365})

The yearly average mixed weather flow, also indicated as Q_{365}, is the result of all dry and wet weather days averaged over the 365 days of influent flow measurements. The measured mixed weather flow in the example is 21,800 m^3/d (Figure 5.6 and Table 5.1). The plant used as an example has a large fraction of industrial wastewater (72% of the total sewage loading) resulting in Q_{365} being low relative to ADWF (Table 5.1: Q_{365} is 102% of ADWF). Depending on the designer's choice, dry weather flow or yearly average flow can be used to calculate the yearly average influent load (for TSS, BOD_5, COD, TN and TP). As explained in the previous sections, mixed weather data (Q_{365}) by itself can be used for design calculations if there are enough representative measurements also including wet days. However, storm weather measurements are often not measured in practice, or not enough to produce representative WWF concentration data. Therefore it is advised to use the influent filtering method to determine ADWF instead.

5.7.4 Average wet weather flow (AWWF)

By statistical filtering, the wet weather flow (WWF) is determined. Average wet weather flow (AWWF) is 23,900 m^3/d (Figure 5.6 and Table 5.1). However, AWWF is not used for design purposes; the number of WWF days resulting from the filtering method are used to determine if the filtering method yields (approximately) the same amount of actual rainy days. The filtering method resulted in 90 wet days in 485 measurements in 2010, which seems a reasonable estimation.

5.7.5 Maximum hydraulic loading (PWWF)

The maximum hydraulic loading is the maximum flow that the plant is designed for; above the maximum sewage capacity, sewage is bypassed to the nearby recipient. The measured maximum flow is 27,000 m^3/d (Figure 5.6 and Table 5.1). The maximum sewage capacity is 58,300 m^3/d. The maximum hydraulic loading is a key design parameter for settler design (Chapter 14). The maximum hydraulic loading can be established from the measured PWWF or it can be related to the maximum hydraulic sewage (overflow) capacity.

Table 5.1 Example of flow retrofit at WWTP Varaždin (data from 2010, corresponding to Figure 5.6)

Flow	Code	% ADWF	m^3/s	m^3/h	m^3/d
Minimum measured flow	QMIN	63	0.16	562	13,478
Average DWF[1)]	ADWF	100	0.25	888	21,321
Average mixed flow	Q365	102	0.25	909	21,808
Peak DWF[3)]	PDWF	120	0.30	1,066	25,585
Average WWF[2)]	AWWF	112	0.28	998	23,947
Maximum sewerage (overflow) capacity	QMAX	274	0.68	2,430	58,320
Installed capacity (including spare pumps)	QPUMP	547	1.35	4,860	116,640
Maximum measured flow	PWWF	127	0.31	1,127	27,043

1) Dry weather days: 485
2) Wet weather days: 90
3) Peak DWF is calculated assuming inflow of 20 L/h
% ADWF is a peak factor of ADWF

5.8 Statistical flow analysis

An approach leading to a more compact hydraulic (settler) design is to calculate the influent flow frequency distribution from the time range of influent flow data (Figure 5.7 and Figure 5.8).

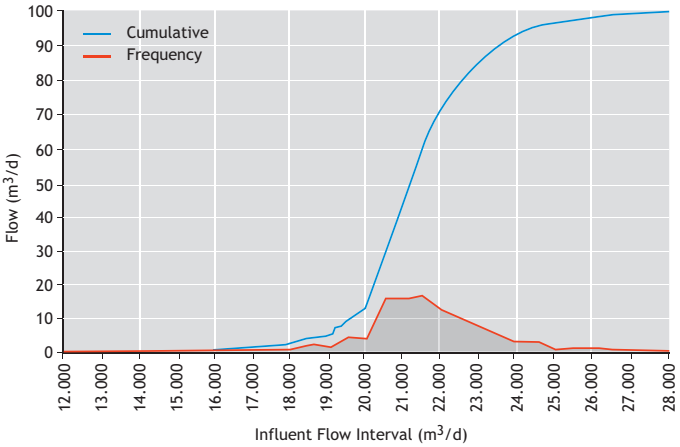

Figure 5.7 Example of an influent flow statistical assessment (at WWTP Varaždin in 2010): relative cumulative flow distribution (WWTP Varaždin has a big industrial contribution - 71% of total influent flow).

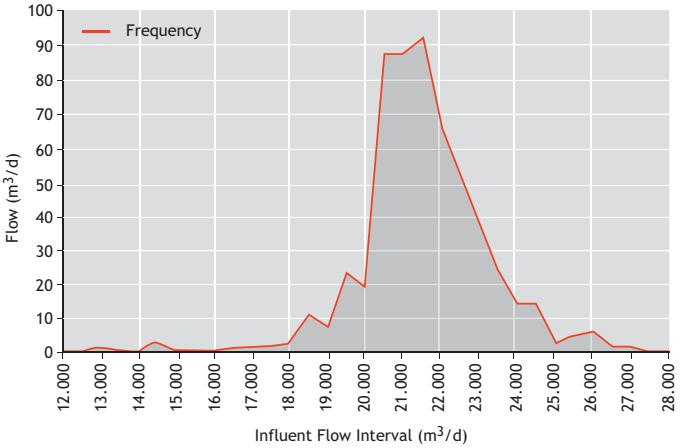

Figure 5.8 Example of an influent flow statistical assessment (at WWTP Varaždin in 2010): relative flow distribution. The influent flow bandwidth is narrow and only very slightly influenced by rain storm events.

Based on the flow distribution it is possible to select and design using the maximum allowed percentile value. Typically the 98 percentile is selected thereby excluding the top 2 percentile flow extremes from the data. Generally, this results in a significant smaller design at an acceptable additional risk. To calculate the flow distribution all the flow measurements are collected. This can be time independent and therefore it is also possible to obtain several years of flow data. The data is arranged in (increasing) magnitude and divided into intervals; in this case 500 m^3/d. In this example, designing (e.g. the secondary settler) on the 98 percentile (i.e. 98% of Figure 5.7), PWWF can be reduced to 25,500 m^3/d instead of the measured maximum

27,000 m^3/d. However, this results in a smaller settler design (in this example slightly smaller), at the cost of occasional sewage overflow (raw sewage bypassing the WWTP). From the frequency distribution (Figure 5.8) it can be calculated that in this example, sewage overflow would occur during the 11 heaviest rain events per year. During these events only a flow larger than 25,500 m^3/d would bypass the WWTP. What would bypass is diluted (has a low concentration) and therefore the bypassing loads would in practice be limited. To save costs on settler capacity but also to avoid sewage overflow, a storm water buffer can be used. The main advantage of using a buffer tank is that it is cheaper than to build the additional settler volume.

5.9 Design temperature

The influent temperature is a key parameter for wastewater treatment design. Typically, temperature is measured online in the inflow of a WWTP and automatically recorded. Figure 5.9 shows the sample data measured at WWTP Čakovec in 2009. Generally, the influent temperature follows the air temperature and consequently is determined by the seasons of the year. As a result the influent temperature is very much regionally determined. In the winter, melting water from snow events can cause sudden drops in influent temperature (also observed in Figure 5.9). Therefore such events should also be noted in the plant inventory phase. In the case of industrial discharge, the influent can be warmer depending on the temperature and amount of the industrial discharge. The water temperature is directly related to the flow volume (in the same way that a load is related to the flow volume) and therefore the temperature of two mixed flows (e.g. industrial and domestic) can be calculated accordingly.

For wastewater treatment design, three design temperatures need to be identified from the measurements;

- The yearly average temperature;
- The minimum design temperature;
- The maximum design temperature.

Typically, the yearly average influent temperature is around 15°C, the maximum influent temperature around 21°C and the minimum influent temperature around 8°C. Figure 5.9 shows one year of sample data for WWTP Čakovec. In the graph, the average maximum and minimum influent water temperatures are calculated. Measured temperature extremes were indicated by applying the 98-percentile range. This is done in the same way as previously demonstrated for the influent flows (see this chapter). This will be further explained in the following sections.

5.9.1 Average design temperature

According to the example in Figure 5.9, the yearly average influent temperature is 14.4^{o}C. This is calculated based on the evenly distributed mean of one year of measurements (346 measurement days). The yearly average temperature is used in the design to calculate the yearly average effluent performance of the wastewater treatment plant. When dynamic modeling is used (e.g. BioWin), a 365-day dynamic temperature range can be simulated resulting in an (yearly average) effluent concentration. Hereby, seasonal temperature fluctuations are all taken in account. In general, this is a more accurate way of determining the plant performance and preferable to a static approach.

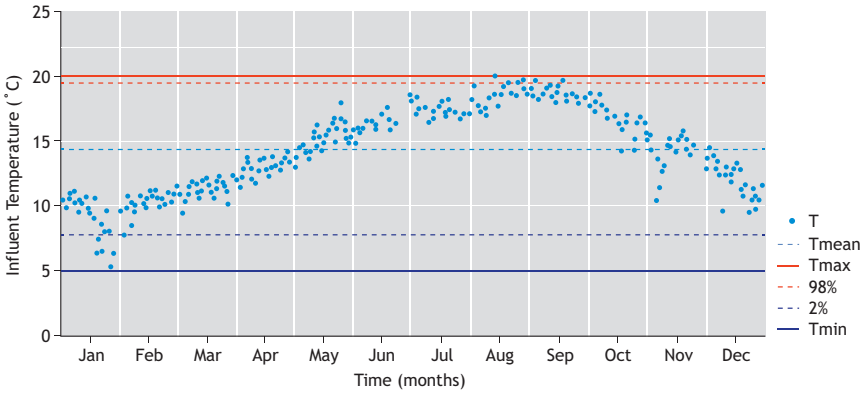

Figure 5.9 Example of an influent water temperature evaluation (sample measurements from WWTP Čakovec in 2009)

5.9.2 Minimum design temperature

The absolute minimum temperature is measured at 5.4°C. The figure shows this is an extreme winter condition, most probably with melting water from a snow event. When the 98-percentile range for the minimum design temperature is adopted this results in a more general minimum design temperature of 7.8°C. The minimum design temperature determines the minimum size for the aerobic reactor volume allowing for full nitrification also during the coldest winter conditions and therefore is a key design parameter. This is also because it determines to a large extent the cost (i.e. volume) of a wastewater treatment design. Nitrification is the limiting process because the nitrifying bacteria are the slowest growing organisms; especially in cold winter conditions because their growth is a function of the water temperature. This means in winter the bacteria need a larger aerobic volume which is expressed by the anaerobic sludge retention time.

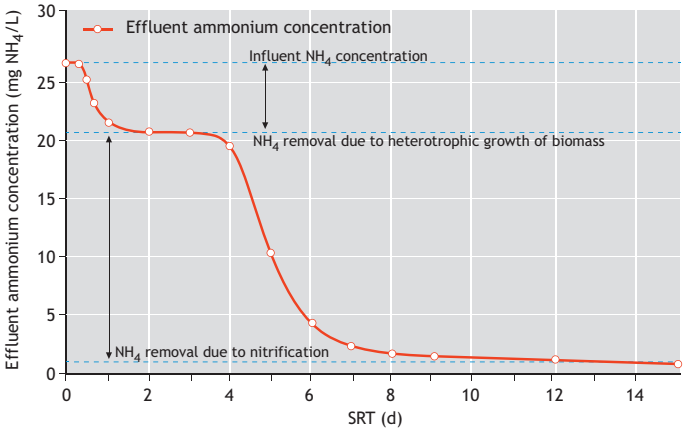

Figure 5.10 Example of the relation between water temperature, aerobic sludge retention time and nitrification (effluent ammonium concentration).

If the minimal aerobic SRT is not maintained, more bacteria will be removed via the waste sludge than will grow in the activated sludge plant, and as a result nitrification will be washed out (i.e. the nitrification process will cease). This deterioration will not happen instantaneously however, it will only occur after several days (approximately one time the aerobic SRT, being usually 3-5 days) of operation at low temperature A general rule of design is that at 8°C approximately eight days of aerobic SRT is required to maintain full nitrification capacity (Figure 5.10, calculated from the BioWin model, being a general example). However, there are several other factors that can influence this relation. Therefore it is good design practice to use the activated sludge model to calculate the minimum aerobic sludge retention time at the lowest operating temperature.

5.9.3 Maximum design temperature

The maximum measured influent temperature determines the maximum aeration (installed blower) capacity. With increasing temperatures the solubility of oxygen decreases. Therefore more blower capacity is required to provide oxygen to the activated sludge system at high temperatures. The maximum measured temperature is 20.3°C. After ruling out extremes based on the 98 percentile the maximum design temperature is 19.6°C.

5.9.4 Considerations for selecting design temperatures

Measured temperature extremes can be ruled out by applying the 98-percentile and 2-percentile range. Ideally, the temperature range is measured over multiple years. Therefore a frequency distribution can be made according to Figure 5.11.

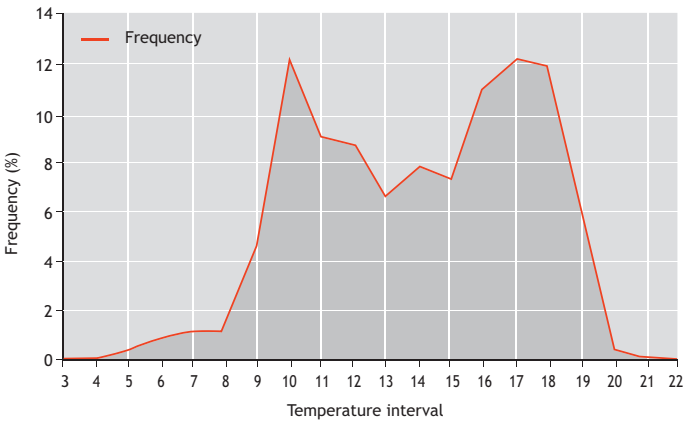

Figure 5.11 Example of frequency distribution in 2009 for the measured temperatures of WWTP Čakovec (at intervals of 1°C).

This is done in the same way as previously explained for the influent flow (see this chapter). Over the total measuring range, the temperature occurrence is counted using intervals of 1.0°C. E.g., 18°C occurred at 12% of all measurements (346 days of measurements). However, 20°C was measured only once (representing 0.3% of all measurements). Therefore, 20°C could be ruled out as the maximum design temperature. This can also be represented in the cumulative distribution in Figure 5.12. The frequency distribution is more or less homogeneously (lineally) distributed between 9°C and 19°C. Outside this range temperatures occur less frequently. By ruling out the top 2% and bottom 2% of the temperature measurements (e.g. the 98-percentile

value) a representative temperature distribution remains without any extremes. This temperature range is more representative for design purposes.

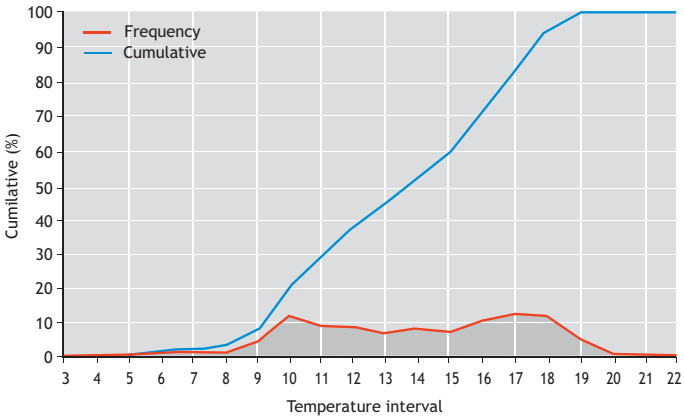

Figure 5.12 Example of cumulative temperature measurement frequency distribution over 2009 (at WWTP Čakovec) combined with the measured frequency per temperature interval (1°C).

5.12 Selection of safety factors

A safety factor is often applied to the design temperature. To do this, the average and maximum temperature are increased by a safety factor and the minimum design temperature is decreased by a safety factor. This results in a larger (aerobic) tank volume to compensate for the yearly average reduced effluent quality and it also results in an increased blower design capacity. Rather than using multiple safety factors, it is preferable to apply one general safety factor to the final design; simultaneously applied safety factors are likely to result in an uncontrolled enhancing effect where safety upon safety is applied. This is avoided when the wastewater treatment plant is designed with the smallest possible safety factor and in the end, one general safety factor is applied. This general safety factor in the design should be chosen based on the largest required safety factor, thereby incorporating also smaller operational risks. This approach is justified when it is realized that the possibility of all extremes occurring at the same time is very small and for some processes completely unlikely or even impossible e.g. an occurrence of the highest and lowest temperature at the same time. Another more likely example would be that the maximum PDWF, for which a separate safety is often applied of 110-120%, would need to occur on the hottest day of the year (for which a safety factor is often also applied of 105-110%) and only under these extremely rare conditions would result in blower under-capacity (being designed with the multiplied safety factor of both PDWF and the maximum temperature). This example may explain why applying a general safety factor results not only in better insight into the overall risk (which does not have to be fully covered under all circumstances) but also results in a more compact plant, thereby reducing the (building) cost. In the example presented here, if a safety factor is applied for the temperature, a realistic safety factor would be a factor on top of the 98 percentile which also includes the absolute measured maximum value. In this example, the 98 percentile for the maximum would be 19.8°C, and then a safe design temperature would be higher than 20.3°C. A possible safety factor then could be 21/19.8×100%=106% safety for the maximum temperature. The maximum temperature determines the maximum blower capacity as a result of the lower solubility of oxygen at high

temperatures. Typically, the blower capacity would have a much larger safety factor than 106% to cope with PDWF (e.g. 150%). If it is assumed that the two peak events (PDWF and a temperature extreme) are unlikely to occur simultaneously, then the PDWF safety factor (150%) for the blower capacity will also typically cover a maximum temperature event.

Chapter 6

Wastewater characterization and fractionation

Based on the chapter 'Wastewater Characterization' by Mogens Henze and Yves Comeau in 'Biological Wastewater Treatment: Principles, Modeling and Design' edited by M. Henze, M.C.M. van Loosdrecht, G.A. Ekama and D. Brdjanovic. ISBN: 9781843391883. Published by IWA Publishing, London, UK. (2008)

6.1 The origin of wastewater

The production of waste from human activities is unavoidable. A significant part of this waste will end up as wastewater. The quantity and quality of wastewater is determined by many factors. Not all humans or industries produce the same amount of waste. The amount and type of waste produced in households is influenced by the behaviour, lifestyle and standard of living of the inhabitants as well as the technical and juridical framework by which people are surrounded. In households most waste will end up as solid and liquid waste, and there are significant possibilities for changing the amounts and composition of the two waste streams generated. For industry similar considerations apply. The design of the sewer system affects the wastewater composition significantly. In most developing countries separate sewer systems are used and in these the storm water is transported in trenches, canals or pipes. Old urban areas might have combined sewer systems where different types of wastewater are mixed (Table 6.1). In combined systems a part (either small or large) of the total wastewater is discharged into local water bodies, often without any treatment.

Table 6.1 Wastewater types

Wastewater from society	Wastewater generated internally in treatment plants
Domestic wastewater	Thickener supernatant
Wastewater from institutions	Digester supernatant
Industrial wastewater	Reject water from sludge dewatering
Infiltration into sewers	Drainage water from sludge drying beds
Storm water	Filter wash water
Leachate	Equipment-cleaning water
Septic tank wastewater	

6.2 Wastewater constituents

The constituents in wastewater can be divided into main categories according to Table 6.2. The proportion of constituents can vary strongly.

Table 6.2 Constituents present in domestic wastewater (based on Henze *et al.*, 2001)

Wastewater constituents		
Microorganisms	Pathogenic bacteria, viruses and worm eggs	Risk when bathing and eating shellfish
Biodegradable organic materials	Oxygen depletion in rivers, lakes and fjords	Fish death, odours
Other organic materials	Detergents, pesticides, fat, oil and grease, colouring, solvents, phenols, cyanide	Toxic effect, aesthetic issues, bio accumulation in the food chain
Nutrients	Nitrogen, phosphorus, ammonium	Eutrophication, oxygen depletion, toxic effect
Metals	Hg, Pb, Cd, Cr, Cu, Ni	Toxic effect, bioaccumulation
Other inorganic materials	Acids, for example 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

6.3 BOD and COD

Organic matter is the major pollutant in wastewater. Traditionally organic matter has been measured as BOD and COD. The COD analysis is 'quick and dirty' (if mercury is used). BOD is slow and cumbersome due to the need for dilution series.

The COD analysis measures the majority of the organic matter present in the sample through chemical oxidation by dichromate. COD measurements are needed for mass balances in wastewater treatment. The COD content can be subdivided into fractions which are useful for consideration in relation to the design of treatment processes. Suspended and soluble COD measurement is very useful. Beware of false COD measurements with permanganate, since this method only measures part of the organic matter, and should only be used in relation to planning of the BOD analysis.

The theoretical COD of a given substance can be calculated from an oxidation equation. For example, the theoretical COD of ethanol is calculated based on the following equation:

$$C_2H_6O + 3O_2 \rightarrow 2CO_2 + 3H_2O \qquad (6.1)$$

or, 46 g of ethanol requires 96 g of oxygen for full oxidation to carbon dioxide and water. The theoretical COD of ethanol is thus 96/46 = 2.09.

The BOD analysis measures the oxygen used for oxidation of part of the organic matter. BOD analysis has its origin in effluent control, and this is what it is most useful for. The standard BOD analysis takes 5 days (BOD_5), but alternatives are sometime used: BOD_1, if speed is needed, and BOD_7 if convenience is the main option, as in Sweden and Norway. If measurement of (almost) all biodegradable material is required, then BOD_{25} is used. It is possible to estimate the BOD values from the single measured value (Table 6.3).

Table 6.3 The relationship between BOD and COD values in urban wastewater

BOD_1	BOD_5	BOD_7	BOD_{25}	COD
40	100	115	150	210
200	500	575	750	1,100

In this chapter, the term BOD refers to the standard carbonaceous BOD_5 analysis.

Figure 6.1 shows the dependency of time and temperature for the BOD analysis. It is important that the BOD test is carried out in standard conditions.

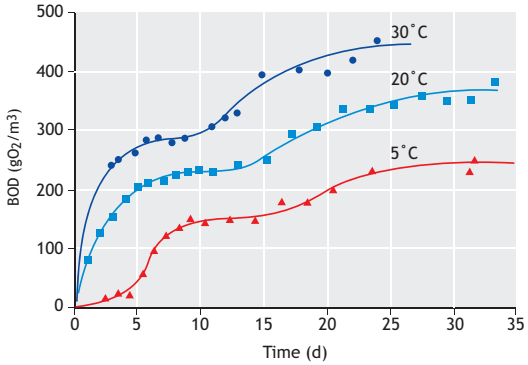

Figure 6.1 The BOD analysis result depends on both the test length and temperature. The standard is 20°C and 5 days.

6.4 Population equivalent and personal load

The wastewater from inhabitants is often expressed in the unit Population Equivalent (PE). PE can be expressed in water volume or BOD. The two definitions used worldwide are:

1 PE = 0.2 m^3/d
1 PE = 60 g BOD/d

These two definitions are based on fixed non-changeable values. The actual contribution from a person living in a sewer catchment, the so-called Person Load (PL), can vary considerably (Table 6.4). The reasons for the variation can be a working place outside the catchment, socio-economic factors, lifestyle, type of household installation etc.

Table 6.4 Variations in Person Load (Henze *et al.*, 2001)

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^3/cap.d	0.05-0.40

Person Equivalent and Person Load are often confused or misunderstood, so one should be careful when using them and be sure to define clearly what they are based upon. PE and PL are both based on average contributions, and used to give an impression of the loading of wastewater treatment processes. They should not be calculated from data based on short time intervals (hours or days). The Person Load varies from country to country, as demonstrated by the yearly values given in Table 6.5.

Table 6.5 Person Load in various countries in kg/cap.yr (based on Henze *et al.*, 2002)

Parameter	Brazil	Egypt	India	Turkey	US	Denmark	Germany
BOD	20-25	10-15	10-15	10-15	30-35	20-25	20-25
TSS	20-25	15-25		15-25	30-35	30-35	30-35
N total	3-5	3-5		3-5	5-7	5-7	4-6
P total	0.5-1	0.4-0.6		0.4-06	0.8-1.2	0.8-1.2	0.7-1

6.5 Important components

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 of municipal wastewater varies significantly from one location to another. At a given location the composition will vary with time. This is partly due to variations in the amounts of substances discharged. However, the main reasons are variations in water consumption in households and infiltration and exfiltration during transport in the sewage system.

The composition of typical domestic/municipal wastewater is shown in Table 6.6 where concentrated wastewater (high) represents cases with low water consumption and/or infiltration. Diluted wastewater (low) represents high water consumption and/or infiltration. Storm water will further dilute the wastewater as most storm water components have lower concentrations compared to a lot of diluted wastewater.

Table 6.6 Typical composition of raw municipal wastewater with minor contributions of industrial wastewater

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 treatment options for wastewater. Since most of the nutrients are normally soluble, they cannot be removed by settling, filtration, flotation or other means of solid-liquid separation. Table 6.7 gives typical levels for these components.

In general, the distribution between soluble and suspended matter is important in relation to the characterization of wastewater (Table 6.8).

Since most wastewater treatment processes are based on biological degradation and conversion of the substances, the degradability of the components is important (Table 6.9).

Table 6.7 Typical content of nutrients in raw municipal wastewater with minor contributions of industrial wastewater (in g/m^3)

Parameter	High	Medium	Low
N total	100	60	30
Ammonia N	75	45	20
Nitrate + Nitrite N	0.5	0.2	0.1
Organic N	25	10	15
Total Kjeldahl N	100	60	30
P total	25	15	6
Ortho-P	15	10	4
Organic P	10	5	2

Table 6.8 Distribution of soluble and suspended material for medium-concentrated municipal wastewater (in g/m^3)

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

Table 6.9 Degradability of medium-concentrated municipal wastewater (in g/m^3)

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

6.6 Special components

Most components in wastewater are not the direct target for treatment, but they contribute to the toxicity of the wastewater, either in relation to the biological processes in the treatment plant or to the receiving waters. The substances which are found in the effluent might end up in a drinking water supply system in which case it is dependent on surface water extraction. The metals in wastewater can influence the possibilities for reuse of the wastewater treatment sludge for farmland. Typical values for metals in municipal wastewater are given in Table 6.10.

Table 6.10 Typical content of metals in municipal wastewater with minor contributions of industrial wastewater (in mg/m^3) (Henze 1982, 1992, Ødegaard 1992, from Henze *et al.*, 2001)

Metal	High	Medium	Low
Aluminium	1,000	600	350
Cadmium	4	2	1
Chromium	40	25	10
Copper	100	70	30
Lead	80	60	25
Mercury	3	2	1
Nickel	40	25	10
Silver	10	7	3
Zinc	300	200	100

Table 6.11 gives a range of hydro-chemical parameters for domestic/municipal wastewater.

Table 6.11 Different parameters in municipal wastewater (from Henze, 1982)

Parameter	High	Medium	Low	Unit
Absolute viscosity	0.001	0.001	0.001	kg/m.s
Surface tension	50	55	60	Dyn/cm^2
Conductivity	120	100	70	mS/m^1
pH	8.0	7.5	7.0	
Alkalinity	7	4	1	Eqv/m^3
Sulphide	10	0.5	0.1	gS/m^3
Cyanide	0.05	0.030	0.02	g/m^3
Chloride	600	400	200	gCl/m^3

Wastewater may also contain specific pollutants such as xenobiotics (Table 6.12).

Table 6.12 Special parameters in wastewater such as xenobiotics with toxic and other effects (in mg/L)

Parameter	High	Medium	Low
Phenol	0.1	0.05	0.02
Phthalates, DEHP	0.3	0.2	0.1
Nonylphenols, NPE	0.08	0.05	0.01
PAH	2.5	1.5	0.5
Methylene chloride	0.05	0.03	0.01
LAS	10,000	6,000	3,000
Chloroform	0.01	0.05	0.01



Figure 6.2 Hydrogen sulphide is often present in the influent to treatment plants, especially in the case of pressurized sewers. It is very toxic and can result in casualties if personnel do not take the necessary precautions. The picture shows measurement in a pumping station with high hydrogen sulphide concentration in the air (photo: M. Henze).



Figure 6.3 Detergents in high concentrations create problems for a wastewater treatment plant operator (photo: M. Henze)

6.7 Microorganisms

Wastewater is infectious. Most historic wastewater handling was driven by the wish to move the infectious elements to outside the reach of the population in the cities. In the 19th century microorganisms were identified as the cause of diseases. The microorganisms in wastewater come mainly from human excreta, as well as from the food industry. Table 6.13 gives an idea of the concentration of microorganisms in domestic wastewater.

Table 6.13 Concentrations of microorganisms in wastewater (number of microorganisms per 100 mL) (based on Henze *et al.*, 2001)

Microorganisms	High	Low
E. coli	$5 \cdot 10^8$	10^6
Coliforms	10^{13}	10^{11}
Cl. perfringens	$5 \cdot 10^4$	10^3
Fecal *Streptococcae*	10^8	10^6
Salmonella	300	50
Campylobacter	10^5	$5 \cdot 10^3$
Listeria	10^4	$5 \cdot 10^2$
Staphylococus aureus	10^5	5.10^3
Coliphages	$5 \cdot 10^5$	10^4
Giardia	10^3	10^2
Roundworms	20	5
Enterovirus	10^4	10^3
Rotavirus	100	20

The high concentration of microorganisms may create a severe health risk when raw wastewater is discharged into receiving waters.



Figure 6.4 Surface aeration in activated sludge treatment plants creates aerosols which contain large amounts of microorganisms. This poses a health risk to treatment plant employees and in some cases to the plant's neighbours.

6.8 Special wastewaters and internal plant recycle streams

It is not only the wastewater in the sewerage that a treatment plant has to handle. The bigger the plant, the more internal wastewater recycles and also external inputs/flows have to be handled.

If the catchment has areas with decentralised wastewater handling, septic tank sludge will be unloaded into the plant by trucks. Table 6.14 shows the typical composition of septic sludge. This is a typical situation in many developing countries.

Table 6.14 Composition of septic sludge (in g/m^3) (from Henze *et al.*, 2001)

Compound	High	Low
BOD total	30,000	2,000
BOD soluble	1,000	100
COD total	90,000	6,000
COD soluble	2,000	200
N total	1,500	200
Ammonia N	150	50
P total	300	40
TSS	100,000	7,000
VSS	60,000	4,000
Chloride	300	50
H_2S	20	1
pH	8.5	7.0
Alkalinity[1]	40	10
Lead	0.03	0.01
Fe total	200	20
F. coliforms[2]	10^8	10^6

[1] in milliequivalent/L
[2] in No/100 ml

Septic tank sludge can often create problems in biological treatment plants due to a sudden load from a full truck. For treatment plants of over 100,000 person equivalents the unloading of a truck with septic sludge will not create direct problems in the plant. For small treatment plants the septic tank sludge must be unloaded into a storage tank (Figure 6.5), from which it can be pumped to the plant in periods of low loading (often during the night).



Figure 6.5 A truck discharges the content of septic tanks from households into a storage tank at the wastewater treatment plant Illidge Road at St. Maarten, N.A. (photo: D. Brdjanovic)

Another significant external load on a treatment plant can be landfill leachate (Figure 6.6).



Figure 6.6 Collection and storage of leachate at a sanitary landfill for Sarajevo in Bosnia and Herzegovina (photo: F. Babić)

Leachate can be transported or pumped to the central treatment plant. However, it is sometimes simply dumped into the sewer near the landfill. Leachate can contain high concentrations of soluble inert COD which passes through the plant without any reduction or change. In some cases where regulations do not allow discharge of untreated leachate, separate pre-treatment of leachate is required on-site prior to its discharge into a public sewer.

Table 6.15 Leachate quality (in g/m^3)

Parameter	High	Low
COD total	16,000	1,200
COD soluble	15,800	1,150
BOD total	12,000	300
N total	500	100
Ammonia N	475	95
P total	10	1
TSS	500	20
VSS	300	15
Chloride	2,500	200
H_2S	10	1
pH	7.2	6.5

Internal loading at treatment plants is caused by thickening and digester supernatant, reject water from sludge dewatering and filter wash water. Digester supernatant is often a significant internal load, especially concerning ammonia (Table 6.16). This can lead to an overload of nitrogen in the case of biological nitrogen removal.

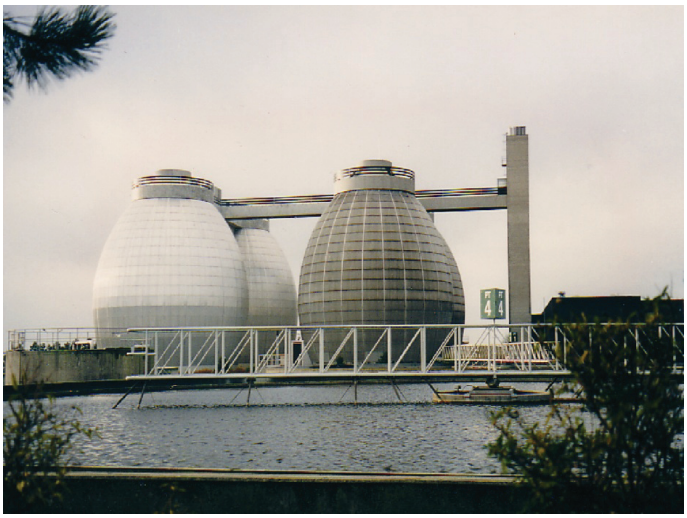

Figure 6.7 Digesters produce digester supernatant which often gives rise to problems in wastewater treatment plants due to the high loads of nitrogen and other substances (photo: M. Henze)

Table 6.16 Digester supernatant (in g/m^3)

Compound	High	Low
COD total	9,000	700
COD soluble	2,000	200
BOD total	4,000	300
BOD soluble	1,000	100
N total	800	120
Ammonia N	500	100
P total	300	15
TSS	10,000	500
VSS	6,000	250
H_2S	20	2

Reject water from sludge dewatering can have rather high concentrations of soluble material, both organics and nitrogen (Figure 6.8).

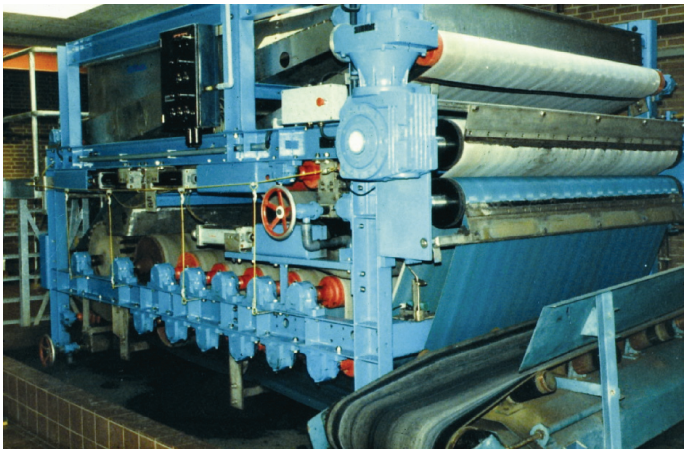

Figure 6.8 A belt filter for sludge dewatering: reject water collection takes place underneath the machinery (photo: D. Brdjanovic)

Table 6.17 Composition of reject water from sludge dewatering (in g/m^3)

Compound	High	Low
COD total	4,000	800
COD soluble	3,000	600
BOD total	1,500	300
BOD soluble	1,000	250
N total	500	100
Ammonia N	450	95
P total	20	5
TSS	1,000	100
VSS	600	60
H_2S	20	0.2

Filter wash water can create problems due to high hydraulic overload of the settling tanks in treatment plants. In some cases it can result in overload with suspended solids. Filter wash water in smaller treatment plants should be recycled slowly.

Table 6.18 Filter wash water (in g/m^3)

Compound	High	Low
COD total	1,500	300
COD soluble	200	40
BOD total	400	50
BOD soluble	30	10
N total	100	25
Ammonia N	10	1
P total	50	5
TSS	1,500	300
VSS	900	150
H_2S	0.1	0.01

6.9 Ratios

The ratio between the various components in wastewater has significant influence on the selection and functioning of wastewater treatment processes. Wastewater with a low carbon to nitrogen ratio may need an external carbon source addition in order that biological denitrification functions fast and efficiently. Wastewater with a high nitrate concentration or low concentration of volatile fatty acids (VFAs) will not be suitable for biological phosphorus removal. Wastewater with a high COD to BOD ratio indicates that a substantial part of the organic matter will be difficult to degrade biologically. When the suspended solids in wastewater have a high volatile component (VSS to SS ratio) these can be successfully digested under anaerobic conditions.

While most of the pollution load in wastewater originates from households, institutions and industry, these contribute only partially to the total quantity of sewage. A significant amount of water in sewage may originate from rainwater (in some countries snow melting) or infiltration. Thus wastewater components are subject to dilution, which however will not change the ratios between the components. Table 6.19 shows typical component ratios in municipal wastewater.

Table 6.19 Typical ratios in municipal wastewater

Ratio	High	Medium	Low
COD/BOD	2.5-3.5	2.0-2.5	1.5-2.0
VFA/COD	0.12-0.08	0.08-0.04	0.04-0.02
COD/TN	12-16	8-12	6-8
COD/TP	45-60	35-45	20-35
BOD/TN	6-8	4-6	3-4
BOD/TP	20-30	15-20	10-15
COD/VSS	1.6-2.0	1.4-1.6	1.2-1.4
VSS/TSS	0.8-0.9	0.6-0.8	0.4-0.6
COD/TOC	3-3.5	2.5-3	2-2.5

The ratio between the components in a given wastewater analysis can also be used to investigate anomalies in the analysis which can be due to special discharges into the sewer system, often from industries, or due to analytical errors. The ratios between the concentrations

of the components shown in Table 6.6 can be used as a rough guideline. If some of the analytical values fall outside the expected range provided in Table 6.6 this should be further investigated and the reason found. If industrial discharges cause the discrepancy, other not (already) analysed components in the wastewater might also deviate from the expected values. Since these discrepancies may affect the treatment process the reason for their appearance should be clarified.

6.10 Variations

The concentration of substances in wastewater varies with time. In many cases daily variations are observed, in some weekly and others are very likely a function of industrial production patterns. The variations are important for the design, operation and control of the treatment plant. For example, ammonia-nitrogen, the main source of which is urine, often shows a diurnal pattern depicted by Figure 6.9. Variations in flow, COD and suspended solids can be significant as shown in Figure 6.10.

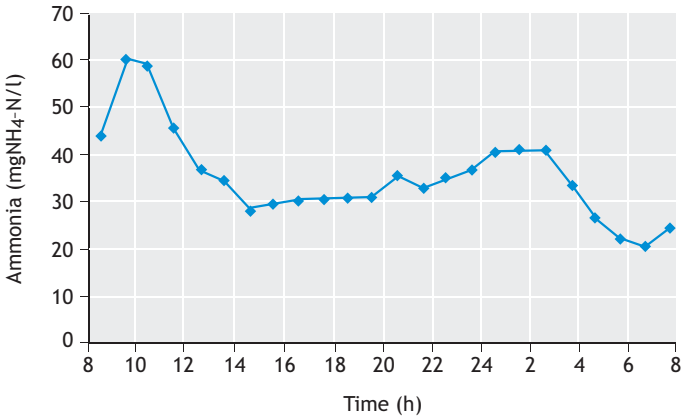

Figure 6.9 Daily variation in the ammonia content in the influent of the Galindo wastewater treatment plant in Spain

Sampling of wastewater is challenging due to the variations in flow and component contractions. It is important to be aware of the fact that the analytical results obtained will vary considerably with the chosen sampling procedure. Floatable materials such as oil and grease are difficult to sample; so are comparatively heavier components, such as sand and grit.

A number of sampling techniques are applied to wastewater:

- Grab samples (one sample collected in a bottle or a bucket at a specific time). This type of sampling gives highly variable results.
- Time proportional samples (this can be a number of samples, e.g. one sample per hour which is combined into one final sample). This type of sampling can be acceptable if the wastewater has only small variations in the concentration of its components.

- Flow proportional sampling (this can be a sample for each specified volume of wastewater flow, typically performed over 24 hours). This gives a reliable estimate of the wastewater quality – or lack of quality.
- 24 hour variations (e.g. one sample per hour kept separate in order to obtain an impression of the variations in wastewater concentrations). These are beneficial for modeling purposes.
- Weekly samples (time or flow proportional). Similarly, these are beneficial for design and modeling purposes.

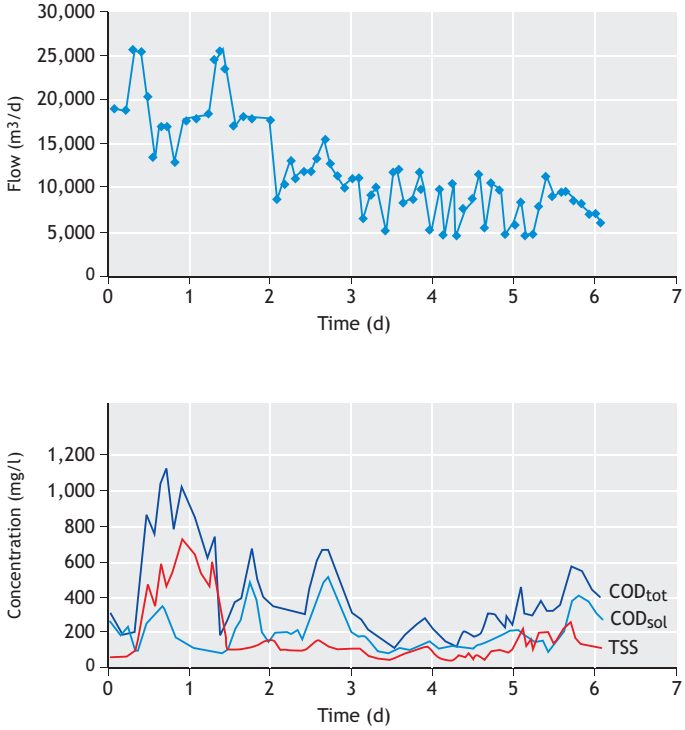

Figure 6.10 Variations in wastewater flow, COD and suspended solids (Henze *et al.*, 2002)

6.11 Wastewater flows

Wastewater flows vary with time and place. This makes them complicated to measure accurately. The basic unit for flow is volume of wastewater (m^3) per unit of time (day). The design flow for different units in a wastewater treatment plant varies. For units with a short hydraulic retention time such as a screen and grit chamber, the design flow is represented by m^3/s, while for settling tanks the design flow is usually expressed by m^3/h. For domestic wastewater typical design calculations are shown in Figure 6.11.

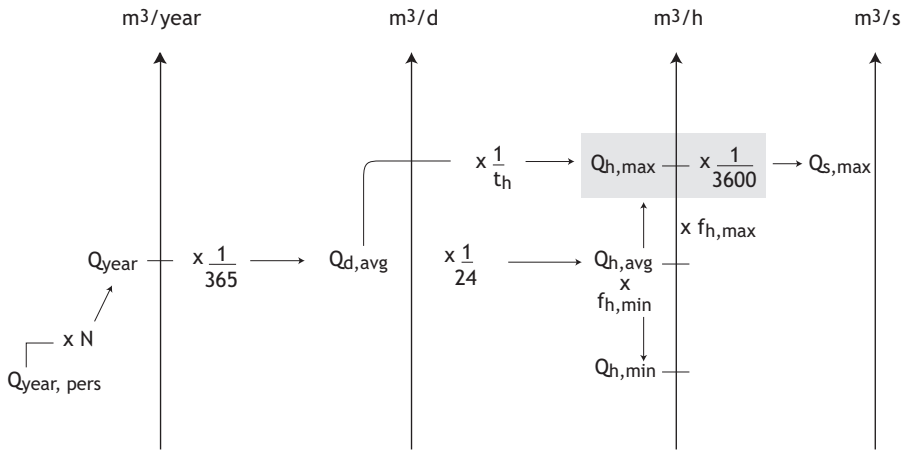

Figure 6.11 Calculation of design volumes for municipal wastewater with a minor industrial wastewater contribution

Average daily flow, $Q_{d,avg}$, is calculated as wastewater flow per year divided by 365. Average hourly flow, $Q_{h,avg}$, is daily flow divided by 24.

The maximum flow per hour can be found by two calculations, either (*i*) from the average daily flow multiplied by the maximum hourly constant, $f_{h,max}$ (this constant varies with the size of the catchment: for large cities it will be 1.3-1.7, and for small towns 1.7-2.4), or (*ii*) by dividing the average daily flow by the hourly factor, $t_{h,d}$ (this factor is 10-14 hours for small towns, and 14-18 hours for large cities).

6.12 Traditional waste from households

The amount of wastewater and pollutants from households varies from country to country. These variations are influenced by the climate, socio-economic factors, household technology and other factors. The amount of organic waste and nutrients produced in households is shown in Table 6.20. From this table one can realize the potential for changing the wastewater composition.

Table 6.20 Sources for household wastewater components and their values for 'non-ecological' lifestyle (from Sundberg, 1995; Henze, 1997)

Parameter	Unit	Toilet		Kitchen	Bath/ laundry	Total
		Total[1]	Urine			
Wastewater	m^3/yr	19	11	18	18	55
COD	kg/yr	27.5	5.5	16	3.7	47.2
BOD	kg/yr	9.1	1.8	11	1.8	21.9
N	kg/yr	4.4	4.0	0.3	0.4	5.1
P	kg/yr	0.7	0.5	0.07	0.1	0.87
K	kg/yr	1.3	0.9	0.15	0.15	1.6

[1] Including urine

In the case of household waste, the composition of wastewater and solid waste from households is the result of contributions from various sources within the household. It is possible to change the amount and the composition of the waste streams. The amount of a given waste stream can be decreased or increased, depending on the optimal solution. For example, a reduction in the amount of waste(d) materials present in the wastewater can be achieved by two means: (*i*) overall reduction of waste generated in the household and, (*ii*) diversion of certain types of waste to the solid waste of the household.

Options for reducing the physiologically generated amount of waste are not obvious, although diet influences the amount of waste produced by the human organism. Thus one has to accept this waste generation as a natural result of human activity. Separating toilet waste (physiological waste or anthropogenic waste) from the waterborne route is reflected in a significant reduction in the nitrogen, phosphorus and organic load in the wastewater. Waste generated after the separation at source has taken place, however, still has to be transported away from the household, and in many cases, the city.

There are several feasible technical options for handling waste separated at source, including:

- The night soil system, used worldwide;
- Compost toilets, mainly used in individual homes in agricultural areas (preferably with urine separation in order to optimise the composting process);
- Septic tanks followed by infiltration or transport by a sewer system.

Urine is the main contributor to nutrients in household wastes, thus separating out the urine will significantly reduce nutrient loads in wastewater (Figure 6.12). Urine separation will reduce nitrogen content in domestic wastewater to a level where nitrogen removal is not needed.



Figure 6.12 Urine-separating toilet

Kitchen waste contains a significant amount of organic matter which traditionally ends up in wastewater. It is relatively easy to divert some liquid kitchen wastes to solid waste by the application of so-called ‘cleantech’ cooking, thus obtaining a significant reduction in the overall organic load of the wastewater (Danish EPA 1993). Cleantech cooking means that food waste is discarded into the waste bin and not flushed into the sewer using water from the tap. The diverted part of the solid organic waste from the kitchen can be disposed together with the other solid wastes from the household. The grey wastewater from the kitchen could be used for irrigation or, after treatment, for toilet flushing. Liquid kitchen waste also contains household

chemicals, the use of which can affect the composition and load of this type of waste. Wastewater from laundry and bath only carries a minor pollution load, part of which comes from household chemicals, the use of which can affect the composition and the load of this waste fraction. Waste from laundry and baths could be used together with the traditional kitchen wastewater for irrigation. Alternatively, it can be reused for toilet flushing. In both cases considerable treatment is needed.

The compostable fraction of the solid waste from the kitchen can either be kept separate or combined with traditionally waterborne kitchen wastes, for later composting or anaerobic treatment at the wastewater treatment plant. The use of kitchen disposal units (grinders) for handling the compostable fraction of the solid waste from households is common in many countries. Sometimes this option is discontinued due to the increased waste load on the sewer. However, waste is generated in households, and it must be transported away from households and out of cities by some means. The discharge of solid waste to the sewer does not change the total waste load produced by the household, but it will change the transportation means and the final destination of the waste.

6.13 Wastewater design for households

The use of one or more of the waste handling technologies mentioned earlier in households in combination with water-saving mechanisms makes it possible to design wastewater with a specified composition, which will be optimal for its further handling. When the goal is to reduce the pollutant load on the wastewater, there are several actions that can achieve this (Table 6.21).

Table 6.21 Reduced waste load to wastewater by toilet separation and cleantech cooking (in g/cap.d) (from Henze, 1997)

Technology	Traditional	Toilet separation[1]	Clean-tech cooking[2]
COD	130	55	32
BOD	60	35	20
N	13	2	1.5
P	2.5	0.5	0.4

[1] Water closet → dry/compost toilet.
[2] Part of cooking waste diverted from the sink to solid waste bin

The coupling of water saving and load reduction is an additional argument for the wastewater design approach (example shown in Table 6.22).

Table 6.22 The concentration of pollutants in raw wastewater with toilet separation and cleantech cooking (in g/m^3) (from Henze 1997)

Wastewater production	250 L/cap.d	160 L/cap.d	80 L/cap.d
COD	130	200	400
BOD	80	125	250
N	6	9	19
P[1]	1.6	2.5	5

[1] Assuming phosphate-free detergents

The changes obtained in the wastewater composition also influence the detailed composition of the COD. This can result in changes between the soluble and the suspended fractions, or changes in degradability of the organic matter, for example, leading to more or less easily degradable organic matter in the given wastewater fraction. The composition of wastewater has

a significant influence on the selection of treatment processes to be applied. By changing technology used in households and by diverting as much of the organic waste to the sewer system as possible, it is possible to obtain wastewater characteristics such as those shown in Table 6.23.

Table 6.23 Concentration of pollutants in raw wastewater by maximum load of organic waste (in g/m^3) (Henze, 1997)

Wastewater production	250 L/cap.d	160 L/cap.d	80 L/cap.d
COD	880	1,375	2,750
BOD	360	565	1,125
N	59	92	184
P[1]	11	17	35

[1] Assuming phosphate-free detergents

The tendency for having rather detailed wastewater and biomass fractionation is the result of the increasing application and requirements of mathematical models in wastewater treatment. In order to place COD, N, and P fractionation in the wider context of mathematical modeling, the list of state variables used by selected models is composed, as depicted in Table 6.25. Here the authors made a proposal for an overarching list of common state variables (the second column in Table 6.25). For the description of each component presented in this table, the reader is referred to the list of references given in the footnote of the table.

The most common separation is the separation of toilet waste from the rest of the wastewater. This will result in grey and black wastewater generation, the characteristics of which can be seen in Table 6.24. For more details on grey wastewater, see Ledin *et al.,* 2000.

Table 6.24 Characteristics of grey and black wastewater. Low values can be due to high water consumption. Low water consumption or high pollution load from kitchens can cause high values (based on Henze, 1997; Sundberg, 1995; Almeida *et al,*. 2000)

Parameter	Grey wastewater		Black wastewater	
	High	Low	High	Low
COD	700	200	1,500	900
BOD	400	100	600	300
N	30	8	300	100
P	7	2	40	20
K[1]	6	2	90	40

[1] Exclusive of the content in the water supply

6.14 Wastewater and biomass fractionation

The relationship between various components of organic and inorganic matter, nitrogen and phosphorus components of either wastewater or sludge are illustrated in Figure 6.14. The definition of each term is given in Table 6.25. For a more detailed description of each component presented here the reader is referred to the list of references attached.

Variable names vary between references depending on the authors' preferences. In this book, the notation used for variables was not standardized but a discussion on this topic was initiated with researchers interested in modeling (Comeau *et al.,* 2008) and the following indications were suggested as guidelines for the notation of variables.

First, a letter indicates the size of the component (capital letter in italics):

- *S*: soluble
- *C*: colloidal
- *X*: particulate
- *T*: total (= *S + C + X*).

The particle size of colloidal matter depends on the purpose of the model used and the method of its determination and may typically be in the range 0.01 to 1 micron. Modeling colloidal matter has increased in importance in recent years due to the need to reach very low effluent concentrations, a condition when the behaviour of colloidal matter becomes significant. Advanced treatment systems including membrane or adsorption processes are increasingly used for such purposes. In some cases, it may be useful to join the letters indicating the size of the matter (e.g. *CX*).

Subscripts are then used to describe the component or its nature (e.g. F: fermentable; OHO: ordinary heterotrophic organisms). Commas may be added to indicate that one component is part of another (e.g. $X_{PAO,PHA}$ for the polyhydroxyalkanoate (PHA) storage component of phosphorus accumulating organisms; PAO). Organisms are proposed to be described with an acronym ending with the letter 'O' (e.g. ANO: ammonia nitrifying organisms).

Each state variable is considered independent of each other (but this is not true for combined variables). Thus, for example, the PHA storage of PAO ($X_{PAO,PHA}$) is not considered to be part of the PAO (X_{PAO}).

Total matter (*T*) is composed of inorganic (IG) and organic (ORG) components, the latter being divided in biodegradable (B) and unbiodegradable (U) matter. The word *unbiodegradable* was proposed instead of *inert,* notably to avoid using the letter 'I' to minimize the risk of confusion with inorganic matter.

Variable names may be used for any location of a wastewater treatment system. It is proposed that a lower case superscript be used to indicate the location of the variable, when needed (e.g. X_{OHO}^{inf} for the OHO's concentration in the influent). Considering that some influent particulate unbiodegradable compounds accumulate in the activated sludge as a function of sludge age and hydraulic retention time, it is sometimes necessary to identify both their source and location (e.g. $X_{INF,U}^{OX}$ for the influent unbiodegradable component in the aerobic zone [OX] of the process).

The component is considered to be expressed in units of dry weight concentration (e.g. mg S_{VFA}/l). The various constituents of this component contribute to its concentration in other units of COD, BOD_U (ultimate BOD), BOD_5, residue (solids), nitrogen and phosphorus using appropriate conversion factors as needed to express them in these units. For example, expressing the VFA concentration in units of COD would require the state variable to be expressed as S_{VFA_COD} with the underscore used as a separator to specify the units of expression. However, since organic matter components in activated sludge models were expressed in COD units by default, the proposed symbol for a variable name in this table is shown without the underscore to indicate COD units (e.g. S_{VFA_COD} is shown as S_{VFA}). Similarly, components that contain essentially only nitrogen or phosphorus have no units specified in the variable name with units being indicated in the Units column (e.g. S_{PO4} instead of S_{PO4_P}).

When a component contributes to COD, BOD (if biodegradable), residue (solids), nitrogen and phosphorus, a star (*) is shown in the appropriate column of Figure 6.14. Note that for expressing variables in units of residues, since the components are considered to be expressed in dry weight units, no '_R' would strictly be required. Optionally, it may be used to specify that the compound is expressed in residue units, especially if the symbols were defined on a COD-unit basis, as often done in activated sludge models.

BOD components are carbonaceous BOD. BOD_U is about 10% less than the corresponding biodegradable COD components. The BOD_5/BOD_U ratio depends on the type of wastewater but is typically 0.67. Oxygen is considered to exert both a negative COD and a negative BOD.

6.15 Symbols of variables for models

A list of symbols for state variables used for various activated sludge models is shown in Table 6.25. Some state and combined variables that were not used in these models but are shown in Figure 6.14 are also described.

6.16 Characterization protocols

Driven by requirements of mathematical modeling of activated sludge systems, several systematic protocols for activated sludge model calibration have been developed and include different wastewater characterization protocols. Four major protocols were developed by as many research groups. The nature of these protocols range from simplified and practical in nature, to those of increased complexity and more of academic and research interest.

- STOWA protocol (Hulsbeek *et al.*, 2002);
- BIOMATH protocol (Vanrolleghem *et al.*, 2003);
- WERF protocol for model calibration (Melcer *et al.*, 2003);
- Hochschulgruppe (HSG) guidelines (Langergraber *et al.,* 2004).

Which protocol to use depends on the purpose of the modeling. For more details on the activated sludge modeling protocols the reader is referred to Chapter 7 and Chapter 8. Typical fractions of the total influent COD for raw and primary effluent wastewaters are shown in Table 6.26 (adapted from EnviroSim, 2007).

6.17 Example composition of influent, bio-reactor and effluent

An example of concentration values for various state and combined variables for the influent, the aerated zone and the effluent of a Phoredox© process is shown in Figure 6.15.



Figure 6.13 Execution of sampling and monitoring program requires expertise and financial resources, but often returns multiple benefits including optimization of plant design, improved operation of wastewater facilities and overall savings (photo: K-water)

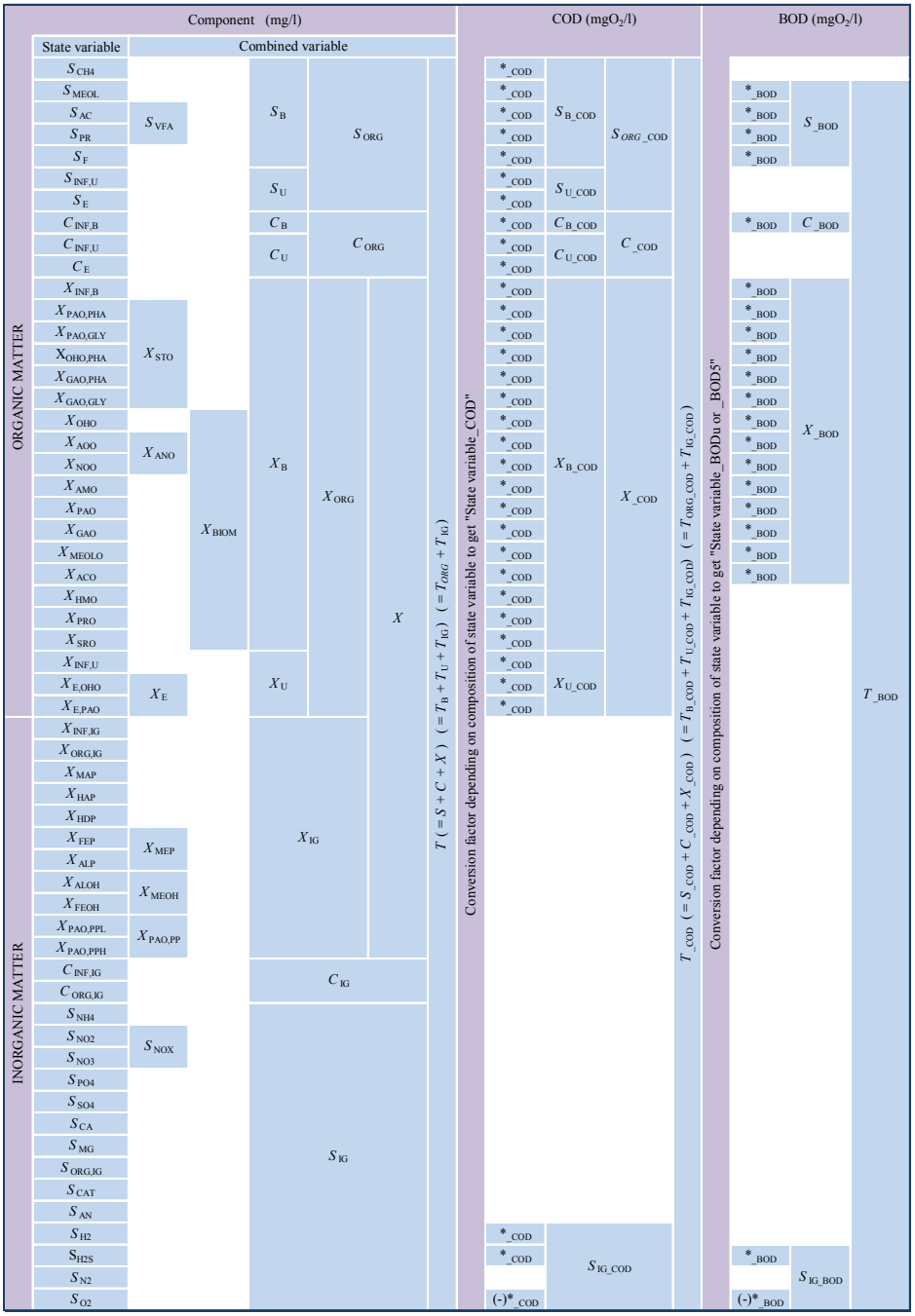

Figure 6.14 Fractionation of organic and inorganic matter components, and relationships between their content in dry weight, COD, BOD, residue (solids), nitrogen and phosphorus (continued from the previous page).

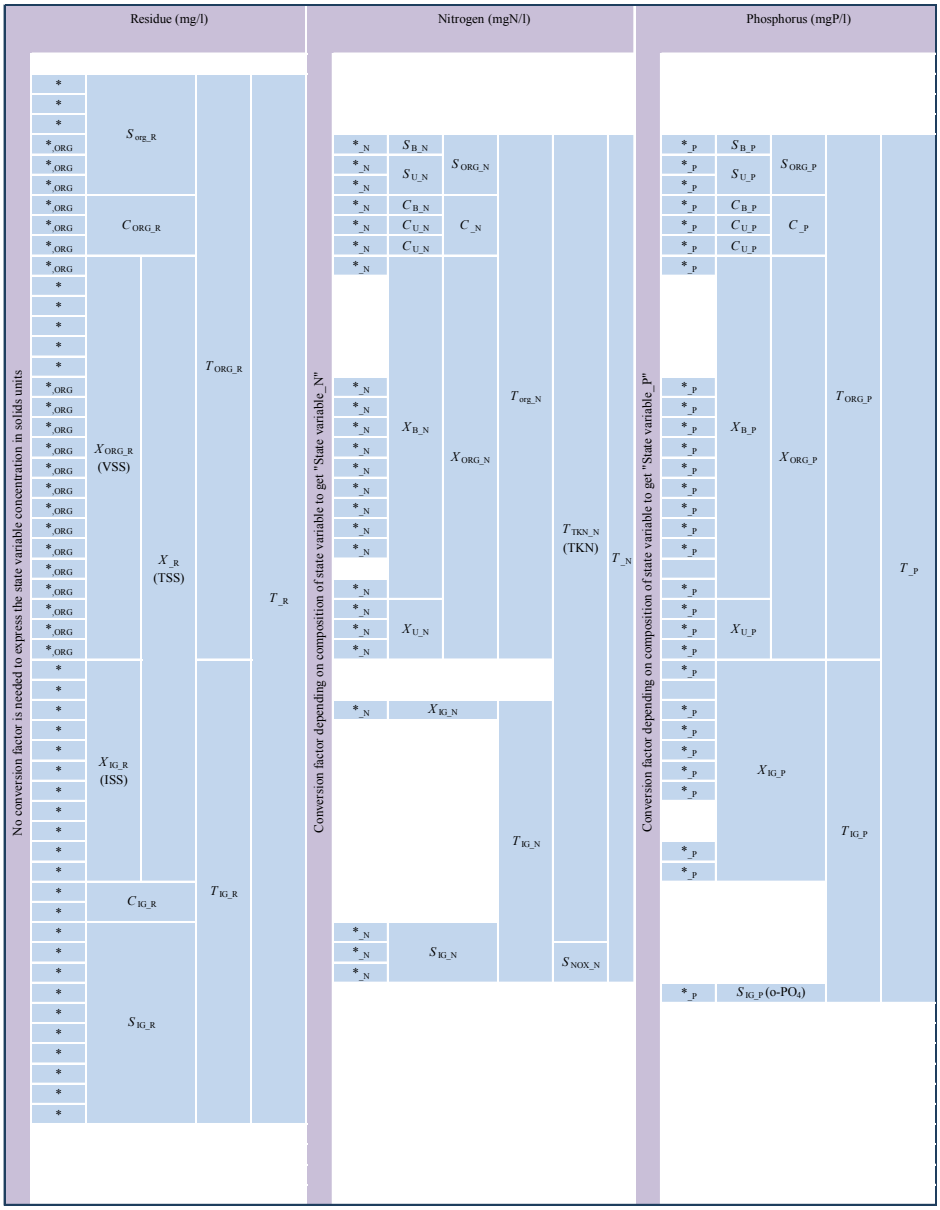

Figure 16.4 Continued

Table 6.25 Symbol list of variables for various models

Group	Proposed symbol	Units	Description	ASM1 [1]	ASM2D [2]	ASM3P [3]	GenASDM [4]	UCTPHO+ [5]	TUDP [6]
S_{COD}									
	S_{CH4}	mgCOD/L	Methane				S_{CH4}		
	S_{MEOL}	mgCOD/L	Methanol				S_{BMETH}		
	S_{AC}	mgCOD/L	Acetate				S_{BSA}		
	S_{PR}	mgCOD/L	Propionate				S_{BSP}		
	S_{VFA}	mgCOD/L	Volatile fatty acids		S_{LF}			S_A	S_{AC}
	S_F	mgCOD/L	Fermentable organic matter		S_F		S_{BSC}	S_F	
	S_B	mgCOD/L	Soluble biodegradable matter	S_S		S_S			
	$S_{INF,U}$	mgCOD/L	Influent soluble unbiodegradable organics	S_I	S_I	S_I	S_{US}	S_I	
	S_E	mgCOD/L	Soluble unbiodegradable endogenous products						
	S_U	mgCOD/L	Soluble unbiodegradable organic matter						
	S_{ORG}	mgCOD/L	Soluble organic matter						
	S_{H2}	mgCOD/L	Dissolved hydrogen				S_{BH2}		
	S_{H2S}	mgCOD/L	Dissolved hydrogen sulfide						
O_2									
	S_{O2}	mgO_2/L	Dissolved oxygen	S_O	S_O	S_O	DO	S_{O2}	S_{O2}
C_{COD} and X_{COD}									
	$C_{INF,B}$	mgCOD/L	Influent slowly biodegradable colloidal matter				X_{SC}		
	C_B	mgCOD/L	Slowly biodegradable colloidal matter						
	$C_{INF,U}$	mgCOD/L	Influent unbiodegradable colloidal matter						
	C_E	mgCOD/L	Colloidal unbiodegradable matter						
	C_U	mgCOD/L	Unbiodegradable colloidal matter						
	C_{ORG}	mgCOD/L	Colloidal organic matter						
	$X_{INF,B}$	mgCOD/L	Influent slowly biodegradable particulate organics (non-colloidal)				X_{SP}		
	$CX_{INF,B}$	mgCOD/L	Influent slowly biodegradable organics (colloidal and particulate)	X_S	X_S	X_S			
	$X_{INF,B,ENM}$	mgCOD/L	Influent $CX_{INF,B}$ instantaneously enmeshed onto the biomass					X_{ENM}	
	$X_{ADS,B}$	mgCOD/L	$X_{INF,B,ENM}$ adsorbed or produced from biomass decay					X_{ADS}	
	$X_{PAO,PHA}$	mgCOD/L	Stored polyhydroxyalkanoates (PHA) in phosphorus accumulating organisms (PAO)			X_{PHA}	S_{PHB}	X_{PHA}	X_{PHB}
	$X_{PAO,GLY}$	mgCOD/L	Stored glycogen in PAO						X_{GLY}
	$X_{OHO,PHA}$	mgCOD/L	Stored PHA in OHO						
	$X_{GAO,PHA}$	mgCOD/L	Stored PHA in GAO						
	$X_{GAO,GLY}$	mgCOD/L	Stored glycogen in GAO						
	X_{STO}	mgCOD/L	Stored PHA and glycogen		X_{BT}	X_{STO}			
	X_B	mgCOD/L	Particulate biodegradable organics						
	$X_{INF,U}$	mgCOD/L	Particulate unbiodegradable organics from the influent						
	$X_{E,OHO}$	mgCOD/L	Particulate unbiodegradable endogen. products from OHO						
	$X_{E,PAO}$	mgCOD/L	Particulate unbiodegradable endogen. products from PAO						
	X_E	mgCOD/L	Particulate unbiodegradable endogenous products	X_U			Z_E	X_E	
	X_U	mgCOD/L	Particulate unbiodegradable organics	X_I	X_I	X_I	X_I	X_I	X_I
	X_{ORG}	mgCOD/L	Particulate organic matter						

Group	Proposed symbol	Units	Description	ASM1 [1]	ASM2D [2]	ASM3P [3]	GenASDM [4]	UCTPHO+ [5]	TUDP [6]
Organisms									
	X_{OHO}	mgCOD/L	Ordinary heterotrophic organisms (OHO)	$X_{B,H}$	X_{BH}	X_H	Z_{BH}	X_H	
	X_{AOO}	mgCOD/L	Ammonia-oxidizing organisms				Z_{BA}		X_{NH}
	X_{NOO}	mgCOD/L	Nitrite-oxidizing organisms				Z_{BN}		X_{NO}
	X_{ANO}	mgCOD/L	Autotrophic-nitrifying organisms (NH_4^+ to NO_3^-)	$X_{B,A}$	X_{BA}	X_A		X_{AUT}	
	X_{AMO}	mgCOD/L	Anaerobic ammonia-oxidizing (Annamox) organisms				Z_{BAMO}		
	X_{PAO}	mgCOD/L	Phosphorus-accumulating organisms (PAO)		X_{BP}	X_{PAO}	Z_{BP}	X_{PAO}	X_{PAO}
	X_{GAO}	mgCOD/L	Glycogen-accumulating organisms (GAO)						
	X_{MEOLO}	mgCOD/L	Anoxic methanol-utilizing methylotrophic organisms				Z_{BMETH}		
	X_{ACO}	mgCOD/L	Acetoclastic methanogenic organisms				Z_{BAM}		
	X_{HMO}	mgCOD/L	Hydrogenotrophic methanogenic organisms				Z_{BHM}		
	X_{PRO}	mgCOD/L	Propionic acetogenic organisms				Z_{BPA}		
	X_{SRO}	mgCOD/L	Sulfate-reducing organisms						
	X_{BIOM}	mgCOD/L	Organisms (biomass)						
Inorganics									
	$X_{INF,IG}$	mgISS/L	Influent particulate inorganics (excluding other state variables)						
	$X_{ORG,IG}$	mgISS/L	Inorganics that are associated with organic matter (including organisms)						
	X_{MAP}	mgISS/L	Struvite (magnesium ammonium phosphate)				X_{STRU}		
	X_{HAP}	mgISS/L	Hydroxyapatite				X_{HAP}		
	X_{HDP}	mgISS/L	Hydroxydicalcium-phosphate				X_{HDP}		
	X_{FEP}	mgISS/L	Iron phosphate precipitates						
	X_{ALP}	mgISS/L	Aluminium phosphate precipitates						
	X_{MEP}	mgISS/L	Metal phosphate precipitates		X_{MEP}				
	X_{ALOH}	mgISS/L	Aluminium hydroxide precipitates						
	X_{FEOH}	mgISS/L	Iron hydroxide precipitates						
	X_{MEOH}	mgISS/L	Metal hydroxide precipitates		X_{MEOH}				
	T_{ME}	mgME/L	Metals (Al-Fe)				C_{ME}		
	$X_{PAO,PPL}$	mgP/L	Releasable stored phosphates in PAO				PP_{LO}		
	$X_{PAO,PPH}$	mgP/L	Non-releasable stored phosphates in PAO				PP_{HI}		
	$X_{PAO,PP}$	mgP/L	Stored polyphosphates in PAO		X_{PP}	X_{PP}		X_{PP}	X_{PP}
	X_{IG}	mgISS/L	Particulate inorganic matter						
	X_{B_P}	mgP/L	P content of particulate biodegradable organic matter				X_{OP}		
	X_{U_P}	mgP/L	P content of particulate unbiodegradable organic matter				X_{IP}		
	$C_{INF,IG}$	mgISS/L	Influent colloidal inorganics (excluding other state variables)						
	$C_{ORG,IG}$	mgISS/L	Inorganics associated with colloidal organic matter						
	C_{IG}	mgISS/L	Inorganics present in colloidal matter						
	S_{NH4}	mgN/L	Ammonia ($NH_4^+ + NH_3$)	S_{NH}	S_{NH}	S_{NH}	S_{NH3}	S_{NH4}	S_{NH4}
	S_{NO2}	mgN/L	Nitrite ($HNO_2 + NO_2^-$)				S_{NO2}		S_{NO2}
	S_{NO3}	mgN/L	Nitrate ($HNO_3 + NO_3^-$)				S_{NO3}		S_{NO3}

Group	Proposed symbol	Units	Description	ASM1 [1]	ASM2D [2]	ASM3P [3]	GenASDM [4]	UCTPHO+ [5]	TUDP [6]
	S_{NOX}	mgN/L	Nitrite + nitrate	S_{NO}	S_{NO}	S_{NO}		S_{NO3}	
	S_{PO4}	mgP/L	Inorganic soluble phosphorus (o-PO_4 test)		S_P	S_{PO4}		S_{PO4}	S_{PO4}
	$S_{PO4} + X_{MEP}$	mgP/L	Total phosphate (soluble P + metal-P)				cPO_4		
	$S_{ORG,IG}$	mgISS/L	Inorganics associated with soluble organic matter						
	$X_{PAO,PP,CAT}$	mgISS/L	Polyphosphate bound cations				X_{PPCat}		
	S_{CAT}	meq/L	Other cations (strong bases)				S_{CAT}		
	S_{AN}	meq/L	Other anions (strong acids)				S_{AN}		
	S_{N2}	mg/L	Soluble nitrogen		S_{NN}	S_{N2}	S_{N2}		S_{N2}
	S_{ALK}	mg$CaCO_3$/L	Alkalinity	S_{ALK}	S_{ALK}				
	S_{TIC}	mmolC/L	Total inorganic carbon			S_{HCO}	S_{CO2t}		
Water									
	S_{H2O}	mgH_2O/L	Water				S_{H2O}		
SS									
	X_{ORG_R}	mgVSS/L	Volatile (organic) suspended solids (residue)						
	X_{IG_R}	mgISS/L	Inorganic suspended solids (residue)						
	X_{T_R}	mgTSS/L	Total suspended solids (residue)			X_{TSS}			

[1] ASM1: Henze *et al.* (1987)
[2] ASM2D: Henze *et al.* (1999)
[3] ASM3-P: Rieger *et al.* (2001)
[4] General ASDM: EnviroSim (2007)
[5] UCTPHO+: Hu *et al.* (2007)
[6] TUDP: de Kreuk *et al.* (2007)

Note: 1) Since organic matter components in activated sludge models were expressed in COD units by default, the proposed symbol for a variable name in this table is shown without the underscore to indicate COD units (e.g. $S_{VFA;COD}$ is shown as S_{VFA}). Similarly, components that contain essentially only nitrogen or phosphorus have no units specified in the variable name with units being indicated in the Units column. 2) Some compounds that were not independent of variables shown in Figure 9.13 were not illustrated in this figure (e.g. as $X_{INF,B,ENM}$ and $X_{ADS,B}$ that are related to $CX_{INF,B}$).

Table 6.26 Typical fractions of total COD for raw and primary effluent wastewaters

State variable	Fraction of TCOD	
	Raw wastewater	Primary effluent
S_U	0.03 - 0.08	0.05 - 0.10
S_{VFA}	0.0 - 0.08	0.0 - 0.11
S_F	0.05 - 0.18	0.06 - 0.23
$C_{INF,B}$	0.47 - 0.53	0.29 - 0.36
$X_{INF,B}$	0.16 - 0.19	0.29 - 0.36
X_{OHO}	0.1	0.1
X_U	0.13	0.08

	State variables	Units	Influent	Aerobic	Effluent
ORGANIC MATTER	S_{CH4}	mgCOD/l	0	0.03	0.03
	S_{MEOL}	mgCOD/l	0	0	0
	S_{AC}	mgCOD/l	15	0	0
	S_{PR}	mgCOD/l	5	0.01	0.01
	S_{F}	mgCOD/l	30	1.7	1,7
	$S_{INF,U}$	mgCOD/l	25	25	25
	$C_{INF,B}$	mgCOD/l	15	0	0
	$X_{INF,B}$	mgCOD/l	110	93	0,3
	$X_{PAO,PHA}$	mgCOD/l	1	12	0.04
	X_{OHO}	mgCOD/l	30	1318	4.8
	X_{AOO}	mg COD/l	1	40.0	0.15
	X_{NOO}	mgCOD/l	1	29.8	0.11
	X_{AMO}	mgCOD/l	1	18.5	0.07
	X_{PAO}	mgCOD/l	1	153.6	0.56
	X_{MEOLO}	mgCOD/l	1	17.1	0.06
	X_{ACO}	mgCOD/l	1	7.3	0.03
	X_{HMO}	mgCOD/l	1	8.6	0.03
	X_{PRO}	mgCOD/l	1	8.3	0.03
	$X_{INF,U}$	mgCOD/l	35	681	2.5
	$X_{E,OHO}$	mgCOD/l	0	221	0.8
INORGANIC MATTER	X_{MAP}	mgISS/l	0	0	0
	X_{HAP}	mgISS/l	0.1	1.9	0.01
	X_{HDP}	mgISS/l	0.1	0.0	0.0
	$X_{PAO,PPL}$	mgP/l	0	31	0.11
	$X_{PAO,PPH}$	mgP/l	0	10	0.04
	S_{NH4}	mgN/l	16	1.8	1.8
	S_{NO2}	mgN/l	0.1	0.2	0.2
	S_{NO3}	mgN/l	1.0	4.1	4.1
	S_{PO4}	mgP/l	2.2	0.55	0.55
	S_{CA}	mgCa/l	66	66	66
	S_{MG}	mgMg/l	12	11	11
	S_{CAT}	meq/l	2.5	2.4	2.4
	S_{AN}	meq/l	3.0	3.0	3.0
	S_{H2}	mgCOD/l	0	0.3	0.3
	S_{N2}	mgN/l	15	19	19
	S_{O2}	mgO_2/l	0.0	2.0	2.0

Combined variables		Units	Influent	Aerobic	Effluent
COD	$SC_{_COD}$	mgCOD/l	90	27	27.5
	$X_{_COD}$	mgCOD/l	184	2608	9.6
	$T_{_COD}$	mgCOD/l	274	2636	37.0
BOD_5	$S_{_BOD5}$	mgO_2/l	46	1	1.2
	$X_{_BOD5}$	mgO_2/l	80	973	3.6
	$T_{_BOD5}$	mgO_2/l	126	975	4.8
Residue	X_{ORG_R}	mgVSS/l	118	1775	6.5
	X_{IG_R}	mgISS/l	17	524	1.9
	$X_{_R}$	mgTSS/l	135	2299	8.4
Nitrogen	S_{TKN_N}	mgN/l	17.3	3.3	3.3
	X_{TKN_N}	mgN/l	9.7	191	0.7
	T_{TKN_N}	mgN/l	27.0	194	4.0
	$T_{_N}$	mgN/l	28.1	198	8.3
Phosphorus	X_{B_P}	mgP/l	1.8	1.7	0.01
	X_{U_P}	mgP/l	0.3	10.7	0.04
	$T_{_P}$	mgP/l	6.6	118	0.98

Figure 6.15 Concentration of various components for a Phoredox process with 5 d SRT operated at 12°C

6.18 Our wastewater fingerprint

'Show me your wastewater and I will tell you who you are'. The wastewater from a particular person gives a very detailed picture of that person and their lifestyle. All human activities are registered and reflected in the wastewater, from the food we eat to the materials we use in our houses and the materials and production processes applied in industry. Through the wastewater one can get information on illness, sex, pregnancy, drug use, personal hygiene, diet, environmental awareness, alcoholism, etc. The 'fingerprint' we deliver with the wastewater affects the environment. It is not the wastewater that spoils the environment; it is humans that pollute the water.

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Chapter 7

Guidelines on model wastewater characterization

7.1 Introduction

Increasing use of activated sludge modeling in the Netherlands, among other things also promoted by the Dutch Foundation for Applied Water Research (Dutch acronym: STOWA), led to an immediate demand for standardized guidelines for wastewater characterization. Therefore, at the request of STOWA (STOWA report 1999-13 at www.stowa.nl), an inventory of different methods for the characterization of activated sludge modeling wastewater was performed based on a recently published ASM family of models (ASM1 and ASM2d). This methodology was published and presented on different occasions based on the work of Roeleveld *et al*. (2002). The wastewater characterization guidelines presented here are based on this published work. These guidelines were extended based on extensive practical experience in wastewater treatment modeling and influent characterization following the period after these publications. Several points in the published protocol have been adjusted (further simplified) by the authors to a level that is reliable and practical as well as financially feasible. The main proposed change is the recommendation to no longer rely only on BOD measurements for the determination of the influent biodegradability. Practical experience has shown that these measurements are costly and time-consuming to perform, difficult to reproduce (especially k_{BOD}) and therefore inaccurate. More importantly, the method does not provide the required accuracy for a model study. Instead, the upgraded protocol recommends a direct approach when applying influent characterization for model retrofit studies, which is less time-consuming and ultimately also more accurate. On the other hand, BOD measurements are still used in the protocol when no wastewater treatment plant is available (no sludge production) or to get a first indication on the plant performance and biodegradability.

In itself, the influent characterization protocol presented here is a valuable (stand-alone) tool for operational process assessments. These guidelines for wastewater characterization are based on a physical-chemical method thereby identifying the sum of the soluble COD fractions in the influent and also determining the biodegradable fraction of the influent COD. Thus, two important wastewater characteristics have already been determined that will give valuable information for a design study, also without application of models, about the possibilities and expectations of an upgrade or extension of a treatment process. For example, the total biodegradable COD indicates the potential of a treatment plant for a large quantity of nitrogen and biological phosphorus removal. The particulate COD provides information about the expected sludge production. By performing a more detailed characterization (following the protocol), detailed information can be obtained for simulation (design) studies for process optimization, trouble-shooting and design assistance.

7.2 Starting points for the characterization protocol

It should be emphasized that the aim was to develop methods useful for the simulation of full-scale WWTP where reproducibility and simplicity of the methods are important criteria.

Simulation of full-scale WWTP operations is less sensitive than simulation of a controlled laboratory batch experiment. Use of the methods proposed here might therefore work very well for practical purposes but may not be accurate enough when a laboratory experiment is concerned.

The guidelines for wastewater characterization in the Netherlands are based on a physical-chemical method to characterize the sum of the soluble COD fractions S_I and S_S. Without performing a model study, as a stand-alone assessment this can be combined with a BOD analysis (measuring the BOD as a function of time) for determining the biodegradable fraction of the biodegradable influent COD (S_S+X_S). The inert particulate fraction X_I is found as the difference between the total COD (TCOD) and all other COD fractions. The BOD calculation, as also introduced in this guideline report (see Chapter 8), is a model calculation and relies on BOD measurements and not on specified BOD model parameters. The calculation method therefore is not fully reliable. In contrast to the recommendations in the original influent characterization report, when the characterization is performed in preparation for a model study, it is recommended to retrofit the influent biodegradable COD fraction based on the measured sludge production. From practical experience, this method has proven much more reliable. BOD calculations therefore are only recommended to be used for a first estimation of wastewater biodegradability.

One important assumption in the fractionation of the influent COD is that the biomass fractions in the influent are ignored. It is not strictly necessary to consider a heterotrophic biomass fraction in the influent because heterotrophs in operational practice will never wash out. Model calculations therefore will not rely on the presence of heterotrophic bacteria in the influent and this fraction can be ignored when modeling. Due to a low growth rate, the autotrophs have to be considered in the influent because they can be washed out in high-loaded treatment plants. However, the fraction is very small in the influent and can be ignored in comparison with the total COD. Therefore typically a very low default influent concentration of biomass is used in the influent and model calculations. The decision to ignore influent biomass is supported by the theory that the bacterial diversity in activated sludge is a product of selection by the environment rather than inoculation by the wastewater (Curtis *et al.*, 1998).

7.3 Introducing the protocol

The guidelines presented here relate to ASM type models (for organic COD, nitrogen and phosphorus fractions). The guidelines follow common principles and generally apply to other models (e.g. the BioWin characterization method as presented in the next chapter). All choices and assumptions are based on theoretical considerations and practical experience. The basic ASM nomenclature is described in Table 7.1. Figure 7.1 shows the parameter inter-relations.

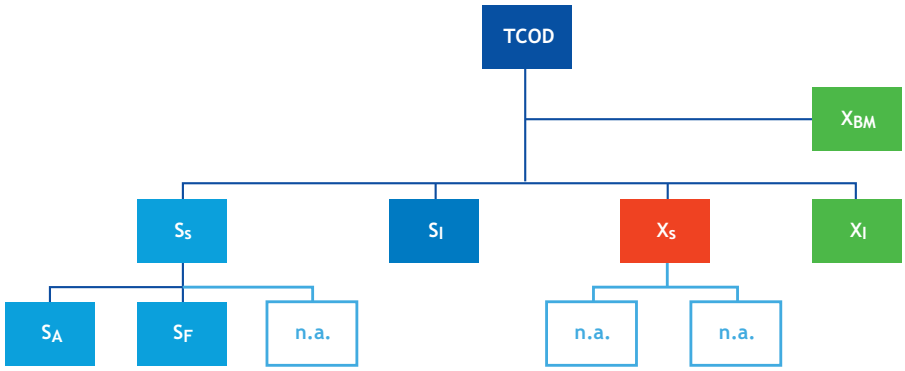

Figure 7.1 Division of total COD (TCOD) into constituent fractions according to the IAWQ notation (e.g. also used in the STOWA method). Blue fractions are soluble and (olive) green fractions particulate. Green-lined boxes are biodegradable and black-lined boxes are inert influent fractions. Yellow text indicates readily biodegradable and green text slowly biodegradable. The grey boxes are fractions used in BioWin but they are not defined in the ASM method.

Table 7.1 Influent fractionation adopted in the ASM family

Influent COD, N and P fractions (ASM)			
		Unit	Description
Organic soluble fractions	S_S (S_A+S_F)	mg COD/L	Total soluble readily biodegradable COD
	S_A	mg COD/L	Acetate
	S_F	mg COD/L	Soluble readily fermentable biodegradable COD
	S_I	mg COD/L	Soluble inert COD
Organic particulate fractions	X_S	mg COD/L	Particulate and slowly biodegradable COD
	X_I	mg COD/L	Particulate inert COD
	X_H	mg COD/L	Particulate biomass, ordinary heterotrophic
	X_{PAO}	mg COD/L	Particulate biomass, heterotrophic (bio-P, PAO)
	X_A	mg COD/L	Particulate biomass, autotrophic (nitrifying)
	X_{BM}	mg COD/L	General biomass indication (COD)
Nitrogen soluble fractions	S_{NH4}	mg N/L	Ammonia
	S_{NO3}	mg N/L	Nitrate
	iN_{SA} S_A	mg N/L	Nitrogen related to soluble organic substrate S_A
	iN_{SF} S_F	mg N/L	Nitrogen related to soluble organic substrate S_F
	iN_{SI} S_I	mg N/L	Nitrogen related to soluble inert substrate S_I
Nitrogen particulate fractions	iN_{XS} X_S	mg N/L	Nitrogen related to particulate organic substrate X_S
	iN_{XI} X_I	mg N/L	Nitrogen related to particulate inert substrate X_I
	iN_{XBM} X_{BM}	mg N/L	Nitrogen related to biomass
Phosphorus soluble fractions	S_{PO4}	mg P/L	Ortho-phosphate
	iP_{SA} S_A	mg P/L	Phosphorus related to soluble organic substrate S_A
	iP_{SF} S_F	mg P/L	Phosphorus related to soluble organic substrate S_F
	iP_{SI} S_I	mg P/L	Phosphorus related to soluble inert substrate S_I
Phosphorus particulate fractions	iP_{XS} X_S	mg P/L	Phosphorus related to particulate organic substrate X_S
	iP_{XI} X_I	mg P/L	Phosphorus related to particulate inert substrate X_I
	iP_{XBM} X_{BM}	mg P/L	Phosphorus related to biomass

7.3.1 Calculating and measuring the organic matter fractions

Total COD, a standard influent measurement, relates to the different organic fractions as given below:

$$TCOD = S_A + S_F + S_I + X_S + X_I + X_H + X_{AUT} + X_{PAO} = TCOD_{INF} \quad (7.1)$$

The biomass components in the influent can typically be ignored.

$$X_{BM} = X_H + X_{AUT} + X_{PAO} \approx 0 \quad (7.2)$$

and therefore the total COD is given by:

$$TCOD = S_A + S_F + S_I + X_S + X_I = TCOD_{INF} \quad (7.3)$$

Influent fractions are separated based on filtration between soluble and particulate components where soluble fractions are indicated by S and particulate fractions by X. The soluble COD fraction is measured by (micro-) filtration and given by:

$$COD_S = S_A + S_F + S_I = COD_{MF,INF} \quad (7.4)$$

The ASM 1 model does not distinguish between acetate and soluble readily fermentable biodegradable COD (S_F). In ASM 1 soluble substrate is presented as S_S:

$$S_{SS} = S_A + S_F \quad (7.5)$$

The particulate COD fraction is calculated from the measured total COD in the influent and the filtrate COD. Particulate COD is also indicated as COD_{VSS}, given by:

$$COD_X = X_S + X_I = TCOD_{INF} - COD_{MF,INF} \quad (7.6)$$

The biodegradable COD (COD_{BD}) can be directly measured by respirometry and is given by:

$$COD_{BD} = X_S + S_S \quad (7.7)$$

It should be noted that the biodegradable COD is not equal to the BOD_5 measurement. Typically BOD_5 only represents 50-90% of the biodegradable COD. Also, the BOD measured after 20 days (BOD_{20}) does not accurately represent the biodegradable COD. The main reason for this is that the BOD measured after 20 days is considered as a very inaccurate measurement with results that are difficult to reproduce. To overcome this problem, in the original protocol it is recommended to use the BOD model for estimation of the infinite BOD (BOD_{tot}). This method is explained in Figure 7.2. However, based on more practical experience, the protocol presented here recommends that this method is used as a first guess regarding influent biodegradability. When an existing wastewater treatment plant is to be modelled (retrofitted) the measured sludge production can be used to calculate the influent BOD_{tot} or biodegradable COD fraction backwards. In the proposed method, the WWTP itself is used as the final BOD test to determine the fraction influent COD that is degraded and the fraction that is recovered in the waste sludge production. This approach has been shown to be more accurate than determining biodegradability based on the BOD model.

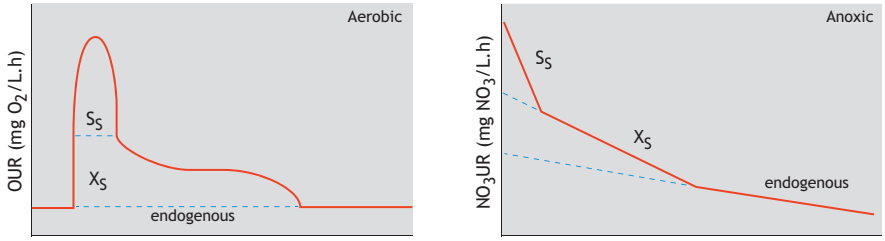

Figure 7.2 Direct measurements of biodegradability based on aerobic and anoxic respiration tests. Readily biodegradable substrate will appear as the first peak of oxygen consumption (or nitrate consumption) followed by a shoulder representing slowly biodegradable substrate. Finally, a baseline is measured representing endogenous respiration.

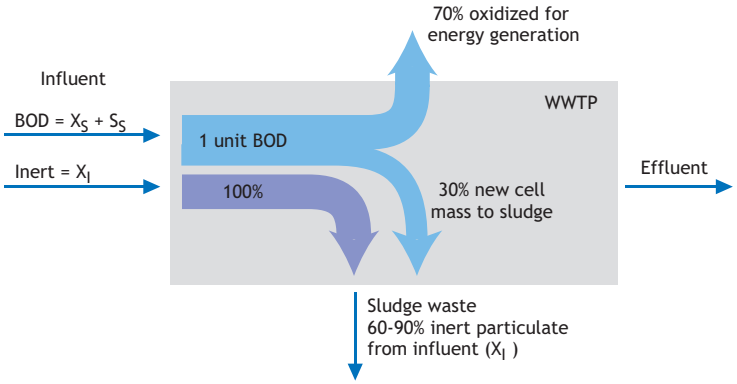

Figure 7.3 Practical determination of influent biodegradability based on the use of the activated sludge model (without using BOD tests); if the sludge production is measured in practice, in the model the ratio X_I/X_S in the influent can be adjusted to fit the model to the measured sludge production. Therefore the sludge production needs to be determined with a high level of accuracy. The BOD model can be used as an indicative starting point.

The inert soluble COD fraction X_I for a wastewater treatment systems operated at a SRT longer than 3 days can be reliably estimated from a filtered effluent COD measurement. All inert soluble COD entering the wastewater treatment plant via influent will leave the plant via effluent. However, some soluble inert COD is also formed during the wastewater treatment by degradation of microbial cell material. To correct for this additional formation, an estimated factor of 90% (0.9) is applied to the effluent measurement to predict the influent inert soluble COD according to:

$$S_I = COD_{MF,EFF} \times 0.9 \qquad (7.8)$$

For wastewater treatment systems which are highly loaded (a typical SRT in the range of 3 days or less) some organic matter (BOD) is also expected to be present in the effluent. For BOD that will also appear in the filtered COD measurement, in the case effluent measurements are used to estimate influent inert soluble COD, a correction factor has to be included for the possible occurrence of effluent BOD according to:

$$S_I = COD_{MF,EFF} \times 0.9 - 1.5\ BOD_{5,EFF} \quad (7.9)$$

Practical experience has shown that the influent and effluent soluble inert COD for all wastewater treatment plants is more or less in the range of 20 to 35 mg/L and can be considered constant.

7.3.2 Calculating and measuring the nitrogen fractions

In the activated sludge models, the conversion of nitrogen is attributed to nitrification and denitrification processes. Therefore, ammonia and nitrate are modelled fractions. The growth of biomass is also modelled and thereby nitrogen is used as a nutrient and incorporated into the organic material. The total nitrogen mass balance is given by:

$$TN_{INF} = TKN_{INF} + \text{N-NO}_{3,\text{INF}} \quad (7.10)$$

Often nitrate in the influent is zero. However, when oxygen is introduced in the sewage, nitrification can occur and some nitrate can be present. Also, when nitrate containing ground or surface water has infiltrated in the sewage system, a considerable amount of nitrate can be measured in the influent. Both nitrate and ammonia are model parameters and directly quantifiable from measurements given by:

$$S_{NO3} = \text{NO}_3\text{-N mg N/L} \quad (7.11)$$

$$S_{NH4} = \text{NH}_4\text{-N mg N/L} \quad (7.12)$$

In the model TKN is expressed as the sum of all particulate and soluble organic nitrogen and inorganic soluble nitrogen (ammonia) according to:

$$TKN = S_{NH4} + (iN_{SA} \cdot S_A) + (iN_{SF} \cdot S_f) + (iN_{SI} \cdot S_I) + (iN_{XS} \cdot X_S) + (iN_{XI} \cdot X_I) \quad (7.13)$$

Both volatile fatty acids (VFA) being mainly acetate $CH_3\text{-}COO^-$ (typically 60%) and other VFA which typically cause sewage odour can be found with decreasing quantity; propionate $C_2H_5\text{-}COO^-$ (25%), butyrate $CH_3CH_2CH_2\text{-}COO^-$ (10%), n-valeric acid $C_4H_9\text{-}COO^-$ (5%), and valeric acid $C_5H_{10}O_2$ (<5%)) are organic compounds which do not contain nitrogen and therefore:

$$iN_{SA} = 0 \quad (7.14)$$

$$iN_{SA} \cdot S_A = 0 \quad (7.15)$$

Inert soluble COD are mostly humic acid, a principal component of humic substances, which are the major organic constituents of soil (humus), peat, coal, many upland streams, dystrophic lakes, and ocean water. It is produced by the biodegradation of dead organic matter. It is not a single acid; rather, it is a complex mixture of many different acids (Figure 7.4). Humic acid is often the reason for commonly observed (however harmless) brownish discoloration of the effluent (coming from ground and surface waters). However, humic acid typically contains only a small amount of nitrogen, so the amount can often be ignored in practice (< 0.01 g N/g COD):

$$iN_{SI} \approx 0 \quad (7.16)$$

$$iN_{SI} \cdot S_I \approx 0 \quad (7.17)$$

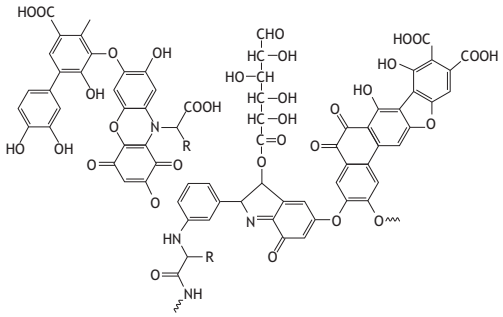

Figure 7.4 Example of the formulation of humic acid (Stevenson, 1994)

TKN can therefore be expressed in the absence of nitrogen in the VFA fraction and soluble inert COD fraction given by:

$$TKN = S_{NH4} + (iN_{SF} \cdot S_f) + (iN_{XS} \cdot X_S) + (iN_{XI} \cdot X_I) \quad (7.18)$$

Soluble TKN is determined by (micro-) filtration according to:

$$TKN_S = S_{NH4} + \left(iN_{SF} \cdot S_f\right) = TKN_{MF,INF} \quad (7.19)$$

The particulate TKN is calculated from the total influent TKN measurement and the micro filtrate in influent measurements according to:

$$TKN_X = (iN_{XS} \cdot X_S) + (iN_{XI} \cdot X_I) = TKN_{INF} - TKN_{MF,INF} \quad (7.20)$$

From this definition, particulate TKN is all organic and related to the volatile suspended solids (or VSS). From practical experience it is observed that the composition of particulate substrate (X_S) and particulate inert COD (X_I) can be very different depending on the origin of the sewage. X_I is mostly related to cellulose, an organic compound with the formula $(C_6H_{10}O_5)_n$ being polysaccharide and consisting of a linear chain made up of between several hundred and ten thousand linked D-glucose units. Cellulose is the structural component of the primary cell wall (of green plants, and many forms of algae and bacteria) and thereby the most common organic compound on earth. About 33% of all plant matter is cellulose (the cellulose content of cotton is 90% and that of wood is 40–50%). Cellulose is difficult to degrade and will typically not degrade in an activated sludge plant and therefore belongs to the X_I fraction in sewage.

Cellulose does not contain nitrogen and therefore also fibre-rich material (X_I) typically contains less nitrogen than the biodegradable particulate influent fraction (X_S), and also includes proteins (the generic formula of amino acids is $H_2NCH(R)$-COOH where R is the organic substituent often also containing phosphorus). The nitrogen content of inert particulate COD is estimated at 3% per gCOD while particulate substrate is estimated at 4%. The soluble organic nitrogen fraction is calculated by subtracting the filtered influent and ammonia measurement and is given by:

$$Organic, TKN_S = iN_{SF} \cdot S_f = TKN_S - S_{NH4} = TKN_{MF,INF} - \text{NH}_4\text{-N} \quad (7.21)$$

The total organic nitrogen fraction is calculated by subtracting TKN and ammonia and is given by:

$$Organic, TKN = \left(iN_{SF} \cdot S_f\right) + \left(iN_{XS} \cdot X_S\right) + \left(iN_{XI} \cdot X_I\right) = TKN_{INF} - \text{NH}_4\text{-N} \quad (7.22)$$

In the influent fraction calculations it should be realized that organic substances contain nitrogen, and by introducing more organics into the influent, the nitrogen load will also increase.

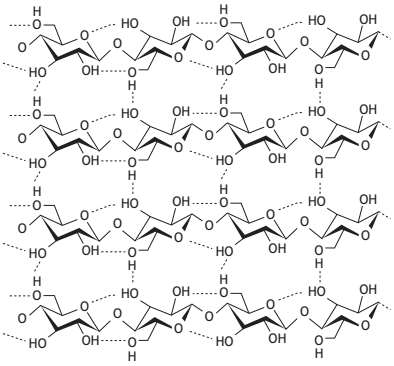

Figure 7.5 Example of the formulation of cellulose (figure by L. Laghi)

7.3.3 Calculating and measuring the phosphorus fractions

In the influent characterization, the phosphorus fractions are calculated in a similar way to the nitrogen fractions. In the activated sludge models, phosphorus is not converted; instead it is accumulated in the biomass by the growth of bacteria (a nutrient for growth). In the biological phosphorus removal process, Phosphorus Accumulating Organisms (PAO) store phosphate as poly-phosphate (PP). This is indicated as biologically enhanced P-uptake, and this PP fraction can be released by the bacteria without cell decay.

Ortho-phosphate (PO_4) and poly-phosphate are modelled fractions. The growth of biomass is also modelled and thereby organic phosphate is used as a nutrient. The total phosphorus mass balance is given by:

$$TP_{INF} = PO_4 + Organic, TP + PP \quad (7.23)$$

Because there will be no phosphorus-accumulating bacteria in the influent, there will also not be poly-phosphate in the influent and therefore:

$$TP_{INF} = PO_4 + Organic, TP \quad (7.24)$$

Ortho-phosphate is a model parameter and is direct observable from measurements, and is given by:

$$S_{PO4} = \text{PO}_4\text{-P mg P/L} \quad (7.25)$$

In the model, TP is expressed as the sum of all particulate and soluble organic phosphorus and inorganic soluble phosphorus (ortho-phosphate) according to:

$$TP = S_{PO4} + (iP_{SA} \cdot S_A) + (iP_{SF} \cdot S_f) + (iP_{SI} \cdot S_I) + (iP_{XS} \cdot X_S) + (iP_{XI} \cdot X_I) \quad (7.26)$$

Acetate (CH_3-COOH) and other VFAs are organic but do not contain phosphorus and therefore:

$$iP_{SA} = 0 \quad (7.27)$$

$$iP_{SA} \cdot S_A = 0 \quad (7.28)$$

Inert soluble COD is mostly humic acid. The structure formula in Figure 7.4 shows that humic acid does not contain phosphorus and therefore the phosphorus content of S_I can often be ignored;

$$iN_{SI} \approx 0 \quad (7.29)$$

$$iN_{SI} \cdot S_I \approx 0 \quad (7.30)$$

In the absence of phosphorus in VFA and soluble inert COD, TP can be expressed as:

$$TP = S_{PO4} + (iP_{SF} \cdot S_F) + (iP_{XS} \cdot X_S) + (iP_{XI} \cdot X_I) \quad (7.31)$$

Soluble TP is determined by (micro-) filtration according to:

$$TP_S = S_{PO4} + (iP_{SF} \cdot S_F) = TP_{MF,INF} \quad (7.32)$$

The particulate TP is calculated from the total influent TP measurement and the micro filtrate in influent measurements according to:

$$TP_X = (iP_{XS} \cdot X_S) + (iP_{XI} \cdot X_I) = TP_{INF} - TP_{MF,INF} \quad (7.33)$$

From this definition, particulate TP is all organic and related to the volatile suspended solids (or VSS). Practical experience has shown that the composition of particulate substrate (X_S) and particulate inert COD (X_I) can be very different depending on the origin of the sewage. Typically, the phosphorus content of organic particulate substrate is higher than the nitrogen content of inert particulate COD. However, because the fraction of phosphorus in organic material is very small, estimated in the range of <1 per gCOD, it is assumed that both fractions are similar (1%). The soluble organic nitrogen fraction in theory can be calculated by subtracting TP from the filtered influent and measured ortho-phosphate. However, in practice the calculated soluble organic TP fraction will be very small and therefore also inaccurate. The calculation is given by:

$$Organic, TP_S = iP_{SF} \cdot S_f = TP_S - S_{PO4} = TP_{MF,INF} - PO_4\text{-P} \quad (7.34)$$

The total organic phosphorus fraction is calculated by subtracting influent TP and ortho-phosphate and is given by:

$$Organic, TP = (iP_{SF} \cdot S_F) + (iP_{XS} \cdot X_S) + (iP_{XI} \cdot X_I) = TP_{INF} - PO_4\text{-P} \quad (7.35)$$

In the influent fraction calculations it should be realized that organics contain phosphorus, and by introducing more organic material in the influent then the load of phosphorus also increases.

Also, it should be realized that in general the phosphorus content of organic material is low and that all calculations related to this fraction are inaccurate and only indicative.

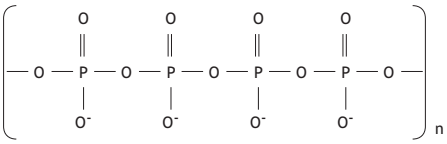

Figure 7.6 Formulation of linear poly-phosphate $(PP)_n$ used by PAO as a cell internal (energy) storage polymer. Typically, no PP is found in the influent, only inside PAO in the activated sludge.

7.4 Physical-chemical characterization

In the protocol presented here for the determination of the readily biodegradable COD (S_S) no biological methods were selected. They were evaluated as more complex in use than physical-chemical methods, and moreover different studies have shown a close similarity between S_S concentrations from a physical-chemical (0.1 µm or after flocculation) and a biological characterization (Torrijos *et al.*, 1994; Mamais *et al.*, 1993; STOWA, 1997).

The protocol in The Netherlands was first presented in 1996 with the recommendation to use a 0.45 µm pore size filter. However, a part of the particulate (colloidal) fraction passes through the filter resulting in an overestimation of the fraction soluble COD. Nevertheless, reasons for the selection of a 0.45 µm filter were the expected clogging problems with 0.1 µm filters and the supposed labour intensiveness of a flocculation step.

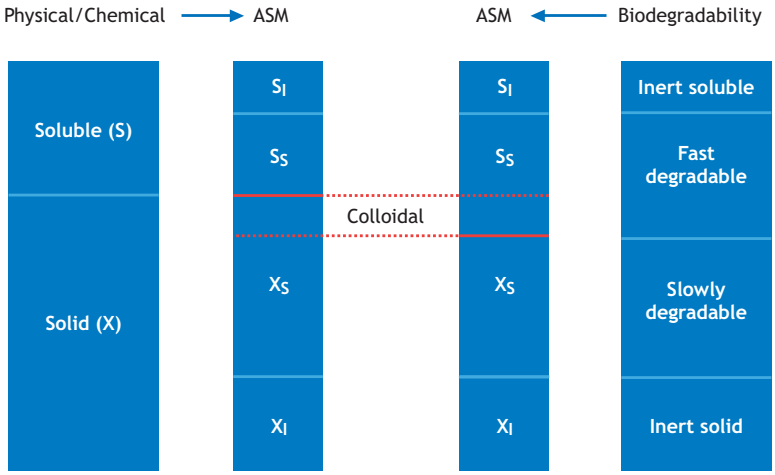

Figure 7.7 Comparing the characterization of organic fractions based on physical-chemical separation (filtration) and biodegradability tests. The uncertainty in the filtration lies in the biodegradability of the colloidal material. Biological tests will typically give a more accurate result; however, for modeling purposes practical experience has shown that in most modeling studies the physical chemical method may be applied.

In further research into the pre-treatment of wastewater (STOWA, 1999), a wastewater characterization was carried out in which the S_S fraction was based on a 0.45 µm filtration. With pre-precipitation at full-scale, this fraction S_S was removed at an average rate of 29% (study on six WWTPs). Based on the significance of the measured differences, the protocol was modified (Roeleveld and Kruit, 1998) and it was recommended not to use a 0.45 µm filter anymore for the determination of the soluble COD fraction, but instead the previously mentioned 0.1 µm filter or flocculation with $Zn(OH)_2$ or another precipitant. An analytical program conducted at seven WWTPs (STOWA, 2001) showed that the difference in COD after 0.1 µm filtration or flocculation is rather small (approximately 1% difference). This means that there is a degree of freedom in the choice between these two options.

Practical experience following the first publications of these guidelines showed that flocculation is relative easy to perform and gives reliable results in combination with a 0.45 µm filter step. In the guidelines presented here, it is recommended to use the flocculation method.

Since both biodegradable and inert COD pass through the filter, the inert fraction S_I has to be determined independently and subtracted from the soluble COD to give the fraction S_S. In the guidelines S_I is evaluated from the inert COD in the effluent of the examined treatment plant. When modeling biological phosphorus removal with ASM2d, the fraction S_F can be found if the volatile fatty acids concentration (S_A) has been measured, e.g. with gas chromatography or titration.

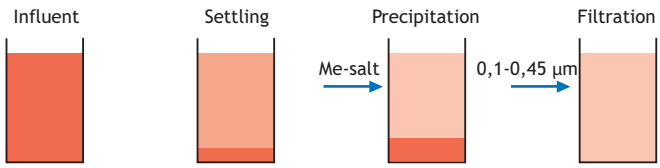

Figure 7.8 Physical-chemical methodology for separating influent by particle size. Flocculation in combination with membrane filtration is in practice easy to perform and delivers accurate reproducible results.

7.5 Biological characterization

The biodegradable COD (BCOD or COD_{BD}) in the influent is the sum of the readily biodegradable soluble COD (S_S) and the slowly biodegradable particulate COD (X_S). In the guidelines the fraction COD_{BD} is determined from a BOD analysis where the BOD is measured as a function of time. The selection of the BOD analysis was made because it is still a widely used parameter in the practice of wastewater treatment. In this way the guidelines can easily be implemented in the routine analysis program of a WWTP.

BOD_5 does not represent the total biodegradable fraction of the COD. Depending on the type of wastewater, 50 to 95% of the COD is oxidized after 5 days. After 20 days 95 to 99% of the COD is oxidized but the measurement of the BOD_{20} is not reliable and therefore not recommended. A better approach is given by following the BOD course as a function of time and calculating the TBOD (BOD_{tot}) of the wastewater. Figure 7.9 shows a BOD course of municipal wastewater where the BOD is measured in duplicate on days 1, 2, 4, 6, and 8. This can be done easily using automated BOD oxygen caps. The TBOD is calculated by the equation given in Figure 7.9.

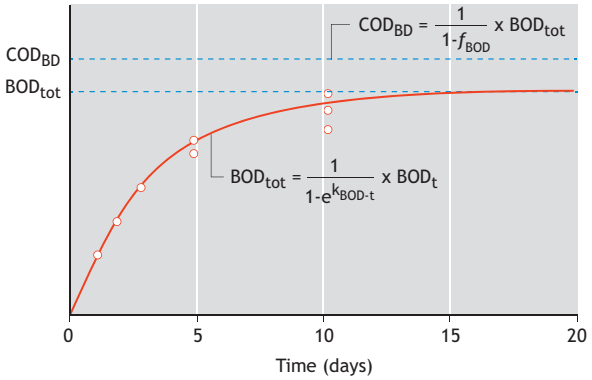

Figure 7.9 Estimation of the biodegradable COD by BOD measurements (at 1, 2, 3, 5 and 10 days in triplicate) and curve fitting k_{BOD}.

The constant k_{BOD} is the first order rate constant of the BOD versus time measurement. For municipal wastewater, k_{BOD} values are found in The Netherlands between 0.15 and 0.8 d^{-1}. This implies no standards can be given and measurements are necessary for each plant. The k_{BOD} can be determined by fitting the BOD curve through the measured data with the TBOD equation, whereby the values for k_{BOD} and TBOD are optimized by linear regression with the least squares method. This method can be easily implemented in an electronic spreadsheet. Although a non-linear method will produce smaller errors, the linear method gives comparable k_{BOD} values (Cutrera *et al.*, 1999).



Figure 7.10 Automated unit for multiple BOD measurements. The equipment is commercially available from different manufacturers and is used in most WWTP labs for BOD measurements. These tests can be used for determining the factor k_{BOD} and influent total BOD (COD_{BD}).

The constant k_{BOD} depends on the division between S_S and X_S, which is influenced by the type and length of the sewer system, the fraction of industrial wastewater and the application of pre-treatment. When the k_{BOD} value is determined for a particular wastewater, the biodegradable COD (COD_{BD}) can be determined from the BOD_5, which is usually measured in routine analysis programs. Based on knowledge of the dynamic behaviour of the influent composition, a verification of the k_{BOD} value should sometimes be carried out. During the BOD measurement there is an interaction of growth and decay of biomass. For a long-term BOD measurement this results in the conversion of a part of the biodegradable COD into an inert fraction. Therefore, the initial concentration of COD_{BD} will be higher than the determined TBOD and a correction factor (f_{BOD}) has to be used. For the factor f_{BOD} a value of 0.15 (0.1–0.2) can be taken, which is in close accordance with the fraction in ASM for inert COD generated in biomass lysis. The fraction COD_{BD} has to be calculated as presented in Figure 7.9. However, in practice this fraction is shown to be more or less constant.

For the determination a measurement device can be used which measures the BOD curve online. Otherwise the BOD can be measured at a minimum of 5 measurement points from day 1 until 10, with a special focus on the first few days. The BOD measurement has to be carried out with a non-filtered sample and with the addition of ATU to inhibit nitrification. All BOD measurements are referred to as carbonaceous BOD. In addition it should be mentioned that the most representative characterization will be obtained when effluent from the simulated treatment plant is used as inoculum for the BOD measurements.

With the methodology presented for the characterization of the organic fractions S_I, S_S and X_S, the fraction X_I is found to be the difference between the total COD and the other fractions. By using the BOD measurement, the heterotrophic biomass fraction will be included in the fractions X_S and X_I and this is acceptable according to the description of the activated sludge process in ASM.

There are some practical warnings when applying BOD measurements for the determination of influent biodegradability:

- COD-measurements
  - Use the dichromate method for accurate estimates.
- Filtration
  - Do not use filters made of cellulose acetate or other filters which can release significant quantities of COD;
  - Always wash the filters with demi-water before use;
  - Use pressure filtration in preference to vacuum filtration in order to limit the stripping of VFA.
- Flocculation
  - Be aware that an excessively high dosage of certain precipitants can influence the COD (e.g. with $FeCl_3$).
- BOD_5 measurement
  - Use effluent from the examined WWTP as inoculum in order to produce the most representative results;
  - Use ATU in a concentration of 5 mg ATU/L in the BOD-flask;
  - Analyse at least in duplicate, and even better in triplicate;
  - Be aware that S_2^- in the wastewater has an oxygen demand.

- k_{BOD} measurement
  - See the comments on the BOD_5 measurement;
  - Pay attention to the correct dilution factor; a BOD_{10} cannot be treated in the same way as a BOD_5.
- General comments
  - Keep the time between sampling and analysis as short as possible; at 4°C there is also biological activity;
  - Be aware of the fact that accurate wastewater characterization is essential for the reliability of all preceding model predictions;
  - Give clear instructions to the laboratory because long-term BOD tests are not standard.

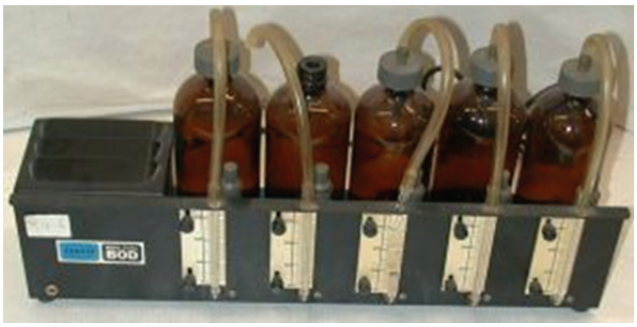

Figure 7.11 Traditional BOD measurement equipment. During the initial development of the influent characterization guidelines the accuracy of long-term BOD tests (>5 days) was problematic. With new equipment, the reproducibility has improved but, however, it is not accurate enough to predict (retrofit) the sludge production of an existing WWTP.

7.6 Discussion on nitrogen and phosphorus fractions

For modeling with ASM2 it is agreed (Henze *et al.*, 1995) that in general there is no need to characterize the nitrogen fractions in as much detail as for organic matter. The reason for this is that the major part of the nitrogen is present as ammonia, which has no coupling to the organic components. For the organic part of the nitrogen, fixed nitrogen fractions (i_N) are used for the various COD components. This has shown an over-simplification of the ASM models ASM1 and ASM2d. The nitrogen characterization should appropriately predict the nitrogen content of the sludge in the activated sludge system and at the same time the effluent concentrations. The amount of nitrogen available for oxidation determines the autotrophic oxygen demand and the required denitrification capacity. Meijer (2001) proved the importance of an accurate nitrogen balance by modeling a full-scale activated sludge system; the fraction of N in X_S and X_I had to be adjusted significantly to fit the nitrogen content of the sludge. For convenience the nitrogen composition of the different COD fractions in the influent are taken to be the same as those for the activated sludge itself. This over-simplification was never introduced in the BioWin model, which uses separate nutrient fractions for the influent and effluent thereby (in theory) giving more accurate results then the ASM models. However, this requires some additional influent measurements to establish these separate fractions. Therefore the balance between accuracy and a practical approach remains crucial. When very strict effluent criteria for nitrogen are required, it becomes necessary to know what the fraction of inert soluble nitrogen is (fraction N

related to S*I* is expressed as i_{NSI}). Elevated values in the effluent might be a result of industrial discharges or high strength waste.

The nitrogen fractions for the various COD components can be found by fitting the model, e.g. on the measured nitrogen content of the sludge (TKN in the VSS). The proposed conversion factors of ASM2d can be used as initial values; however, they will not lead to a reliable estimation of the activated sludge composition. When more clarity is needed, the conversion factors can be determined by analysis of NH_4-N and TKN from filtered and non-filtered influent and effluent samples. With both methods it is important to keep the total nitrogen level in the influent constant (with every change in the model influent fractions, the mass balance has to be maintained). Table 7.2 gives the default values of nitrogen fractions for the various COD components, together with some practical equations. For these guidelines, attention must be paid to the fraction of N in X_S and X_I, because the heterotrophic biomass fraction is ignored in them.

Table 7.2 Conversion factors for nitrogen and typical ranges

Symbol	Unit	Typical range	Default
iN_{SI}	gN/gCOD	0.01-0.02	0.01
iN_{SA}	gN/gCOD	0	0
iN_{SF}	gN/gCOD	0.02-0.04	0.03
iN_{XI}	gN/gCOD	0.01-0.06	0.03
iN_{XS}	gN/gCOD	0.02-0.06	0.04

Concerning the characterization of phosphorus for ASM2d, the same goes as for the characterization of nitrogen. However, based on research in modeling of biological phosphorus removal (Meijer, 2001), it is concluded that the default values of the conversion factors for P of ASM2d can be used as a simplification. Table 7.3 presents the conversion factors for phosphorus and the typical ranges. For municipal wastewater the soluble phosphorus S_{PO4} can be approximated by the soluble inorganic phosphorus concentration.

Table 7.3 Conversion factors for phosphorus and typical ranges

Symbol	Unit	Typical range	Default
iP_{SI}	gP/gCOD	0.002-0.008	0
iP_{SA}	gP/gCOD	0	0
iP_{SF}	gP/gCOD	0.01-0.015	0.01
iP_{XI}	gP/gCOD	0.005-0.01	0.01
iP_{XS}	gP/gCOD	0.01-0.015	0.01

7.7 Examples of influent fractions from the Netherlands

Tables 7.4 and 7.5 show results from the study of Roeleveld *et al.* (2002). The COD fractions of 21 treatment plants were measured, obtained with the guidelines presented here. All wastewater characterizations concern the wastewater after the screen, before any kind of pre-treatment and before mixing with internal flows.

In the performance of the characterization, different methods were used to determine the soluble COD fraction: filtration (0.45 μm) of a pre-treated sample with $Zn(OH)_2$, only filtration (0.1 or 0.45 μm) or only flocculation. The samples were taken as much as possible on days with

dry weather conditions. When rainfall had a significant effect on the wastewater composition, these values were not used for the calculation of the given average values.

Table 7.4 Examples of influent WWTP measurements of raw sewage

WWTP	Measurements [mg/L]				BOD model		Model fractions [mg/L]			
	TCOD	COD_S	$COD_{S,EFF}$	BOD_5	kBOD	COD_{BD}	S_I	S_S	X_S	X_I
Geestmerambacht[1)]	593	134	32	189	0.35	269	29	105	164	295
Niedorpen[1)]	393	150	39	199	0.42	267	35	115	152	91
Stolpen[1)]	827	207	41	373	0.35	531	37	170	361	259
Ursem[1)]	748	238	40	330	0.38	457	36	202	255	255
Wieringermeer	592	140	40	227	0.42	304	36	104	200	252
Wieringen[1)]	668	240	41	330	0.38	457	37	203	253	175
Deventer[2)]	799	125	31	257	0.34	370	28	97	273	401
Nijmegen[2)]	536	154	28	238	0.35	339	25	129	210	172
Franeker[2)]	607	113	53	251	0.36	354	48	65	288	206
Gouda[2)]	662	180	34	254	0.38	351	31	149	202	280
Venlo[3)]	617	222	25	220	0.47	286	23	204	83	308
Apeldoorn[4)]	468	195	33	173	0.70	210	30	165	45	228
Boxtel[4)]	510	172	36	181	0.41	244	32	140	105	233
Nieuwgraaf[4)]	613	297	43	277	0.59	344	39	258	86	230
Groote Lucht[4)]	447	158	32	174	0.43	232	29	129	102	187
Nieuwe Waterweg[4)]	430	113	30	148	0.46	194	27	86	108	209
Haarlem[4)]	328	118	23	120	0.39	165	21	97	67	143
Alphen[5)]	512	145	30	176	0.33	256	27	118	138	229
Groesbeek[5)]	428	180	30	180	0.34	259	27	153	106	142
Hardenberg[5)]	555	219	38	217	0.40	295	34	185	110	226
Papendrecht[5)]	241	127	28	119	0.60	147	25	102	46	68
Statistics										
Average	551	173	35	221	0.42	301	31	142	160	219
Standard deviation	148	50	7.0	67	0.10	98	6.3	49	88	75
Relative SD	0.27	0.29	0.20	0.31	0.23	0.32	0.20	0.34	0.55	0.34
Minimum value	241	113	23	119	0.33	147	21	65	45	68
Maximum value	827	297	53	373	0.70	531	48	258	361	401

1), 2), 3), 4) and 5) - see footnote Table 7.5

In Table 7.4 ratios are calculated for the wastewater parameters in relation to the total COD in the influent. Except for the fraction S_I, which does not vary a lot between the examined WWTPs, a big variation is noticed in the other COD fractions (S_S, X_S and X_I). These variations are more or less determined by the input to the sewer system (e.g. the contribution by industry), the sewer system type and length (separate/combined) and the transformation processes in the sewer (pressure main/gravitational).

In Table 7.5 the ratios of the biodegradable part of the COD (COD_{BD}) and the particulate COD to the influent TCOD are presented. Also without using activated sludge modeling, these two characteristics give a lot of additional information about the possibilities of a treatment process (e.g. denitrification potential and sludge production); definitely more than the traditionally used parameters TCOD, BOD_5 and TSS.

Table 7.5. Example of influent WWTP measurements of raw sewage relative to TCOD

WWTP	Measurements [mg/L]				BOD model		Model fractions [mg/L]			
	TCOD	COD_S	$COD_{S,EFF}$	BOD_S	kBOD	COD_{BD}	S_I	S_S	X_S	X_I
Geestmerambacht[1)]	1.00	0.23	0.05	0.32	-	0.45	0.05	0.18	0.28	0.50
Niedorpen[1)]	1.00	0.38	0.10	0.51	-	0.68	0.09	0.29	0.39	0.23
Stolpen[1)]	1.00	0.25	0.05	0.45	-	0.64	0.04	0.21	0.44	0.31
Ursem[1)]	1.00	0.32	0.05	0.44	-	0.61	0.05	0.27	0.34	0.34
Wieringermeer	1.00	0.24	0.07	0.38	-	0.51	0.06	0.18	0.34	0.43
Wieringen[1)]	1.00	0.36	0.06	0.49	-	0.68	0.06	0.30	0.38	0.26
Deventer[2)]	1.00	0.16	0.04	0.32	-	0.46	0.04	0.12	0.34	0.50
Nijmegen[2)]	1.00	0.29	0.05	0.44	-	0.63	0.05	0.24	0.39	0.32
Franeker[2)]	1.00	0.19	0.09	0.41	-	0.58	0.08	0.11	0.47	0.34
Gouda[2)]	1.00	0.27	0.05	0.38	-	0.53	0.05	0.23	0.31	0.42
Venlo[3)]	1.00	0.36	0.04	0.36	-	0.46	0.04	0.33	0.13	0.50
Apeldoorn[4)]	1.00	0.42	0.07	0.37	-	0.45	0.06	0.35	0.10	0.49
Boxtel[4)]	1.00	0.34	0.07	0.35	-	0.48	0.06	0.27	0.21	0.46
Nieuwgraaf[4)]	1.00	0.48	0.07	0.45	-	0.56	0.06	0.42	0.14	0.38
Groote Lucht[4)]	1.00	0.35	0.07	0.39	-	0.52	0.06	0.29	0.23	0.42
Nieuwe Waterweg[4)]	1.00	0.26	0.07	0.34	-	0.45	0.06	0.20	0.25	0.49
Haarlem[4)]	1.00	0.36	0.07	0.37	-	0.50	0.06	0.30	0.20	0.44
Alphen[5)]	1.00	0.28	0.06	0.34	-	0.50	0.05	0.23	0.27	0.45
Groesbeek[5)]	1.00	0.42	0.07	0.42	-	0.61	0.06	0.36	0.25	0.33
Hardenberg[5)]	1.00	0.39	0.07	0.39	-	0.53	0.06	0.33	0.20	0.41
Papendrecht[5)]	1.00	0.53	0.12	0.49	-	0.61	0.10	0.42	0.19	0.28
Statistics										
Average	1.00	0.33	0.07	0.40	-	0.55	0.06	0.27	0.28	0.39
Standard deviation	0.00	0.09	0.02	0.06	-	0.08	0.02	0.09	0.10	0.08
Relative SD	0.00	0.29	0.28	0.14	-	0.14	0.27	0.32	0.37	0.21
Minimum value	1.00	0.16	0.04	0.32	-	0.45	0.04	0.11	0.10	0.23
Maximum value	827	297	53	373	0.70	531	48	258	361	401

1) Average values of 24-hour composite samples from five days; soluble analysis after filtration (0.45 µm) of pre-treated sample with $Zn(OH)_2$;
2) Average values of 24-hour composite samples from seven days; soluble analysis after filtration (0.45 µm) of pre-precipitated wastewater;
3) Grab sample; soluble analysis after filtration (0.1 µm);
4) Average values of six grab samples; soluble analysis after flocculation;
5) Average values of 24-hour composite samples from four two-day measuring campaigns; soluble analysis after filtration (0.45 µm).

7.8 Discussion

In model studies (ASM1) with an activated sludge system with pre-denitrification (STOWA, 1996, 1999), it was found that a variation in S_S/X_S ratio (with constant COD_{BD}) can significantly affect the NO_3-N concentration in effluent. The NH_4-N concentration and sludge production are less affected. This is explained by the speed in which S_S can be utilized by the bacteria and thereby the smaller anoxic volume that will be required for denitrification with higher influent S_S. When the fraction S_S is based on filtration, the choice of the filter pore size is very important. Based on reported disadvantages of the 0.45 µm filter, with overestimations of S_S as a result of also incorporating as much as almost 30% slower biodegradable colloidal material (STOWA, 1999), it is recommended to use a 0.1 µm filter or to use the flocculation method. Practical experience has showed that the flocculation method is a very practical tool for soluble analysis.

The protocol presented here for wastewater characterization follows the following sequence:

1. Determination of S_I based on the inert soluble COD in the effluent of the WWTP;
2. Determination of S_S by subtracting the fraction S_I from the soluble COD in the influent;
3. Determination of X_S by subtracting the fraction S_S from the biodegradable COD (COD_{BD});
4. Determination of X_I with the equation $X_I = TCOD - S_I - S_S - X_S$.

This sequence means that all errors and inaccuracies are reflected in the fraction X_I, a very sensitive parameter for modeling. This supports the rejection of the use of a 0.45 µm filter for the determination of the soluble COD. In the protocol the ratio X_S/X_I ratio is indirectly determined by the measurement of the COD_{BD} via the k_{BOD} value, and a variation in X_S/X_I is very sensitive to almost every modelled process. Although results show that the method of determining the value of COD_{BD} can be justified, this method is almost intrinsically subject to inaccuracy (10 to 20%). The uncertainty of the BOD analysis therefore gives some freedom in the calibration of the fractions X_S and X_I. With mass balances for phosphorus the mass of excess sludge can be checked (Meijer, 2001) and this is a correct way to evaluate the estimation of the ratio X_S/X_I, or even to use this as a calibration parameter. Mass balances for phosphorus (or iron when simultaneous precipitation is used) can be made simply by measuring the concentrations of phosphorus in the influent and effluent, and in the (excess) sludge.

Because the presented guidelines are partly based on a physical-chemical method, it can serve perfectly as wastewater characterization for ASM3 (Gujer *et al.*, 1999). For ASM3 the advice is also given to use a 0.1 µm filter or flocculation instead of a 0.45 µm filter. The fact is that it is likely that the colloidal fraction between 0.1 and 0.45 µm will be flocculated onto the activated sludge, and therefore it will belong to the fraction X_S or X_I.

The inaccuracy with the colloidal material and filtration methods was recognized by the developers of the BioWin model (Dold *et al.*). Their model is based on the early ASM1 models without the later proposed (over-) simplifications. The BioWin model still distinguishes between different influent and activated sludge organic nitrogen and phosphorus fractions that were later removed (not all justifiably) from the ASM1, ASM2d and ASM3 models that followed. In the following chapter the specific BioWin influent fractionation method is presented, showing a strong similarity in methodology, but, however, delivering potentially more accurate model results, especially on effluent nitrogen fractions. On the other hand, it should be noted that the accuracy of a model is determined by a lot more factors than only the influent composition and that accuracy is always a trade-off between practical feasibility and financial and time investments. Because the BioWin methodology is a bit more complicated on the calculus, especially the BOD model calculations, the above STOWA guidelines serve as an excellent practical introduction to the influent characterization. The BioWin characterization can be taken in hand when the BioWin model is used and further questions on the precise model influent inputs arise.

7.9 Conclusions

The protocol presented here for wastewater characterization is based on a physical-chemical method to characterize the sum of the soluble COD fractions, combined with a BOD analysis (measuring the BOD as a function of time) for determining the biodegradable fraction of the influent COD. Practical warnings and recommendations for an adequate wastewater characterization are presented in this chapter.

The use of this protocol at about a hundred Dutch WWTP has showed that a sufficiently detailed and practical wastewater characterization is obtained for simulation studies on municipal wastewater treatment for process optimization, trouble-shooting and design assistance. The standardization of wastewater characterization in The Netherlands has proven to be of great help in introducing activated sludge modeling and making model studies more comparable. The protocol fits well to routine analysis programs of WWTP. This means that they are easy to use in practice and that a sufficiently accurate wastewater characterization can be obtained. When no

model study is to be carried out, valuable information can still be gathered from the protocol for the daily operation of WWTP. The total biodegradable COD shows e.g. the potential of a treatment plant for high nitrogen and biological phosphorus removal, and the particulate COD provides information about the expected sludge production.

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Chapter 8

BioWin influent characterization

For this text the BioWin 3.1 user manual was used and edited by S.C.F. Meijer and D. Brdjanovic

8.1 Influent (BOD and COD) wastewater fractions

The wastewater fraction tab in BioWin shows the parameters in Table 8.1. These parameters are used to calculate the fractional composition of the influent wastewater. Note that these fractions are in terms of COD. Even though the overall wastewater strength is entered in terms of BOD, the BioWin model calculates the strength in terms of COD, and uses these fractions to place values in the state vector. BOD is calculated from the COD using the methodology presented further on in this chapter. In the BOD and COD influent wastewater fractions tab the following fractional values can be specified.

Table 8.1 BioWin typical influent fractions

Name	Description	Typical raw sewage	Typical settled sewage
Fbs	Fraction of total influent COD which is readily biodegradable	0.160	0.270
Fac	Fraction of readily biodegradable COD which is VFAs	0.150	0.150
Fxsp	Fraction of slowly biodegradable influent COD which is particulate	0.750	0.500
Fus	Fraction of total influent COD which is soluble unbiodegradable	0.050	0.080
Fup	Fraction of total influent COD which is particulate unbiodegradable	0.130	0.080
Fna	Fraction of influent TKN which is ammonia	0.660	0.750
Fnox	Fraction of influent biodegradable organic nitrogen which is particulate	0.500	0.250
Fnus	Fraction of influent TKN which is soluble unbiodegradable	0.020	0.020
Fpo4	Fraction of influent TP which is phosphate	0.035	0.035
FupN	The N:COD ratio for the influent particulate unbiodegradable COD	0.500	0.750
FupP	The P:COD ratio for the influent particulate unbiodegradable COD	0.011	0.011
FZbh	Fraction of total influent COD which is non-polyP heterotrophic organisms	0.0001	0.0001
FZaob	Fraction of total influent COD which is ammonia oxidizing organisms	0.0001	0.0001
FZnob	Fraction of total influent COD which is nitrite oxidizing organisms	0.0001	0.0001
FZamob	Fraction of total influent COD which is anaerobic ammonia oxidizing	0.0001	0.0001
FZbp	Fraction of total influent COD which is polyP heterotrophic organisms	0.0001	0.0001
FZbpa	Fraction of total influent COD which is propionic acid acetogen organisms	0.0001	0.0001
FZbam	Fraction of total influent COD which is acetoclastic methanogen organisms	0.0001	0.0001
FZbhm	Fraction of total influent COD which is H_2-utilizing methanogen organisms	0.0001	0.0001
FZbm	Fraction of total influent COD which is anoxic methanol utilizing organisms	0.0001	0.0001

Technical note on influent fractions: Setting fractions to zero may result in steady state solution difficulties. If values for biomass fractions are entered (FZ-fractions) it should be remembered that organisms contain nitrogen and phosphorus.

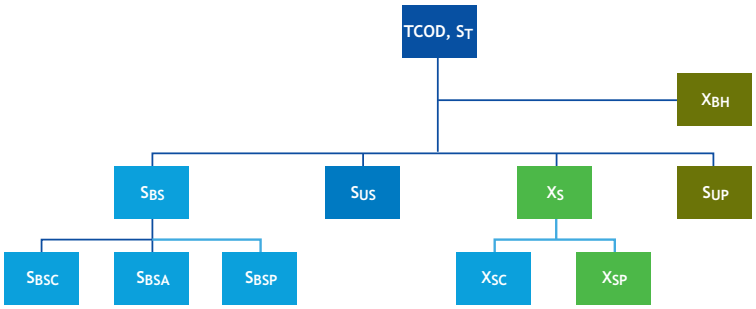

Figure 8.1 Division of BioWin municipal wastewater total COD (TCOD) into constituent fractions. Blue are soluble (S) and green particulate or colloidal particulate (X). Green-lined boxes are biodegradable and black-lined boxes are inert. Yellow text indicates readily biodegradable and green text slowly biodegradable.

Table 8.2 BioWin influent measurements for determining model fractions

Main influent measurements	Notations	Unit	Belišće	Zagreb	Vinkovci	Varaždin	Čakovec
Flow	Q	m^3/d	2,202	331,324	9,760	21,808	8,380
Total COD	TCOD, ST	mgCOD/L	730	419	578	395	495
Total Kjeldahl nitrogen	TKN	mgN/L	62	58	32	34	28
Total P	TP	mgP/L	8.4	4.6	3.3	4.3	3.7
Other influent measurements							
Nitrate-N	NO_3-N, SNO_3	mgN/L	0.0	0.5	0.0	0.6	3.6
pH	pH	-	7.8	7.8	7.8	7.5	7.8
Alkalinity ($CaCO_3$ equivalent)	Alk	$mgCaCO_3/L$	450	450	247	350	340
Calcium	Ca, SCa	mg/L	103	103	92	70	100
Magnesium	Mg, SMg	mg/L	36	36	41	18	19
Dissolved oxygen	DO, (SO_2)	mgO_2/L	0.0	1.2	3.1	1.0	0.0
Additional measurements for modeling							
Effluent filtered COD	CODS_EFF	mgCOD/L	25	16	29	22	26
Influent filtered COD (including colloidal)	CODGF_INF,	mgCOD/L	292	110	210	245	147
Influent FF COD (excluding colloidal)	CODMF_INF	mgCOD/L	204	80	186	220	60
Influent acetate	HAc, VFA, SA	mgCOD/L	0.0	0.0	1.7	0.0	0.0
Influent ammonia	NH4-N, SNH4	mgN/L	31.0	16.8	19.3	10.9	26.2
Influent ortho-phosphate	PO4-P, SPO4	mgP/L	2.5	2.2	2.3	3.0	3.6
Influent carbonaceous BOD_5	TCBOD,TBOD,BOD5	mgO_2/L	281	197	303	155	223
Influent filtered $CBOD_5$	SCBOD,SBOD,BODS	mgO_2/L	190	66	129	150	85
Influent VSS	VSS	mgVSS/L	131	194	272	126	199
Influent TSS	TSS	mgTSS/L	184	244	317	151	227
Derived model fractions							
Fraction unbiodegradable soluble COD	Fus	-	0.03	0.04	0.05	0.06	0.05
Particulate COD non-colloidal	CODp,CODX	mgCOD/L	438	309	368	150	348
Readily biodegradable COD including VFA	Fbs	-	0.25	0.15	0.27	0.50	0.07
Acetate fraction of readily biodegradable COD	Fac	-	0.00	0.00	0.01	0.00	0.00
Ammonia fraction of TKN	Fna	-	0.50	0.29	0.60	0.32	0.93
Phosphate fraction of TP	Fpo4	-	0.30	0.48	0.71	0.71	0.96
TCOD/BOD, total COD over BOD ratio	TCOD/BOD5	-	2.60	2.13	1.91	2.54	2.22
CODX/VSS, particulate (non-colloidal) COD/VSS	Fcv	-	3.34	1.60	1.35	1.19	1.75
Inorganic suspended solids (ash fraction of TSS)	ISS	mg/L	53	50	46	25	28

Note on the table: pH, Ca, Mg and alkalinity are used for pH and chemical precipitation reactions. Effluent COD_S is for systems with SRT > 3 days equal to the unbiodegradable (inert) soluble COD (Fus, SI). Glass-filtered COD (COD_{GF}) - Glass fibre filter 1.2 microns and typically is 40% of TCOD. Membrane-filtered COD (COD_{MF}) - flocculated and membrane filtered 0.45 microns and thereby CODMF<CODGF. BOD measurements all include ATU-inhibiting nitrification. Volatile fatty acids are often inaccurately assumed to be acetate.

8.2 Calculating BioWin influent fractions from influent measurements

The influent is divided into its fractions and used in the BioWin model calculations. Therefore, several influent fractions are introduced to facilitate the calculations in the model (Table 8.1, BW-influent tab). An overview of the fractions is presented below including their relation to actual analytical measurements.

8.2.1 BioWin COD calculations

The influent unbiodegradable COD (S_{US}) for systems with a SRT > 3 days is based on the effluent measurement of soluble (glass-filtered) COD according to:

$$S_{US} = COD_{S,EFF} = COD_{GF,EFF} \quad (8.1)$$

The fraction for unbiodegradable COD is calculated according to:

$$F_{US} = \frac{S_{US}}{TCOD} = \frac{COD_{GF,EFF}}{TCOD_{INF}} \quad (8.2)$$

Soluble COD includes the colloidal and is expressed as COD_S as the sum of all soluble model fractions. It can be measured from glass-filtered COD according to:

$$COD_S = S_{BSA} + S_{BSP} + S_{BSC} + X_{SC} + S_{US} = COD_{GF,INF} \quad (8.3)$$

Particulate (non-colloidal) COD (CODp or COD_X) is the sum of particulate (non-colloidal) COD, particulate unbiodegradable COD and active biomass in the influent (X_{BH} is often assumed to be zero) given by:

$$COD_X = X_{SP} + S_{UP} + X_{BH} \approx X_{SP} + S_{UP} \quad (8.4)$$

COD_X is calculated by subtracting the total COD and the soluble COD (COD_S) which is calculated based on the glass-filtered COD according to:

$$COD_X = TCOD - COD_S = TCOD_{INF} - COD_{GF,INF} \quad (8.5)$$

Soluble COD excluding the colloidal is expressed as COD_{MF} and measured by membrane filtering the COD according to:

$$COD_{MF} = S_{BSA} + S_{BSP} + S_{BSC} + S_{US} = COD_{MF,INF} \quad (8.6)$$

The total soluble readily biodegradable COD (the total of acetate, propionate and complex soluble COD but without slowly colloidal COD) is calculated from the measured micro-filtered fraction COD_{MF} according to:

$$S_{BS} = S_{BSA} + S_{BSP} + S_{BSC} = COD_{MF} - S_{US} = COD_{MF,INF} - COD_{GF,EFF} \quad (8.7)$$

The fraction of soluble readily biodegradable COD is given by:

$$F_{BS} = \frac{S_{BS}}{TCOD} = \frac{(S_{BSA}+S_{BSP}+S_{BSC})}{TCOD} = \frac{COD_{MF,INF}-COD_{GF,EFF}}{TCOD_{INF}} \quad (8.8)$$

Influent acetate (+ propionate) is direct measured as VFA:

$$S_{BSA} + S_{BSP} = VFA_{INF} \tag{8.9}$$

The fraction of readily biodegradable COD (which is acetate-COD) is given by:

$$F_{AC} = \frac{S_{BSA}}{S_{BS}} = \frac{S_{BSA}}{(S_{BSA}+S_{BSP}+S_{BSC})} = \frac{VFA_{INF}}{COD_{MF,INF}-COD_{GF,EFF}} \tag{8.10}$$

From the difference between the glass and membrane-filtered COD, the colloidal fraction can be calculated according to:

$$X_{SC} = COD_S - COD_{MF} = COD_{GF,INF} - COD_{MF,INF} \tag{8.11}$$

The last soluble parameter to be calculated is the complex soluble COD S_{BSC} calculated from the measurements according to:

$$S_{BSC} = COD_{MF} - S_{BSA} - S_{BSP} - S_{US} = COD_{MF,INF} - VFA_{INF} - COD_{GF,EFF} \tag{8.12}$$

The total soluble (readily and slow colloidal) biodegradable COD (S_S) is the total of acetate, propionate, complex soluble COD and colloidal COD (influent methanol is assumed to be zero) given by:

$$S_S = S_{BSA} + S_{BSP} + S_{BSC} + X_{SC} \tag{8.13}$$

And calculated according to:

$$S_S = COD_S - S_{US} = COD_{GF,INF} - COD_{GF,EFF} \tag{8.14}$$

Figure 8.2 shows the division of municipal wastewater Biodegradable COD (S_S) into constituent fractions. BioWin notation of S_S should not be confused with IAWQ notation where S_S equals BioWin S_{BS}.

The last two influent fractions that need to be calculated are related to the solids; particulate biodegradable COD and unbiodegradable COD according to Eq. 8.4:

$$COD_X = X_{SP} + S_{UP} + X_{BH} \approx X_{SP} + S_{UP}$$

These fractions are estimated from the BOD measurements in the influent as explained in the following sections of this chapter.

The BioWin influent tab the fraction of slowly biodegradable influent COD (which is particulate) is given by:

$$F_{XPS} = \frac{X_{SP}}{X_{SC}+X_{SP}} \tag{8.15}$$

VSS is often calculated from the ISS (Ash) measurement according to:

$$VSS = TSS - ISS \tag{8.16}$$

The influent ratio of particulate (non-colloidal) COD to VSS is given by:

$$F_{CV} = \frac{COD_X}{VSS} = \frac{TCOD - COD_S}{VSS} = \frac{TCOD_{INF} - COD_{GF,INF}}{TSS - ISS} \quad (8.17)$$

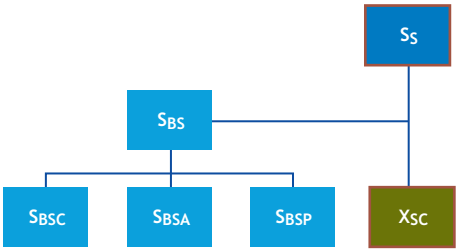

Figure 8.2 BioWin municipal wastewater soluble biodegradable COD (S_S). The fraction is measured by glass filtering and includes all soluble and colloidal material. Blue fractions are soluble and green fractions (colloidal) particulate.

8.2.2 BioWin N and P calculations

Ammonia is given by:

$$NH_3 = F_{NA} \times TKN \quad (8.18)$$

Soluble unbiodegradable organic nitrogen is given by:

$$N_{US} = F_{NUS} \times TKN \quad (8.19)$$

Nitrogen from organisms present in the influent is calculated by the sum of the products of the various organism concentrations and their respective nitrogen fractions, i.e.:

$$Organisms, N = \sum Zb_x - f_{N,Zbx} \quad (8.20)$$

Unbiodegradable particulate nitrogen is given by:

$$X_{IN} = F_{UP,N} \times F_{UP} \times TCOD \quad (8.21)$$

The remaining organic nitrogen is broken into particulate and soluble components. Particulate biodegradable organic nitrogen is given by:

$$X_{ON} = (TKN - NH_3 - N_{US} - X_{IN} - Organisms, N) \times F_{NOX} \quad (8.22)$$

Soluble biodegradable organic nitrogen is given by:

$$N_{OS} = (TKN - NH_3 - N_{US} - X_{IN} - Organisms, N) \times (1 - F_{NOX}) \quad (8.23)$$

Similarly, an explanation of the fractionation of influent phosphorus is as follows. Soluble orthophosphate is given by:

$$PO_4 = F_{PO4} \times TP \quad (8.24)$$

Phosphorus from organisms present in the influent is calculated by the sum of the products of the various organism concentrations and their respective phosphorus fractions, i.e.:

$$Organisms, P = \sum Zb_x - f_{P,Zbx} \quad (8.25)$$

Unbiodegradable particulate phosphorus is given by:

$$X_{IN} = F_{UP,P} \times F_{UP} \times TCOD \quad (8.26)$$

The remaining particulate biodegradable organic phosphorus is given by:

$$X_{OP} = TP - PO_4 - X_{IP} - Organisms, P \quad (8.27)$$

Chapter 9

BioWin BOD calculations

9.1 Introduction

The basis for modeling organic components in BioWin is the chemical oxygen demand (COD) parameter. However, BioWin allows for the calculation of soluble (filtered) and total carbonaceous biochemical oxygen demand (BOD) for any input element, process unit, or stream. The user may specify the time basis for the BOD calculation (5, 7, or 20 days).

The calculation of BOD is based on differing rates of degradation of the different components (e.g. influent biodegradable material (readily and slowly biodegradable), active organism masses which exert an endogenous oxygen demand and hence a BOD, etc.). Essentially BioWin uses analytical equations to estimate BOD.

The objective here is to demonstrate the method used by BioWin in calculating filtered and total BOD concentrations, and show how the equations were developed. Understanding the development of the following equations is facilitated by the figure below, which illustrates the division of a wastewater stream into its different components. The BioWin model defines additional fractions for propionate and colloidal (glass-filtered) and particulate material. Note that SS has different meanings in the BioWin and IAWQ models (BioWin includes colloidal material which the IAWQ models do not take into account). The approach for calculating BOD is to distinguish three components, and to calculate the BOD contribution for each component independently. These components are as follows:

1. BOD associated with utilization of soluble COD (both readily biodegradable and colloidal slowly biodegradable) and the biomass generated from this utilization;
2. BOD associated with the utilization of slowly biodegradable particulate COD and the biomass generated from this utilization, and
3. BOD exerted by active biomass presented in the sample. That is the biomass initially present; not the biomass generated through the utilization referred to in 1 and 2.

The simulator provides values for S_{BSC}, S_{BSA}, X_{SC}, X_{SP} and X_{BH} in any stream or process unit. When calculating BOD concentrations for input streams, the influent fractions provided by the user (or the default values) are used to calculate the appropriate concentrations. A definition of terms for the BOD calculation is given in Table 9.1.

Table 9.1 BioWin definition of terms used in influent calculations

Parameter	Description
b	Endogenous decay rate constant (≈ 0.24 d^{-1})
BOD_E	Biochemical oxygen demand due to endogenous respiration
BOD_S	Soluble component of biochemical oxygen demand
BOD_{XSP}	Particulate slowly biodegradable component of biochemical oxygen demand
BOD_{XBH}	Active biomass component of biochemical oxygen demand
BOD_T	Total biochemical oxygen demand
f	Fraction of active mass remaining as endogenous residue (≈ 0.20)
f_{BS}	Fraction of total influent COD which is readily biodegradable
f_{UP}	Fraction of total influent COD which is unbiodegradable particulate
f_{US}	Fraction of total influent COD which is unbiodegradable soluble
f_{XBH}	Fraction of total influent COD which is active organisms
f_{XSP}	Fraction of slowly degradable COD which is particulate
k	First order rate constant for X_{SP} degradation (≈ 0.40 d^{-1})
MO_G	Mass of oxygen utilized for growth on soluble substrate
MO_E	Mass of oxygen utilized for endogenous metabolism
OUR	Oxygen utilization rate (mg/L.day)
OUR_E	Oxygen utilization rate due to endogenous metabolism
OUR_G	Oxygen utilization rate for substrate utilization (growth)
S_{BS}	Soluble readily biodegradable COD concentration (mgCOD/L)
S_{BSA}	Soluble readily biodegradable volatile fatty acid COD concentration (mgCOD/L)
S_{BSP}	Soluble readily biodegradable propionate COD concentration (mgCOD/L)
S_{BMETH}	Soluble readily biodegradable methanol COD concentration (mgCOD/L)
S_{BSC}	Soluble readily biodegradable complex COD concentration (mgCOD/L)
S_S	Soluble (filtered) biodegradable COD concentration (mgCOD/L)
S_{UP}	Particulate unbiodegradable COD concentration (mgCOD/L)
S_{US}	Soluble unbiodegradable COD concentration (mgCOD/L)
S_T	Total COD concentration (mgCOD/L)
t	Time (d)
X_{BH}	Active organism concentration (total of seven organism concentrations, mg COD/L)
X_{BH0}	Active organism concentration at time zero (mgCOD/L)
X_S	Slowly biodegradable COD concentration (mgCOD/L)
X_{SC}	Slowly biodegradable colloidal COD concentration (mgCOD/L)
X_{SP}	Slowly biodegradable particulate COD concentration (mgCOD/L)
X_{SP0}	Slowly biodegradable particulate COD concentration at time zero (mgCOD/L)
Y	Yield of active organisms (≈ 0.666 mg cell COD per mg COD)

Note: fractions in BioWin are indicated both as standard (f) and capital (F).

9.2 BOD associated with soluble biodegradable COD (S_S)

For the purpose of BOD calculations, the 'soluble' biodegradable COD (S_S) can be considered to be the sum of the readily biodegradable COD (methanol is usually zero) and the biodegradable colloidal COD (X_{SC}), i.e.

$$S_S = S_{BSC} + S_{BSA} + S_{BSP} + S_{BMETH} + X_{SC} \quad (9.1)$$

The oxidation of this material occurs rapidly, and for the purpose of these calculations it is assumed to occur instantaneously (i.e. at t = 0). Assuming that the soluble biodegradable COD is oxidized instantly to form new cells:

$$S_S \rightarrow X_{BH,0} = Y \cdot S_S \text{ (at time t=0)} \tag{9.2}$$

then the mass of oxygen consumed for the growth of these organisms MO_G is:

$$MO_G = (1 - Y) \cdot S_S \tag{9.3}$$

Assuming that the rate of change in organism concentration due to endogenous decay is first order with respect to the active organism concentration, an expression for active organism concentration can be obtained, i.e.

$$\frac{dX_{BH}}{dt} = -b \cdot X_{BH} \tag{9.4}$$

Therefore,

$$X_{BH} = X_{BH,0} \cdot e^{-b \cdot t} \tag{9.5}$$

The BOD due to endogenous metabolism by the organisms generated from growth on S_S can then be calculated using either of the methods described below.

9.3 BOD due to endogenous metabolism; method 1

From (9.1) and (9.5), the endogenous mass loss at time t can be calculated:

$$\Delta X_{BH} = X_{BH,0} - X_{BH} = X_{BH,0} \cdot (1 - e^{-b \cdot t}) = Y \cdot S_S \cdot (1 - e^{-b \cdot t}) \tag{9.6}$$

The mass of oxygen consumed for endogenous metabolism:

$$MO_E = (1 - f) \cdot \Delta X_{BH} = (1 - f) \cdot Y \cdot S_S \cdot (1 - e^{-b \cdot t}) \tag{9.7}$$

This is equal to the BOD due to endogenous decay, i.e.

$$BOD_E = (1 - f) \cdot X_{BH,0} \cdot (1 - e^{-b \cdot t}) = (1 - f) \cdot Y \cdot S_S \cdot (1 - e^{-b \cdot t}) \tag{9.8}$$

The soluble BOD component can now be calculated by summing (9.3) and (9.8), i.e.

$$BOD_S = MO_G + MO_E = (1 - Y) \cdot S_S + (1 - f) \cdot Y \cdot S_S \cdot (1 - e^{-b \cdot t}) \tag{9.9}$$

9.4 BOD due to endogenous metabolism; method 2

The OUR due to endogenous metabolism can also be related to the rate of change in active organism concentration (taking into account that a fraction of the active mass becomes endogenous residue), i.e.

$$OUR_E = -(1 - f) \cdot \frac{dX_{BH}}{dt} = b \cdot (1 - f) \cdot X_{BH} = b \cdot (1 - f) \cdot X_{BH,0} \cdot e^{-b \cdot t} \tag{9.10}$$

The BOD due to endogenous decay is then the cumulative mass of oxygen used over time, i.e.

$$BOD_E = \int_0^t OUR_E \cdot dt = b \cdot (1-f) \cdot X_{BH,0} \cdot \int_0^t e^{-b \cdot t} \cdot dt \quad \cdots =$$
$$= b \cdot (1-f) \cdot X_{BH,0} \cdot \left(-\frac{1}{b} \cdot e^{-b \cdot t}\right) + C = -(1-f) \cdot X_{BH,0} \cdot e^{-b \cdot t} + C \qquad (9.11)$$

and since at t = 0, $BOD_E = 0$,

$$C = (1-f) \cdot X_{BH,0}$$

Substituting the expression obtained for X_{BHO} (9.2) into the above gives an expression for the BOD due to endogenous decay:

$$BOD_E = -(1-f) \cdot X_{BH,0} \cdot e^{-b \cdot t} + (1-f) \cdot X_{BH,0} =$$
$$= (1-f) \cdot X_{BH,0} \cdot (1 - e^{-b \cdot t}) = (1-f) \cdot Y \cdot S_S \cdot (1 - e^{-b \cdot t}) \qquad (9.12)$$

This is the same expression obtained using Method 1 (9.8). The total soluble BOD portion is given by (9.9).

$$BOD_S = (1-Y) \cdot S_S + (1-f) \cdot Y \cdot S_S \cdot (1 - e^{-b \cdot t})$$

9.5 BOD associated with slowly biodegradable particulate COD (X_{SP})

The biochemical oxygen demand related to the slowly biodegradable particulate material can be calculated from the cumulative oxygen consumption for growth on X_{SP} and endogenous respiration exerted by organisms from this growth. The Oxygen Utilization Rate (OUR) is the sum of two components:

$$OUR = OUR_G + OUR_E = (1-Y) \cdot \frac{-dX_{SP}}{dt} + (1-f) \cdot \frac{-dX_{BH}}{dt} =$$
$$= (1-Y) \cdot \frac{-dX_{SP}}{dt} + (1-f) \cdot b \cdot X_{BH} \qquad (9.13)$$

The rate of change in slowly biodegradable particulate substrate concentration is assumed to be first order with respect to substrate concentration, i.e.

$$\frac{dX_{SP}}{dt} = -k \cdot X_{SP}$$

Therefore,

$$X_{SP} = X_{SP,0} \cdot e^{-k \cdot t} \qquad (9.14)$$

The active organism concentration in 9.4 is obtained by integrating an expression for the rate of change of X_{BH}. This rate is a consequence of an increase due to growth on X_{SP}, minus a decrease due to endogenous metabolism, i.e.

$$\frac{dX_{BH}}{dt} = Y \cdot \frac{-dX_{SP}}{dt} - b \cdot X_{BH} \qquad (9.15)$$

Substituting 9.14 into the above results in a linear differential equation for X_{BH} as a function of time, i.e.

$$\frac{dX_{BH}}{dt} = -Y \cdot \frac{dX_{SP}}{dt} - b \cdot X_{BH} = -Y \cdot \frac{d}{dt} \cdot \left(X_{SP,0} \cdot e^{-k \cdot t}\right) - b \cdot X_{BH} =$$
$$= k \cdot Y \cdot X_{SP,0} \cdot e^{-k \cdot t} - b \cdot X_{BH} \quad (9.16)$$

Or equivalently,

$$\frac{dX_{BH}}{dt} + b \cdot X_{BH} = k \cdot Y \cdot X_{SP,0} \cdot e^{-k \cdot t} \quad (9.17)$$

An integration factor of e^{bt} can be used to solve the differential equation. Multiplying both sides of the equation by e^{bt} results in a simplified expression:

$$e^{b \cdot t} \cdot \frac{dX_{BH}}{dt} + e^{b \cdot t} \cdot b \cdot X_{BH} = e^{b \cdot t} \cdot k \cdot Y \cdot X_{SP,0} \cdot e^{-k \cdot t} \frac{d}{dt}(e^{b \cdot t} \cdot X_{BH}) =$$
$$= k \cdot Y \cdot X_{SP,0} \cdot e^{(b-k) \cdot t} \quad (9.18)$$

Integrating both sides of 9.18:

$$(e^{b \cdot t} \cdot X_{BH}) = \int k \cdot Y \cdot X_{SP,0} \cdot e^{(b-k) \cdot t} \cdot dt (e^{b \cdot t} \cdot X_{BH}) = \frac{k \cdot Y \cdot X_{SP,0}}{b-k} \cdot e^{(b-k) \cdot t} + C X_{BH} =$$
$$= \frac{k \cdot Y \cdot X_{SP,0}}{b-k} \cdot e^{-k \cdot t} + C \cdot e^{-b \cdot t} \quad (9.19)$$

At t = 0, $X_{BH} = X_{BHO}$, therefore solving C gives:

$$X_{BH} = \frac{k \cdot Y \cdot X_{SP,0}}{b-k} \cdot e^{-k \cdot t} + \left(X_{BH,0} - \frac{k \cdot Y \cdot X_{SP,0}}{b-k}\right) \cdot e^{-b \cdot t} \quad (9.20)$$

Returning to (9.13) for the oxygen utilization rate and substituting for X_{BH}:

$$OUR = (1-Y) \cdot \frac{-dX_{SP}}{dt} + (1-f) \cdot b \cdot X_{BH} = (1-Y) \cdot \frac{-dX_{SP}}{dt} + (1-f) \cdot b \cdot X_{BH}$$
$$= (1-Y) \cdot k \cdot X_{SP} + (1-f) \cdot b \cdot X_{BH} =$$
$$= (1-Y) \cdot k \cdot X_{SP,0} \cdot e^{-k \cdot t} + (1-f) \cdot b \cdot \left\{\left(\frac{k \cdot Y \cdot X_{SP,0}}{b-k}\right) \cdot e^{-k \cdot t} + \left(X_{BH,0} - \frac{k \cdot Y \cdot X_{SP,0}}{b-k}\right) \cdot e^{-b \cdot t}\right\} \quad (9.21)$$

Integrating 9.21 gives an expression for the BOD of the slowly biodegradable material as a function of time, i.e.

$$BOD_{XSP} = \int_0^t OUR \cdot dt \; =$$
$$= \int_0^t (1-Y) \cdot k \cdot X_{SP,0} \cdot e^{-k \cdot t} + (1-f) \cdot b$$
$$\cdot \left\{\left(\frac{k \cdot Y \cdot X_{SP,0}}{b-k}\right) \cdot e^{-k \cdot t} + \left(X_{BH,0} - \frac{k \cdot Y \cdot X_{SP,0}}{b-k}\right) \cdot e^{-b \cdot t}\right\} \cdot dt$$

Assuming $X_{BHO} \approx 0$ and re-arranging the above expression,

$$BOD_{XSP} = X_{SP,0} \cdot \begin{bmatrix} \left((1-Y) + \frac{(1-f)\cdot b \cdot Y}{b-k}\right) \cdot \\ (1 - \cdot\, e^{-k\cdot t}) - \left(\frac{(1-f)\cdot k \cdot Y}{b-k}\right) \cdot (1 - e^{-b\cdot t}) \end{bmatrix} \quad (9.22)$$

The total BOD (if no biomass is present) is then the sum of BOD_S and BOD_{XSP}.

$$BOD_T = X_{SP,0} \cdot \begin{bmatrix} \left((1-Y) + \frac{(1-f)\cdot b \cdot Y}{b-k}\right) \cdot (1 - \cdot\, e^{-k\cdot t}) \\ -\left(\frac{(1-f)\cdot k \cdot Y}{b-k}\right) \cdot (1 - e^{-b\cdot t}) \end{bmatrix} +$$

$$+(1-Y)\cdot S_S + (1-f)\cdot Y \cdot S_S \cdot (1 - e^{-b\cdot t}) \quad (9.23)$$

9.6 Example of BOD calculation

This example demonstrates the BOD calculation procedure for an influent wastewater stream with the following characteristics:

Table 9.2 Example of calculation to demonstrate the BioWin BOD calculation from COD fractions

Symbol	Value	Description
S_T	500	Total Influent COD (mgCOD/L)
f_{US}	0.10	Fraction of total influent COD which is unbiodegradable soluble
f_{UP}	0.08	Fraction of total influent COD which is unbiodegradable particulate
f_{BS}	0.20	Fraction of total influent COD which is readily biodegradable
f_{XBH}	0.00	Fraction of total influent COD which is active organisms
F_{XSP}	0.75	Fraction of slowly degradable COD which is particulate

Note: fractions in BioWin are indicated both as standard (f) and capital (F).

The influent 'soluble' and slowly biodegradable concentrations are calculated using the influent fractions:

$$S_{BS} = f_{BS} \cdot S_T \quad (9.24)$$

According to Table 9.2:

$$S_{BS} = 0.2 \times 500 = 100 \text{ mg COD/L}$$

$$X_S = (1 - f_{US} - f_{UP} - f_{BS} - f_{XBH}) \cdot S_T \quad (9.25)$$

According to Table 9.2:

$$X_S = (1 - 0.1 - 0.08 - 0.2 - 0) \times 500 = 310 \text{ mg COD/L}$$

$$X_{SC} = (1 - f_{XPS}) \cdot X_S \quad (9.26)$$

According to Table 9.2:

$$X_{SC} = (1 - 0.75) \times 310 = 77.5 \text{ mg COD/L}$$

$$X_{SP} = f_{XPS} \cdot X_S \tag{9.27}$$

According to Table 9.2:

$$X_{SP} = 0.75 \times 310 = 232.5 \text{ mg COD/L}$$

$$S_S = S_{BS} + X_S \tag{9.28}$$

According to Table 9.2:

$$S_S = 100 + 77.5 = 177.5 \text{ mg COD/L}$$

The soluble BOD (BOD_S) can then be calculated using the parameters listed below:

Table 9.3 BioWin BOD model parameters using in example calculations

Symbol	Value	Description
f	0.20	Fraction of active mass remaining as endogenous residue (≈ 0.20)
Y	0.67	Yield of active organisms (≈ 0.666 mg cell COD per mg COD)
b	0.24	Endogenous decay rate constant (≈ 0.24 d^{-1})
k	0.40	First order rate constant for XSP degradation (≈ 0.40 d^{-1})

Note: The parameter values for *f* and b are for endogenous respiration, and should not be confused with the corresponding terms used in the death-regeneration approach for modeling biomass decay. The time factor is based on BOD_5 (t = 5 days).

$$BOD_S = (1 - Y) \cdot S_S + (1 - f) \cdot Y \cdot S_S \cdot \left(1 - e^{(-b \cdot t)}\right) \tag{9.29}$$

$$BOD_S = (1 - 0.666) \cdot 177.5 + (1 - 0{,}2) \cdot 0.666 \cdot 177.5 \cdot \left(1 - e^{(-0.24 \cdot 5)}\right) = 125.4 \text{ mg COD/L}$$

Similarly, the slowly degradable BOD_5 can be calculated:

$$BOD_{XSP} = X_{SP}\left(\begin{array}{c}\left[(1 - Y) + \frac{(1-f)\cdot b \cdot Y}{(b-k)}\right] \cdot \\ \left(1 - e^{(-k \cdot t)}\right) - \left[\frac{(1-f)\cdot k \cdot Y}{(b-k)}\right] \cdot \left(1 - e^{(-b \cdot t)}\right)\end{array}\right) \tag{9.30}$$

$$BOD_{XSP} = 232.5\left(\begin{array}{c}\left[(1 - 0.666) + \frac{(1 - 0.2) \cdot 0{,}24 \cdot 0{,}666}{(0.24 - 0.4)}\right] \cdot \\ \left(1 - e^{(-0.4 \cdot 5)}\right) - \left[\frac{(1 - 0.2) \cdot 0.4 \cdot 0.666}{(0.24 - 0.4)}\right] \cdot \left(1 - e^{(-0.24 \cdot 5)}\right)\end{array}\right) = 122.9 \text{ mg COD/L}$$

In this example the active organism fraction is zero, therefore the total BOD_5 is the sum of the BOD_S and the BOD_{XSP}, i.e.

$$BOD_T = BOD_S + BOD_{XSP} \tag{9.31}$$

$$BOD_T = 125.4 + 122.9 = 248.3 \text{ mg BOD/L}$$

9.7 Effluent BOD

Knowledge of the effluent BOD is useful for quantifying the impact of plant discharges on receiving water bodies, and often is of interest in terms of regulatory requirements and consent limits. Currently BioWin provides information on a number of biodegradable components in any plant output stream (and internal streams). Therefore it is a simple step to quantify both the soluble and total (soluble + particulate) BOD. In BioWin this calculation is based on differing rates of degradation of the different components [e.g. un-degraded influent biodegradable material (readily and slowly biodegradable), active organism masses which exert an endogenous oxygen demand and hence a BOD, etc.]. Essentially BioWin simulates a BOD test for the specified number of days and wastewater composition. This should allow for more accurate estimation of BOD than is provided with models based on the BOD parameter.

9.8 COD versus BOD as a modeling parameter

The advantage of selecting COD as the parameter for quantifying the 'strength' of organic material in the influent, as opposed to BOD or TOC, is that it provides a consistent basis for the description of the activated sludge process. Marais and Dold (1985) outlined the rationale which makes COD the appropriate parameter. It is worth reviewing this rationale briefly as selection of the COD parameter is fundamental to the application of the models. Briefly, the suitability of COD is established by considering the utilization of organic substrate. In the process of metabolism the organic substrate serves two functions for the organisms as shown schematically in the arrow diagram below:

1. A portion of the organic material is oxidized into CO_2 and water, providing energy for maintaining the homeostatic balance for the existing cell mass (osmotic pressure, ionic balance, membrane potential, etc.) and for (2) below. The energy is provided by transferring electrons from the organic substrate through the electron transport chain to the terminal electron acceptor (oxygen in the case of an aerobic system, or nitrate under anoxic conditions). Under the substrate-limited conditions usually encountered in activated sludge systems the organisms utilize a relatively fixed fraction of the energy available from oxidation for the energy consuming processes.
2. The remaining portion of the organic material is converted into a new heterotrophic cell mass, utilizing energy available from process (1).

Regarding the relative proportions allocated between (1) and (2), this is quantified by the ratio termed the specific yield:

$$Y_H = \frac{\text{cell mass formed}}{\text{substrate utilized}} \tag{9.32}$$

From theoretical considerations, under balanced growth conditions the specific yield, in terms of the mass of organisms produced *per electron available for transfer* in the biodegradable organic substrate, should be near constant. This was confirmed by Payne (1970), who reviewed a large number of experimental results for both pure and mixed cultures of heterotrophic organisms. Therefore, if the electron-donating potential of the organic substrate in the activated sludge system influent (consisting of a broad spectrum of compounds) is measured, it is possible to quantify sludge production [from Y_H] and oxygen demand [from $(1\text{-}Y_H)$]. This observation provides the rationale for selecting the COD parameter.

The electron-donating capacity of organic material is measured in the COD test. In the test each mole of oxygen (O_2) accepts four electron equivalents (e^- eq); therefore, the COD is a direct measure of the electron-donating potential. The link between electron equivalents (COD) of the substrate, the near constant yield of organism mass per unit substrate COD, and the corresponding fixed oxygen requirement per unit substrate COD metabolized, make the COD a fundamental parameter in the analysis of activated sludge behaviour. A factor that adds impetus to the selection is that mass balances can be made in terms of COD. As electrons cannot be created or destroyed, in a bio system operated at steady state the mass of COD entering with the influent per unit time must equal the sum of (1) the mass of COD leaving in the effluent, (2) the COD of the wasted sludge, and (3) the oxygen (or oxygen equivalent) consumed in the utilization of the organic material (from the oxygen utilization rate measurement); that is, a mass balance is possible.

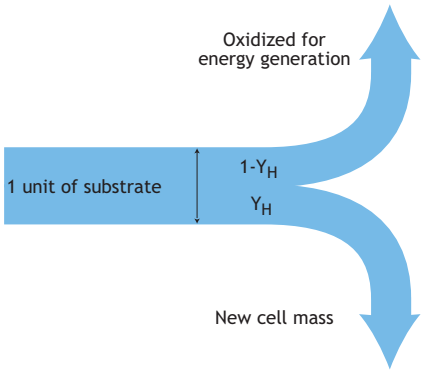

Figure 9.1 Schematic representation of the utilization of substrate by heterotrophic organisms showing the division between substrate oxidized for energy generation and growth of new cell mass.

Characteristics of the COD as a measure of the 'strength' of wastewater are not realized by either the BOD or the TOC parameters. The BOD measures only that portion of the e^- eq. in the substrate utilized for energy generation and excludes the portion of the substrate e^- eq. transformed into new cell mass. Therefore BOD cannot be used as the basis for a mass balance. The TOC is deficient in that the ratio of carbon/e^- eq. differs between organic compounds and therefore TOC is an inappropriate parameter when dealing with the mixed substrate influents to wastewater treatment plants. Therefore, the user must always specify influent concentrations in terms of COD; however, the simulator can estimate the soluble and total BOD for any input element, process unit, or stream. The user may specify the time basis for the BOD calculation (5, 7, or 20 days).

9.9 Example results of BioWin influent characterization

Based on the influent characterization method presented here, the influent is calculated for the five WWTP model design case studies. Table 9.5 shows the different influent fractions as used by the BioWin model for COD, nitrogen, phosphorus and TSS. The numbers in the table are calculated based on Table 9.4.

Table 9.4 BioWin model fractions used to calculate the influent characterization

Symbol	Description	Unit	Belišće	Čakovec	Varaždin	Vinkovci	Zagreb
Fbs	Readily biodegradable COD (including Acetate)	gCOD/g of total COD	0.248	0.070	0.506	0.275	0.157
Fac	Acetate COD	gCOD/g of readily biodegradable COD	0.000	0.000	0.000	0.011	0.000
Fxsp	Non-colloidal (particulate) slowly biodegradable	gCOD/g of slowly degradable COD	0.680	0.750	0.150	0.935	0.880
Fus	Unbiodegradable soluble COD	gCOD/g of total COD	0.032	0.052	0.052	0.047	0.035
Fup	Unbiodegradable particulate COD	gCOD/g of total COD	0.350	0.180	0.390	0.047	0.225
Fna	Ammonia	gNH3-N/gTKN	0.500	0.931	0.322	0.599	0.290
Fnox	Particulate organic nitrogen (biodegradable)	gN/g Organic N	0.720	0.810	0.442	0.678	0.809
Fnus	Soluble Unbiodegradable Organic Nitrogen	gN/gTKN	0.000	0.010	0.020	0.010	0.010
FupN	N:COD ratio for unbiodegradable part. COD	gN/gCOD	0.000	0.001	0.069	0.005	0.068
Fpo4	Phosphate	gPO4-P/gTP	0.300	0.960	0.706	0.710	0.483
FupP	P:COD ratio for unbiodegradable part. COD	gP/gCOD	0.000	0.001	0.006	0.001	0.002
FZbh	Non poly-P heterotrophs	gCOD/g of total COD	0.001	0.001	0.001	0.001	0.001
FZbm	Anoxic methanol utilizers	gCOD/g of total COD	0.001	0.001	0.001	0.001	0.001
FZaob	Ammonia oxidizers	gCOD/g of total COD	0.001	0.001	0.001	0.001	0.001
FZnob	Nitrite oxidizers	gCOD/g of total COD	0.001	0.001	0.001	0.001	0.001
FZamob	Anaerobic ammonia oxidizers	gCOD/g of total COD	0.001	0.001	0.001	0.001	0.001
FZbp	PAOs	gCOD/g of total COD	0.001	0.001	0.001	0.001	0.001
FZbpa	Propionic acetogens	gCOD/g of total COD	0.001	0.001	0.001	0.001	0.001
FZbam	Acetoclastic methanogens	gCOD/g of total COD	0.001	0.001	0.001	0.001	0.001
FZbhm	H2-utilizing methanogens	gCOD/g of total COD	0.001	0.001	0.001	0.001	0.001
-	Particulate substrate COD:VSS ratio	mgCOD/mgVSS	2.000	1.950	1.600	1.370	1.950
-	Particulate inert COD:VSS ratio	mgCOD/mgVSS	1.000	1.350	1.240	1.080	1.070

Table 9.5 BioWin influent fractions for COD, N, P and TSS

Symbol	Description	Unit	Belišće	Čakovec	Varaždin	Vinkovci	Zagreb
COD components							
	Total COD	mgCOD/L	729.8	495.1	394.6	577.9	419.2
	Biodegradable COD of which;	mgCOD/L	451.0	380.2	220.2	523.4	310.2
Sbsc	Readily biodegradable COD (excluding acetate)	mgCOD/L	181.0	34.7	199.7	157.3	65.8
Sbsa	Acetate (COD-VFA)	mgCOD/L	0.0	0.0	0.0	1.7	0.0
	Slowly biodegradable COD of which;	mgCOD/L	270.0	345.6	20.5	366.1	244.4
Xsc	Colloidal slowly biodegradable	mgCOD/L	86.4	86.4	17.4	23.8	29.3
Xsp	Non-colloidal (particulate) slowly biodegradable	mgCOD/L	183.6	259.2	3.1	342.3	215.1
	Unbiodegradable of which;	mgCOD/L	278.8	114.9	174.4	54.6	109.0
SI	Unbiodegradable soluble COD	mgCOD/L	23.4	25.7	20.5	27.4	14.7
XI	Unbiodegradable particulate COD	mgCOD/L	255.4	89.1	153.9	27.2	94.3
Nitrogen components							
	Total Nitrogen	mgN/L	62.0	31.8	34.4	32.2	58.2
NO3	Nitrate-N	mgN/L	0.0	3.6	0.6	0.0	0.5
	Total Kjeldahl N	mgN/L	62.0	28.1	33.8	32.2	57.7
NH3	Ammonia	mgN/L	31.0	26.2	10.9	19.3	16.7
	Organic N of which;	mgN/L	31.0	1.9	22.9	12.9	41.0
	Unbiodegradable organic N of which;	mgN/L	0.0	0.4	11.3	0.4	7.0
Nus	Soluble unbiodegradable organic N	mgN/L	0.0	0.3	0.7	0.3	0.6
Xin	Particualte unbiodegradable organic N	mgN/L	0.0	0.1	10.6	0.1	6.4
	Biodegradable organic N of which;	mgN/L	31.0	1.6	11.6	12.5	34.0
Xon	Particulate biodegradable organic N	mgN/L	22.3	1.3	5.1	8.5	27.5
Nos	Soluble organic biodegradable N	mgN/L	8.7	0.3	6.5	4.0	6.5
Phosphorus components							
	Total P	mgP/L	8.4	3.7	4.3	3.3	4.6
PO4	Phosphate	mgP/L	2.5	3.6	3.0	2.3	2.2
	Organic P of which;	mgP/L	5.9	0.1	1.3	1.0	2.4
	Unbiodegradable organic P of which;	mgP/L	0.0	0.1	0.9	0.0	0.2
Xip	Particulate unbiodegradable organic P	mgP/L	0.0	0.1	0.9	0.0	0.2
	Soluble unbiodegradable organic P (=0)	mgP/L	0.0	0.0	0.0	0.0	0.0
	Biodegradeble organic P of which;	mgP/L	5.9	0.1	0.4	0.9	2.2
Xop	Particulate biodegradable organic P	mgP/L	5.9	0.1	0.4	0.9	2.2
	Soluble biodegradeble organic P (=0)	mgP/L	0.0	0.0	0.0	0.0	0.0
Total solids							
TSS	Total suspended solids of which;	mgTSS/L	400.4	226.9	150.9	320.5	248.4
ISS	Inorganic suspended solids	mgISS/L	53.2	28.0	24.9	45.5	50.0
VSS	Volatile suspended solids of which;	mgVSS/L	347.2	198.9	126.0	275.0	198.4
	Slowly biodegradable organic VSS of which;	mgVSS/L	91.8	132.9	1.9	249.9	110.3
	Colloidal (not accounted particulate)	mgVSS/L	0.0	0.0	0.0	0.0	0.0
	Non-colloidal (particulate)	mgVSS/L	91.8	132.9	1.9	249.9	110.3
	Unbiodegradable particulate organic VSS	mgVSS/L	255.4	66.0	124.1	25.2	88.1

Chapter 10

Model protocol; step 1: plant inventory

10.1 Introduction

In a preliminary plant assessment (the inventory phase), information is gathered regarding the plant history, construction, operation and operational issues. This information usually contains a collection of unprocessed ('raw') plant information that is not yet fully verified. From the initial plant assessment, follow-up visits can be planned with the aim of collecting more detailed information, executing interviews with plant operators and managers, and acquiring additional data and measurements that may need to be performed. The information gathered by such an inventory (which is usually presented in a kind of inception report) should be considered as an indicative starting point required for each modeling study. Because the information contains mainly raw data from a quick scan obtained from different sources, the information is likely to contain conflicting data and sometimes errors.

After the first evaluation of the raw data, it may be decided to make use of selected parts of the historical data, focusing either only on periods of typical (usual) plant performance or focusing on the most representative (and preferably complete) set of data for the study which best represent the average plant performance over a longer period. Therefore information collected and presented from the inventory phase does not necessarily reflect the information used in the further design calculations and modeling design study; it all depends on the purpose of the study and the quality of the collected data. This statement should be mentioned when presenting raw data in the reports and therefore raw (indicative) data should also be clearly marked as such.

The information that is eventually used in the design calculations is typically obtained from different sources and first needs to be checked for consistency. For this purpose different methods for error detection and data reconciliation can be used. Some general methods for this are presented elsewhere in this report. Eventually the modeling and retrofitting study is the final checkpoint for the consistency of the design input data. The ultimate goal of a design study is to fit a mathematical model describing the plant performance to the collected information and measurements on site. In the (increasingly unlikely) event that a satisfactory description of the reality by a mathematical model cannot be (fully) obtained, this is most probably an indication of an incorrect process description, errors in the process layout (Process Flow Diagram - PFD), an incorrect description of the hydraulic system, inaccurate measurements due to inaccurate analytical procedures or inadequate sampling protocols. In practice, these issues are very likely to occur and therefore the reason to perform a plant operational assessment in the first place is often to identify practical operational deficiencies. That is also why the identified gaps in the plant information coming to the fore from a model study will often need to be addressed in order to obtain a fit of the model to the reality within a reasonable margin of error. Typically this is carried out based on sound reasoning and expert judgment. To do this assessment properly, information collected in the inventory phase can be used to underpin the necessary estimations and assumptions in the design argumentation. For this reason the data collected in the inventory

phase often serves as a important source of information for the rest of the plant study and therefore should be paid attention to and collected with care, always with the particular purpose of modeling in mind. From a broader project perspective, the inventory phase has an important role in the modeling protocol; it provides an early project overview and a general idea of the quality and availability of the design information. It furthermore serves for planning the following steps in the data acquisition and gives direction to the approach and focus of the design study for plant optimization and/or upgrade. The inventory is therefore the first step in the modeling protocol with the purpose of collecting and screening/assessing/filtering the information useful for plant modeling and design.

In the sections to follow, the typical information to be gathered in a plant inventory is listed below (Table 10.2). The information of interests will differ from project to project depending on its goal; however, this chapter gives a generic (however not absolutely exhaustive) overview of typical information that should be collected in such an exercise. The chapter can serve as an exemplary checklist for WWTP model design and retrofit/upgrade studies. In Appendix 10 of these guidelines the full checklists are presented in a tabular form. These are in fact the same checklists that were used to collect the critical information from the five selected case studies of the project.

10.2 General information

The inventory phase should start with collecting general project and contact information. A summary example is presented in Table 10.1 below.

Table 10.1 Typical inventory data: contact information

Company contact information	Company name: Contact name: Function in the organization: E-mail address: Telephone: Office address (Post Box): Visiting address:
WWTP contact information	Name WWTP: Contact name: Function: E-mail address: Telephone: WWTP visiting address:
Date of visit to plant by consultants	
Google Maps	Satellite and areal location overview: Map coordinates:



Figure 10.1 Location information showing a map overview of a connected residential area, WWTP and recipient (e.g. WWTP Belišće, Google Maps, 45.684807,18.418944, +45° 41' 5.31", +18° 25' 8.20"). The WWTP is indicated by the arrow.

10.3 References on the gathered information

When collecting information, all the information sources and references should be systematically stored and organized. References can include all relevant information; e.g. date of publication, name of publication or report, name of the publication source, or number of total pages and pages from the reports that are of interest, etc. For verbal information, the name of the source contact person, and time and nature of the meeting (e.g. plant visit, telephone interview, email, etc.). Reference information should also be easily traceable (via the information source/reference) to assure quality control of the information and to possibly reproduce (validate) the collected information in a later phase. Typical information sources are:

- Archives (e.g. of Hrvatske Vode, company data storage);
- Technical questionnaires (as presented in this report);
- Interviews with operators and plant managers (verbal information);
- Plant inventory visit(s) by consultants;
- Photographs (e.g. taken during plant inventory visits);
- Public information (published information in publications, internet, newspapers, etc.);
- Technical studies and reports;
- Original plant design documents.

10.4 General description of the plant history

To better understand the current plant operation, historical information of the plant performance often gives insight into the current operation and typical functioning of process units. Usually, this information is obtained from verbal sources, design reports and (optimization/upgrading) studies. For better understanding of the current operational situation, a sequence of the developments can be briefly described starting from the first development and finishing with the current operations. General historical information of interest can be:

- Different (planned) phases in time of the design and operation;
- Changes in plant capacity (PE and flow);
- Changes in effluent discharge regulation (designed MAC: Maximum Allowed Concentration);
- Process units that are introduced later, rebuilt or removed;
- Information on the WWTP design and construction company;
- Information on the historical state of the sewer system;
- Changes in industrial and touristic activities;
- Changes in the influent composition or flow (quality and quantity);
- Typical operational problems that have occurred in the operational history;
- Any other relevant information.

10.5 Information on legislation and monitoring

In the inventory phase, information is collected on the European, national and local wastewater discharge regulations and the regulatory system of monitoring and discharge control. The following information can be included:

- Listing of relevant effluent parameters and the MAC-value (Maximum Allowed Concentration);
- Required measurement frequency (e.g. daily or weekly) and methodology (e.g. grab samples or 24-hour flow composite);
- Other required regulatory measurements (e.g. influent, sludge production, disposal, etc.);
- References in the official regulation documents (official publications);
- Contact information for the organization responsible for quality and (effluent) discharge control (contact person and address);
- Contact information for the laboratory responsible for the regulatory plant measurements;
- Contact information for the laboratory responsible for the daily plant analysis.

10.6 Description of the WWTP effluent recipient

The recipient is typically classified in a category of water body according to an environmental impact classification of the WWTP effluent recipient. The classification of the recipient will also determine the wastewater treatment plant effluent discharge legislation. Care should be taken that in some cases the recipient is not a sea or a river but can be a reservoir, lake or pond, each likely to have different classification standards prescribed by the national water or environmental law.

10.7 Information on the sewer drainage system

Good insight into the sewer system and origin of the sewage (also called raw influent) can help in determining and predicting the raw influent quality and quantity. Therefore general information on the sewer network must be collected. Typical information of interest regarding the sewer system is:

- Maps (GIS-based) of the sewage system;
- Type of sewer network (e.g. mixed or separated rain drainage, open transport, pressure or gravitational transport mains);
- Number of networks and length (estimated km) of connected sewer;
- Connected communities (community names) and household person equivalents (PE);

- Industrial discharge and pollution equivalents (PE);
- Rural and domestic coverage (percentage of total coverage);
- Sewage connection coverage (percentage of total inhabitants connected);
- Sewage overflow locations (bypass);
- Household water and industrial discharge (PE);
- Location of pumping stations;
- Information on the state of the sewer system, materials used and operational problems;
- Number and location of CSOs (Combined Sewer Overflows);
- Groundwater infiltration;
- Any other relevant information.

Besides general information, design information on the sewer network is also usually collected. Typical design information that can be of interest in wastewater treatment optimization studies is:

- The average and maximum sewage pumping capacity (m^3/h), also expressed as pump over-capacity (POC);
- Rain catchment area and paved surface (m^2);
- Sewage storage capacity (mm/h of rainfall or m^3);
- Weather reports indicating storm events and average mm of rainfall;
- Snow conditions and air temperature to be able to estimate snow melting water, where applicable;
- Any other relevant information.

10.7 Information on the industrial discharges

For plants with a larger fraction of sewage of an industrial origin (e.g. > 30% in terms of flow) or receiving industrial effluents with particularly difficult constituents to treat at municipal WWTP, it is of interest to get a better insight into the type of industry in question. The characteristics of industrial discharge can help explain the typical mixed influent quality and quantity observed at the inlet of the WWTP. Seasonal industrial activity such as processing agricultural products (campaign regime) and tourism, and also batch type of industrial activity (e.g. cleaning etc.), can cause typical influent dynamic patterns over the seasons, weeks or a day. It is often helpful to list the main sources of industrial discharge, together with the name and type of the industry followed by an estimation of the amount and type of wastewater (quantitative and qualitative analysis). Often, just knowledge of the general type of industrial activity (e.g. food processing, wood processing, chemical plants, etc.) will already give a general idea on the quantity and quality of the discharged industrial effluents. In doing this it is important to establish whether or not the industry in question has an (operational) on-site wastewater treatment plant (called industrial pre-treatment) and if the answer is yes, to obtain operational data for the plant including the plant performance and plant effluent quality and quantity. If this data is not readily available, literature data can be used from wastewater handbooks, although some knowledge of the industry in question is needed to make the necessary estimates (size of the industry, processes employed, type of operation, quantity of products, assortment of products, cleaning regime, water/wastewater reuse practices, operation in shifts etc.).

10.8 Short summary of the operational data

In the inventory phase, preliminary influent and effluent data is collected to give an overview and impression of the general plant performance (in fact mostly focused on the plant treatment

efficiency). Usually this type of data is readily available from periodical publications of the plant performance required by law. Based on the recorded influent and effluent quality data, the efficiency of the wastewater treatment plant is assessed. Traditionally, this is done as a percentage of (influent to effluent) pollution load reduction. Quality parameters that are normally assessed (also parameters typically required by national legislation) are Total Suspended Solids (TSS), Chemical Oxygen Demand (COD) or Biological Oxygen Demand measured over five days (BOD_5), Total Nitrogen (TN) and Total Phosphorus (TP). The EU and Croatian WWTP effluent discharge standards are presented in Chapter 1.

Once again, it is very important to properly reference the presented data and to indicate from which period the data comes from. The better the data origin is described, the more confidence the user can have in it.

Table 10.2 Example of initial plant inventory data: influent flow data (example from one of the Croatian WWTP)

Flow	Designed	Domestic	Industrial
ADWF	22,080 m^3/d	2,880 m^3/d	19,200 m^3/d
ADWF	920 m^3/h	120 m^3/h	800 m^3/h
PDWF	23,200 m^3/d	3,960 m^3/d	19,200 m^3/d
PDWF	965 m^3/h	150 m^3/h	800 m^3/h
Total sludge produced	600 m^3/h	70 m^3/h	530 m^3/h

ADWF, average dry weather flow. PDWF, peak dry weather flow

Table 10.3 Example of initial plant inventory data: raw influent quality (example from one of the Croatian WWTP)

Influent quality	Design values	Domestic sewage	Industrial sewage
BOD_5	600 mg/L	249 mg/L	889 mg/L
COD	3,000 mg/L	390 mg/L	3,416 mg/L
TSS	200 mg/L	242 mg/L	2.254 mg/L
TN	n.a.	n.a.	18 mg/L
TP		10 mg/L	4 mg/L

10.9 General WWTP classification

Typically, a design project starts with a general classification of the wastewater treatment plant regarding the level of wastewater treatment (and applied processes). This type of information is usually published in a short report introducing the design study. This helps to get a (holistic) overview of the entire system. The classification can start with an inventory of effluent legislation; the current legislation presented in a table (the MAC standards) and finish by presenting the local strategy regarding the effluent discharge standards (from the perspective of working towards EU effluent discharge standards and directions for future development).

Table 10.4 General plant classification example: assessment of the effluent quality of one of the Croatian WWTP

Parameter	MAC[(1)]	Designed value	Measured in 2010[(2)]
BOD_5	250 mg/L	20 mg/L	138 mg/L
COD	700 mg/L	n.a.	357 mg/L
TSS	10 mg/L	20 mg/L	259 mg/L
TN	n.a.	n.a.	7 mg/L
TP	250 mg/L	20 mg/L	4 mg/L

n.a. not available, (1) MAC, maximum allowed concentration, (2) Mean value

Next, the technical wastewater treatment process can be classified according to the employed level of treatment. Therefore the following categories are traditionally used in Croatian practice (presented in Figure 10.3):

(i) *Preliminary treatment* - targeting grid removal from raw sewage by screening, removal of heavy particular material (mainly sand) and floating material (grease, oil and fats);
(ii) *Primary treatment* – targeting (settleable) suspended materials and partial removal of organic matter by mechanical/physical mechanisms;
(iii) *Secondary treatment* – which, in addition to the removal provided by the primary treatment, is designed for biological removal of the remaining organic matter and possibly nitrification;
(iv) *Tertiary treatment* – by which besides the removal of suspended solids and organic matter, both nitrogen (biological nitrification and denitrification) and phosphorus (either biological or chemical) removal takes place, and
(v) *Final treatment (effluent polishing)* - by which pollutants are further removed by physical and/or chemical techniques and e.g. sand or membrane filtration. Also disinfection by ozone or chloride can be applied in the final treatment step.

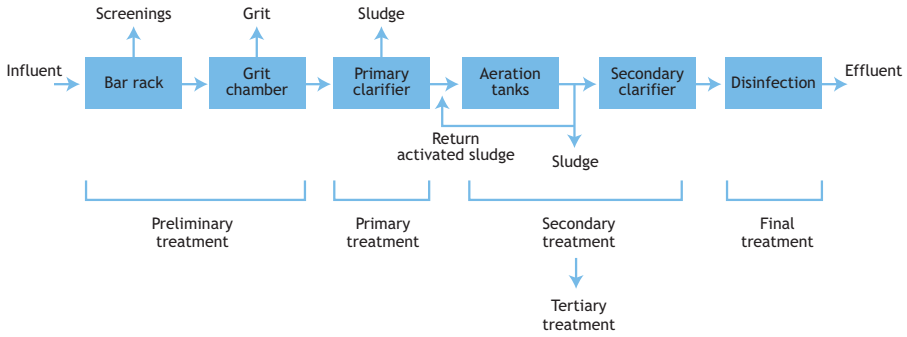

Figure 10.3 Schematic representation of a wastewater treatment process indicating preliminary, primary, secondary-tertiary treatment and final treatment.

Following the classification of the process, a quick scan of the operation can be performed. This can be done based on the following five criteria:

1. Design capacity;
2. Influent concentrations and loads (measured quantitative and qualitative parameters);
3. Treatment efficiency (% of influent pollution load);
4. Measured effluent quality;
5. Effluent quality assessment based on the prescribed effluent standards.

Based on this, the goals of the design project can be verified and further specified according to the actual operational conditions of the wastewater treatment plant. As a result, the boundary conditions of the simulation model can also be formulated and further specified. The type of questions that should be answered are: should the model include secondary treatment?, should the model include the sludge treatment?, is the plant overloaded?, does the design study very likely require plant extension?, is the plant under loaded?, and does the design study very probably require optimization of the current process units?, etc.

Table 10.5 General plant classification example: Assessment of load removal efficiency (example from one of the Croatian WWTP)

Parameter	% Load removal (influent to effluent)
BOD_5	85
COD	85
TSS	82
TN	70
TP	61

10.10 Plant operation history

The inventory is concluded by noting down the plant operational experience from the operators. Usually a range of practical problems occur during operation which can be of interest when modeling the plant. Also the plant will be maintained for which temporarily some units need to be closed down. In most treatment facilities there will be a maintenance record. Based on such a record, during the inception visit, it should be established by interviewing the operator which occurrences have changed the operation, thereby possibly influencing the design study. The collected data should be checked for any abnormal periods of plant operation.

Typical operational problems are:

- Plant overloading or under capacity of process units;
- High internal TSS, and nutrient loading from sludge dewatering;
- Uneven flow separation causing local plant over and under loading;
- Sludge bulking problems (often seasonally);
- Sludge flotation in settlers;
- Sand accumulation in process units;
- Toxic influent (pH shock, oils or gasoline);
- Typical problems related to industrial discharge (fibres, fat, salts, etc.);
- Influent peak loading (storm events and incidental peak loading);
- Foam formation on activation tanks;
- Mixer not functioning properly, resulting in (activated) sludge settling in tanks;
- On-line measurement devices that are out of order or not well calibrated;
- Pumps taken out of operation;
- Centrifuges or sludge dewatering malfunction;
- Maintenance or malfunction of aeration units;
- Odours, often the result of uncontrolled anaerobic conversions of organic matter;
- Excessive noise;
- Diluted influent conditions by sewer infiltration, etc.

10.11 Conclusions

It is quite obvious that preliminary data collection is an extremely important step in assessing the current state of the WWTP and its appropriateness for the modeling purposes from the perspective of the quality of the readily available information and data. The result of this stage will to a great extent determine what kind of investments and efforts will be needed in order to reach the minimum required quantity and quality of data necessary for meaningful execution of a modeling study and achievement of the study goals. Acquisition of trustful and reliable plant information is an absolute prerequisite for a successful modeling study.

Chapter 11

Model protocol; step 2: technical description of the plant

11.1 Introduction

In the modeling design protocol, it is recommended to make a full plant inventory including all the process units for preliminary, primary and secondary/tertiary treatment of wastewater as well as for sludge processing, storage and disposal, even if only a section of the total plant is of concern. A complete operational overview is necessary to effectively assess the plant performance and (re)design specific process units. For example, it is relevant to know whether (and how) primary treatment is applied when designing the activated sludge system as primary treatment significantly determines the influent quality at a biological treatment stage. In the following sections a methodology is presented which is based on identification of the different functional systems in a wastewater and sludge treatment process and their interaction. In doing this, often not all the process units have to be studied separately; they can be grouped together and studied as sub-systems. This can greatly simplify the plant assessment and can also reduce the amount of data needed to be collected, thereby saving time and money.

11.2 Classification of structural process units

A first inventory of the process is best carried out by listing all the process units involved in the treatment process. In this chapter lists of the most common process units are provided that can serve as a general checklist for the technical plant inventory. Therefore, a WWTP is generally divided into four distinctive sections, namely, preliminary treatment, primary treatment, the wastewater treatment line, and the sludge treatment line.

Preliminary treatment involves the mechanical treatment of the raw wastewater. Typically, it can be assumed that only large objects, debris, sand and grease are removed by the mechanical treatment step and that the biochemical wastewater composition is not changed. Therefore, preliminary treatment is often not included in the model calculations.

Primary treatment is studied as a separate sub-system, often together with primary sludge gravitational thickening (especially when the overflow water, also called supernatant, from the thickener is returned to the primary settler). The functioning of primary settling strongly determines both the sludge treatment line (because of the quality and quantity of primary sludge) and the wastewater treatment line (because of the quality of the so-called settled influent). Modeling the primary settling process is often difficult and inaccurate. In many cases an average settling removal efficiency is assumed. It is always advised to investigate effluent of primary settler(s) and not to rely on model calculations for the settled influent quality because the range of removal efficiency of particularly COD and TSS (and sometimes of N and P) can vary

significantly. Primary treatment removal efficiency can be found in any good wastewater treatment handbook. Note that primary settlers are sometimes also called clarifiers.

The wastewater treatment line deals with the inflow of the activated sludge plant (called raw influent for systems without primary treatment or settled influent when primary treatment is applied). It most often includes the activated sludge (biological) reactors, return activated sludge (RAS) flow, other internal recycling flows, secondary settlers (also called clarifiers) , waste activated sludge (WAS) and effluent flows. The conventional sludge treatment line usually consists of the thickened primary sludge flow, the waste sludge flow (WAS), thickening and dewatering units, digestion units, drying and storage, and all the technological (dirty) water flows.

Figure 11.1 presents a schematic overview of the four sub-systems that can be identified in a conventional municipal WWTP. In the following sections of this chapter, for each of the above categories a list of commonly applied process units is presented. These lists can serve as checklists for collecting and summarizing plant information.

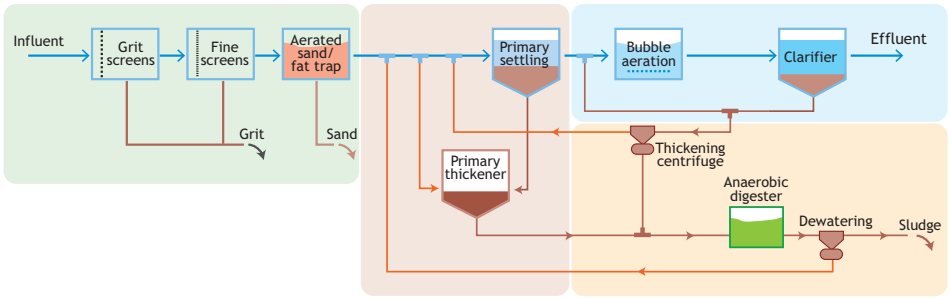

Figure 11.1 Division of a WWTP processes in four categories: preliminary treatment (green), primary treatment (yellow), wastewater treatment line (secondary and/or tertiary treatment - blue), and the sludge treatment line including thickening, digestion, sludge drying and storage (red).

11.2.1 Preliminary treatment

For the description of preliminary treatment the following process and technical units are typically of relevance (this list can be completed or extended, based on the actual plant design):

- **Sewage inflow channel**
  - type of flow measurement device
  - online pH and conductivity measurements
- **Coarse (automatic) screens and grit removal**
  - number of screens
  - screen opening width (mm)
  - maximum hydraulic capacity (m^3/h)
- **Fine (automatic) screens or rotating drum screens**
  - number of screens
  - screen opening width or mash size (mm)
  - maximum hydraulic capacity (m^3/h)

- **Sand and grease removal tanks**
  - number of (parallel) tanks
  - type of unit
  - aerated or non-aerated
  - tank dimensions (depth, length, width, surface, volume)
  - maximum hydraulic flow capacity (m^3/h)
  - hydraulic retention time (HRT, min)
- **Sewage bypass or overflow facility**
  - sewage bypass overflow limit (m^3/h)
- **Storm water retention tank**
  - tank dimensions (depth, length and width)
  - storm water pump capacity (m^3/h)
- **Raw influent pumps**
  - type of influent pumps
  - number of influent pumps
  - influent pump capacity (m^3/h)
  - installed spare capacity (m^3/h)
  - pump arrangement
- **Influent (24-hour flow composite) quality measurement device (refrigerated)**
  - measuring frequency (e.g. daily, every other day or weekly)
  - number of samples per day (depending on measurement carousel unit)
- **Raw influent flow division works (towards primary or secondary treatment)**
  - number of parallel lanes
  - flow division over the parallel lanes (in % of the total flow)

11.2.2 Primary treatment

For the description of primary treatment, the following process units and technical information are typically of relevance (this list can be completed or extended, based on the actual plant design):

- **Primary settling tanks**
  - number of (parallel) tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - for round tanks: slope of bottom (% decline) and side depth (m)
  - maximum hydraulic flow capacity (m^3/h)
  - number of primary sludge pumps (primary sludge underflow)
  - installed primary sludge pump capacity (m^3/h)
  - hydraulic retention time (HRT, min)
  - weir overflow rate (WOR, $m^3/m^1.h$)
  - surface overflow rate (SOR, $m^3/m^2.h$)
- **Primary sludge thickening tanks**
  - number of (parallel) tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - slope of bottom (% decline) and side depth (m)
  - maximum hydraulic flow capacity (m^3/h)
  - number of thickened sludge pumps (primary thickening underflow)
  - installed thickened sludge pump capacity (m^3/h)
  - hydraulic retention time (HRT, min)

- **Settled influent (24-hour flow composite) quality measurement device (refrigerated)**
  - measuring frequency (e.g. daily, every other day or weekly)
  - number of samples over the day (depending on measurement carousel)
- **Settled influent flow division works (towards biological tanks)**
  - number of parallel lanes
  - flow division over the parallel lanes

11.2.3 The wastewater treatment line

For the description of the water line, the following process units and technical information are typically of relevance (this list can be completed or extended, based on the actual plant design):

- **Number of parallel activated sludge lines**
  - type of activated sludge process configuration (e.g. UCT, Phoredox, A2O)
- **Anaerobic, aerobic or anoxic selector tank (influent mixed with return activated sludge)**
  - number of parallel tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - maximum hydraulic flow capacity (m^3/h)
  - aeration type (e.g. surface aeration or diffuse air aeration)
  - hydraulic retention time (HRT, min)
- **Anaerobic tank**
  - number of parallel tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - maximum hydraulic flow capacity (m^3/h)
  - hydraulic retention time (HRT, min)
  - anaerobic sludge retention time (SRT) or sludge age (d^{-1})
- **Anoxic tank**
  - number of parallel tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - maximum hydraulic flow capacity (m^3/h)
  - hydraulic retention time (HRT, min)
  - anoxic sludge retention time (SRT) or sludge age (d^{-1})
- **Aerobic tank or aeration tank**
  - number of parallel tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - maximum hydraulic flow capacity (m^3/h)
  - aeration type (e.g. surface aeration or diffuse air aeration)
  - number of installed (blower or surface aeration) units
  - aeration capacity (Nm^3/h or kWh)
  - spare aeration capacity (Nm^3/h or kWh)
  - hydraulic retention time (HRT, min)
  - aerobic sludge retention time (SRT) or sludge age (d^{-1})
- **Membrane bioreactor**
  - number of parallel tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - maximum hydraulic flow capacity – membrane flux (m^3/h)
  - aeration type (e.g. diffuse air aeration)
  - number of installed blower units
  - aeration capacity (Nm^3/h)
  - spare aeration capacity (Nm^3/h)

- **De-aeration tank**
  - number of parallel tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - maximum hydraulic flow capacity (m^3/h)
- **Activated sludge flow division works**
  - number of parallel lanes
  - flow division over the parallel lanes towards secondary settling tanks
- **Secondary settling tanks or secondary clarifiers**
  - number of (parallel) tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - for round tanks: slope of bottom (% decline) and side depth (m)
  - maximum hydraulic flow capacity (m^3/h)
  - number of return activated sludge pumps (RAS underflow)
  - type of return activated sludge pumps
  - number of return activated sludge pumps
  - installed return activated sludge pump capacity (m^3/h)
  - hydraulic retention time (HRT, min)

11.2.4 The sludge treatment line

For the description of the sludge line, the following process units and technical information are typically of relevance (this list can be completed or extended, based on the actual plant design):

- **Gravity sludge thickening tanks - gravity thickeners (primary, WAS or combined sludge)**
  - number of (parallel) tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - for round tanks: slope of bottom (% decline) and side depth (m)
  - maximum hydraulic flow capacity (m^3/h)
  - number of thickened sludge pumps (thickening underflow)
  - installed thickened sludge pump capacity (m^3/h)
- **Mechanical sludge thickening (centrifuge or filter press)**
  - number of mechanical thickening units
  - hydraulic unit capacity (m^3/h)
  - installed concentrate/filtrate pump capacity (m^3/h)
- **Aerobic sludge digestion/stabilization**
  - number of (parallel) tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - maximum hydraulic flow capacity (m^3/h)
  - aeration type (e.g. surface aeration or diffuse air aeration)
  - number of installed aeration units (blowers or surface aeration units)
  - aeration capacity (Nm^3/h or kWh)
  - spare aeration capacity (Nm^3/h or kWh)
  - number of digested sludge pumps
  - digested sludge pump capacity (m^3/h)
- **Anaerobic sludge digestion**
  - number of (parallel) tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - for round tanks: slope of bottom (% decline) and side depth (m)
  - volume of the gas holdup (m^3)
  - gas flow and concentration measuring device (CH_4 and CO_2)

- maximum hydraulic capacity (m^3/h)
- number of digested sludge pumps
- digested sludge pump capacity (m^3/h)
- Mechanical sludge dewatering (centrifuge or filter press)
- number of mechanical dewatering units
- hydraulic unit capacity (m^3/h)
- installed concentrate/filtrate pump capacity (m^3/h)

- **Chemical sludge stabilization**
  - type of chemical applied
  - concentration of chemical applied
  - flow of chemical applied
- **Sludge storage**
  - number of (parallel) tanks
  - tank dimensions (depth, length, width, diameter, volume, surface)
  - for round tanks: slope of bottom (% decline) and side depth (m)
  - number of dewatered sludge pumps
  - dewatered sludge pump capacity (m^3/h)

In primary treatment, the wastewater treatment line and the sludge treatment line, chemicals are often used. Chemical storage, preparation and dosing equipment should also be listed as well as the type of chemicals and dosing concentrations used.

11.2.5 Process flow diagrams (PFD)

In a design project, various types of PFD are made and used simultaneously, each PFD providing different information such as plant location, relative position of process units, connections between different units, structure of process units, mechanical and electrical units, flows, valve shutters, sample collection, on-line measurements etc. The main function of the process flow diagram is capturing the process setup in a single overview. The PFD is the main work document at the starting phase of a model design study; it serves as the basis for establishment of the model hydraulic flow scheme. In the PFD all flows and process units need to be numbered and named. In practice, designers and operators use different (logical) names or numbers for identical process units; to avoid miscommunication, possible synonyms should also be listed. During the entire project design trajectory the PFD will be used for communication with all parties involved in the modeling design exercises such as managers, engineers, technicians, operators and laboratory personnel.

As mentioned earlier there are different types of PFD, each serving a specific purpose:

- **Plant overview flow diagram or plant layout scheme** - this type of PFD represents the actual situation and can serve as a map, indicating the different unit locations. A 'bird's eye' overview drawing or aerial photograph can typically be used for this type of diagram. It is a diagram that can be used to point out measurement locations and can help by identifying the process flows during the first plant inventory. An example of such a PFD is shown in Figure 11.2.
- **Process mass flow diagram or block process scheme** - this type of PFD is typically used by designers to indicate the mass flows over the different process units. This type of PDF is the most simplified (containing the least detailed information), however, also the most close to a model set-up. An example of such a PFD is shown in Figure 11.3.

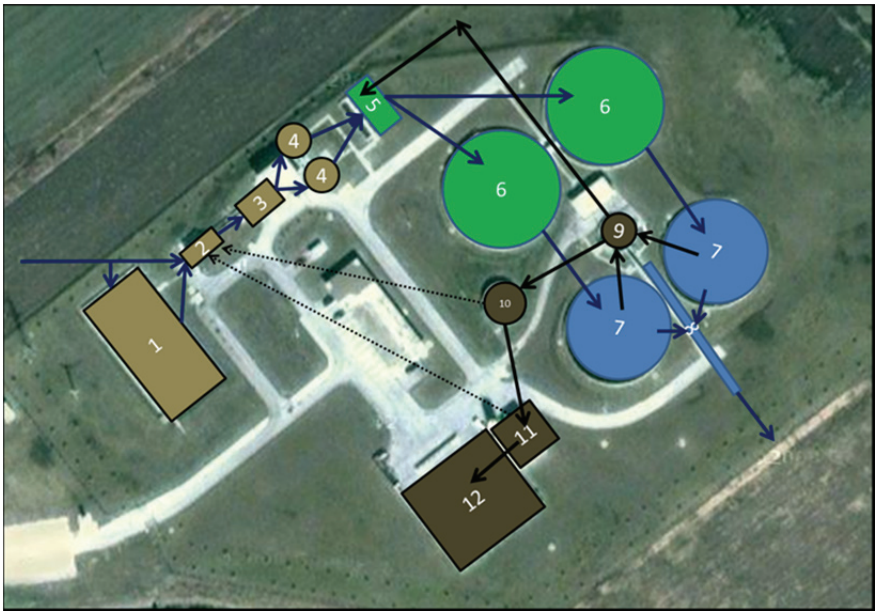

Figure 11.2 Plant overview flow diagram or plant layout scheme based on an aerial photograph of the WWTP Vinkovci (Google Earth).

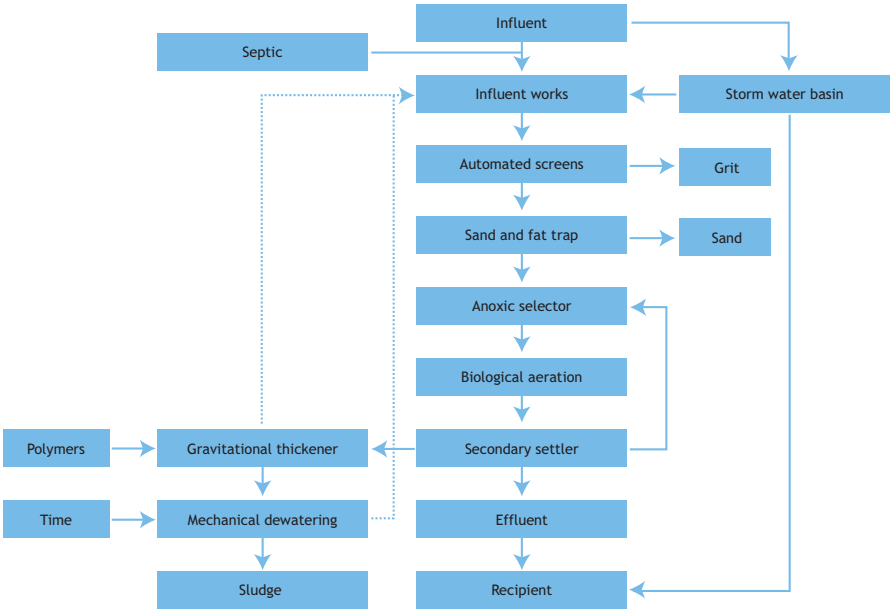

Figure 11.3 Process mass flow diagram or block process scheme indicating the main process units and flows (WWTP Vinkovci)

- **Operational process flow diagram or Process Instrumentation Diagram (PID)** - this type of PFD is often made finalizing the build of the plant indicating all the technical units (mechanical installations, pumps, shutters, valves and indicators). This type of PFD is often represented on the main process operation control boards, switchboards and Supervisory Control And Data Acquisition (SCADA) systems. These types of PFD are the most detailed and most complex to make, however, nowadays they are available for the most modern plants (Figure 11.4).

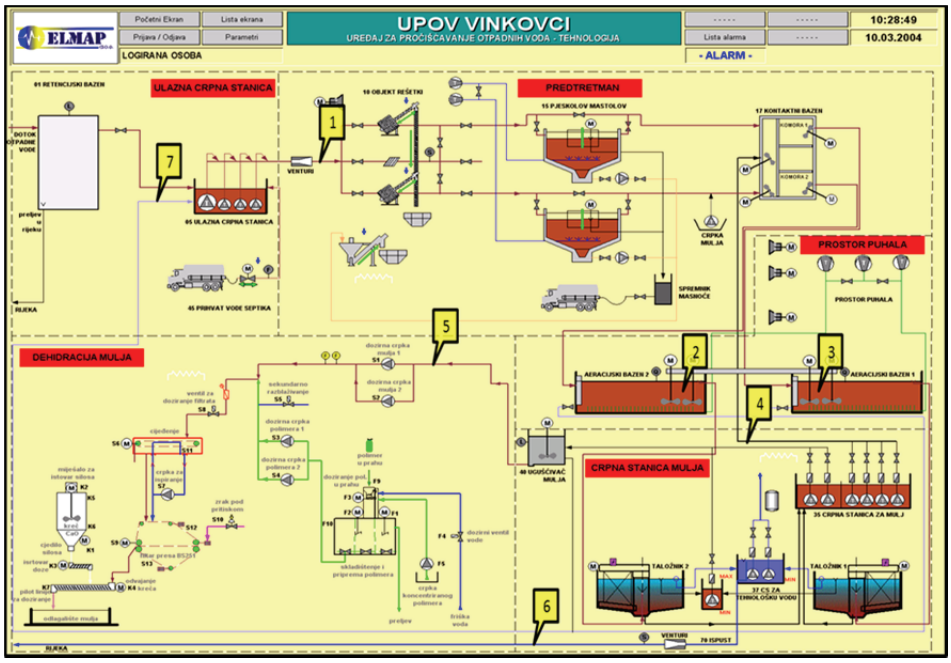

Figure 11.4 Operational process flow diagram or Process Instrumentation Diagram (PID) combined with structural and mechanical information (WWTP Vinkovci). Measurement and sampling points are shown in yellow.

- **Model process flow diagram or model hydraulic scheme** - this type of PFD represents the process model indicating only the modelled process units and flows (Figure 11.5). It is an abstract representation of the plant and not typically suited for communication with the lab staff and operators, as the reality may often be very different to the model scheme. Therefore it is strongly advised to start by constructing a (recognizable) overview diagram and from there the mass flow diagram in preparation for the actual model setup.

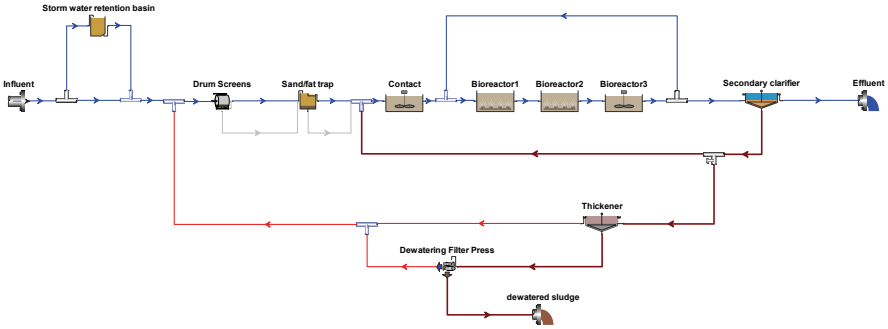

Figure 11.5 Model process flow diagram or model hydraulic scheme indicating the modelled process units and flows (WWTP Vinkovci). This diagram is the combined result of the structural, mechanical and technological PFDs previously presented. The diagram is a simplified representation of reality only giving information on those processes relevant as described in the project goals. Usually all measurements of the measurement campaign should relate to one of the (combined) flows in the model diagram, or otherwise not have been measured.

11.3 Conclusions

The first step of the system description directly following the inventory phase is a description of the entire wastewater treatment plant. Therefore, the plant is divided into four main distinguishable sub-systems; preliminary treatment, primary treatment, the wastewater treatment line and the sludge treatment line. All the relevant process units of the treatment plant are described based on an information checklist summarizing the key design parameters of each process unit. This information is used to construct the process flow diagrams (PFDs) that serve as the main means of communication between all the involved parties (modellers, designers, operators, lab personnel, managers) during the modeling design study. Typically, four types of diagrams are used: a plant overview flow diagram or plant layout scheme indicating the actual plant units and their relative position used for pointing out measurement locations; the process mass flow diagram or block process scheme describing the system from a technological point of view the operational process flow diagram or process instrumentation diagram (PID) which is often used by the operators indicating key operational process units and controls, and finally the model process flow diagram or model hydraulic scheme which is often the most abstract and simplified.

Chapter 12

Model protocol; step 3: historical plant data acquisition

12.1 Classification of process flows

Like the process units, the process flows of the wastewater treatment plant can also be classified. This is done based on the expected concentrations in the flows (mainly the total suspended solids, TSS). The classification is useful for the designer to get an initial idea of the expected flow quality especially when evaluating the model simulation results. The laboratory also uses a similar flow quality classification to anticipate the expected (solids) concentration range and prepare the samples for analytical measurement by the appropriate dilution.

The five main classified flows are:

- Influent flows under which,
  - sewage before screening and sand removal
  - raw influent (after screening and sand removal)
  - settled influent (after primary settling)
- Activated sludge flows under which,
  - activated sludge in the reactors,
  - internal anaerobic activated sludge recycling (for biological phosphorus removal),
  - internal anoxic activated sludge recycling (for denitrification),
  - internal aerobic activated sludge recycling (for Carousel©-type aeration tanks)
  - return activated sludge (RAS),
  - waste activated sludge (WAS).
- Effluent flow
- Sludge flows under which,
  - primary sludge,
  - thickened sludge (WAS, primary sludge or combined),
  - digested sludge,
  - dewatered sludge,
  - (chemical) stabilized sludge.
- Dirty water flows (internal loads)
  - overflow water from sludge thickeners,
  - concentrate from mechanical sludge thickening or dewatering (centrifuge or cyclone),
  - filtrate from mechanical sludge thickening or dewatering (filter press),
  - overflow water from digesters,
  - overflow water from sludge drying.

Table 12.1 Example of TSS concentrations and dry mass in typical process flows

	TSS		DS	
Flow	Low mg/L	High mg/L	Low % of TSS	High % of TSS
Sewage	250	350	-	-
Raw Influent	200	250	-	-
Settled Influent	60	80	-	-
Activated sludge	2,500	5,500	0.3	0.6
Returnactivated sludge	5,000	12,000	0.5	1.2
Waste activated sludge	5,000	12,000	0.5	1.2
Effluent	5	20	0.5	2.0
Dirty water sludge line	500	3,000	0.5	3.0
Primary sludge	7,000	10,000	0.7	0.9
Gravitational Thickened sludge	12,000	20,000	1.2	1.8
Mechanically Thickened sludge	15,000	36,000	1.5	3.6
Digested sludge	30,000	40,000	3.0	4.0
Dewatered sludge	200,000	300,000	20.0	30.0
Chemical stabilized sludge	250,000	350,000	25.0	35.0

TSS: total suspended solids, DS: dry solids

12.2 Locating plant measurements

After summarizing the process units and flows in a PFD, this diagram can be used to indicate all the measurement points; all the existing measurements used for daily operation and control and also new measurements dedicated to the model design study. Most modern wastewater treatment plants have an automated data acquisition system; typically a SCADA system (a PLC-based Supervisory Control And Data Acquisition) that monitors and controls the automated processes at the wastewater treatment plant.

Typical operational data logged by the SCADA system includes:

- Pump flows;
- Pump operating time;
- energy consumption of mechanical devices;
- Blower capacity;
- Online pH measurements;
- Conductivity;
- Temperature;
- Dissolved oxygen;
- Analytical and REDOX electrode readings;
- (Bio) gas flows;
- Automated methane and CO_2 readings;
- Chemical dosing;
- Etc.

Recording of this data is done digitally and also often presented on the main control cabinet in the control room of the plant-administration building. When this data is used in a modeling study, the exact measurement location has to be identified. A PFD is very useful to indicate the measurement location of the SCADA measurements.

Different additional measurements can be taken at different locations for daily control of the plant operation. Normally these are acquired by the lab and operational personnel and are often controlled at the laboratory of the wastewater treatment facility. Most laboratories work according to prescribed regulations (e.g. Limit Values of Hazardous and Other Substances in Wastewater, Official Gazette, number: 94/08). The WWTP laboratory results can also be tested on a regular basis by a third-party authorized laboratory. When conducting a modeling study, these third-party laboratory plant measurements are also a useful (and reliable) source of information. Often (spreadsheet) reports are made on quality control of the (regulatory) effluent and influent measurements and other analytical measurements used for daily plant operation. This data can be combined with operational SCADA data in a database environment.

Table 12.2 shows an overview of typical (daily) plant measurements that are performed by the plant staff. The measurement table is presented in a matrix notation; sample points (flow or tank reactor) in the columns and a listing of all possible analytical measurements in the rows. Information on the sample type is also indicated (on-line measurement, 24-hour composite or grab sample). An overview of typical samples taken for daily plant operation is presented in Table 12.2.

Table 12.2 Typical measurements performed for daily operational plant control by plant staff

Description	Code	Unit	Raw sewage	Raw sewage	Settled sewage	Effluent	Effluent	Aeration tanks	Aeration tanks	Settled sludge	Dewatered sludge	Digester	Digester	Waste sludge
		Type of the sample	24-hour composite	Online	24-hour composite	24-hour composite	Online	Online	Grab sampled	Grab sampled	Grab sampled	Online	Grab sampled	Grab sampled
pH	pH	-		1			1							
Conductivity	E	µS/cm		1			1							
Temperature	T	°C		1			1							
Methane and CO_2 gas composition	CH4	vol%										1		
Hydrogen sulphite	H2S	ppm										1		
Carbon dioxide	CO2	vol%										1		
Dissolved oxygen	DO	mgO_2/L						1						
Total suspended solids	TSS	mg/L	1		1	1			1	1				
Inorganic suspended solids (ash)	ASH	% of TSS							1	1				
Dry matter	DM	mg/L									1		1	1
Inorganic fraction of dry mass	IM	% of DM									1		1	1
Total phosphorus	TP	mgP/L	1		1	1								
Ortho phosphate	PO_4	mgP/L	1			1								
Total COD	TCOD	mgCOD/L	1		1	1								
BOD_5	BOD	mgBOD/L	1		1	1								
Total Kjeldalh nitrogen	TKN	mgN/L	1		1	1								
Ammonium	NH_4	mgN/L	1			1								
Nitrate and nitrite	NO_x	mgN/L				1								

The additional samples collected at the site for operation control can either be automated (on-line) data, manually collected by 24-hour flow composite measurement devices (often refrigerated) or by incidental grab sampling. When this data is used in the model design study it should be indicated precisely on the process flow diagram. Also, the method of sampling and analytical measurement should be known and therefore, it is advisable to be present and to visually inspect the daily grab sampling procedure and analytical measurements during one of the plant visits.

12.3 Typical daily influent measurements

Typically, the influent flow measurement device is located immediately before or after preliminary treatment (grid and sand removal). Influent flow measurement is often carried out by a 'Venturi' damper installed in a concrete drain (channel). In the Venturi channel, the water height is measured by means of an echo probe, mounted in front of the Venturi damper and above the water level in the measuring drain. It is also possible to measure the influent flow in a (filled) pipe by electromagnetic or Doppler-type flow meters. Sometimes the flow over the wastewater treatment plant is measured both in the influent and the effluent flow; an electromagnetic flow meter in the influent (pipe) and a Venturi measurement in the effluent channel. In general (when properly calibrated), a Venturi measurement is more accurate than an electromagnetic or Doppler flow measurement device. Therefore, it is recommended to use Venturi measurements when available. In some plants, the influent flow is measured before the influent storm flow bypass. This will affect the yearly average flow that is measured (more rain water is included that actually passes through the plant). For this reason it is also recommended to use effluent flow measurements, as this measurement represents the actual flow that has passed through the wastewater treatment system. Online measurements of pH, conductivity and water temperature are often also performed at the location of the influent flow measurement. These measurements are used in practice to detect abnormal influent conditions indicating possible toxic waste. For larger wastewater treatment plants, 24-hour flow composite measurements (refrigerated) at the location of the raw influent are often required by law. Here the influent composition is sampled and analysed according to typical influent parameters (TSS, BOD_5, COD, TN, NH_4, TP, PO_4). Typically, an automatic sampling device is installed next to the measuring drain in a watertight cabinet. The automatic wastewater sampler has multiple vessels (e.g. 4, 12 or 24 sample collecting vessels in a carousel) for keeping the 24-hour composite wastewater sample. The vessels are filled (one by one) based on a timer-controlled automatic dosing pump which pumps wastewater samples from the measuring drain. The fully automated procedure is done proportionally to the measured flow (the amount of samples is larger when the influent flow is high). Therefore, the pump needs to receive an electronic signal from an influent flow-measuring device. At the end of each collecting day, the composite samples are taken from the (refrigerated) cabinet for the needs of laboratory testing of the wastewater characteristics, with the purpose of determining the input parameters and optimization of the technological process of wastewater purification.

12.4 Typical daily effluent measurements

The flow over the wastewater treatment plant is often measured in the effluent channel, mostly by a Venturi damper and echo-probe measurement. A 24-hour flow composite measurement (refrigerated) at the location of the effluent is required by law for all WWTPs. These measurements are often taken at the location of the effluent channel towards the recipient. The effluent composition is sampled and analysed according to regulatory parameters (TSS, BOD_5, COD, TN, TP), depending on the local legislation.

Chapter 13

Model protocol; step 4: activated sludge plant assessment

13.1 Introduction

For plant operational assessment and model design, historical data, as well as additional plant data is needed. This data is:

- Influent characterization measurements (see the chapter on influent characterization)
  - raw influent characterization
  - settled influent characterization
- Mass balance measurements (see this chapter)
  - to check the flow data and calculate missing flows
  - to check sludge production numbers and verify the SRT
  - to calculate key conversions of nitrogen and COD
- Activated sludge characterization measurements (see this chapter)
  - used for model validation
  - used to support mass balance and SRT calculations
  - used to support influent characterization
- Effluent characterization (see the chapter on influent characterization)
  - used to support influent characterization
- All other additional samples (based on Table 12.2)
  - standard measurements that are missing
  - historical measurements to be verified

Most of this specific data is not (all) available from daily sampling programs (Table 12.2) and therefore needs to be measured in an additional measurement campaign. Performing additional analytical measurements is logistically the most difficult task in a model design study; it is the most time-consuming in preparation and execution and also requires a considerable financial budget for lab measurements, sample transportation and manpower. It is therefore essential to make an efficient and realistic plan for collecting additional plant data. The first concern is minimizing the amount of measured data while at the same time to be sure of collecting all the necessary information to be able to perform the study. More information on sampling is given in Chapter 15. The chapter on influent characterization presents a table (Table 15.3) in which the additional influent (and effluent) measurements are required. This chapter explains which measurements are needed for characterization of the activated sludge. It also explains how to determine which measurements are required to check the flow data and calculate missing flows, how to check the sludge production numbers and verify the SRT and finally, how to calculate the key conversions of nitrogen and COD from mass balance calculations.

13.2 Activated sludge characterization

Traditionally, in wastewater treatment practice, the activated sludge (or biomass) composition is not measured. The main reason is that traditional design does not predict the activated sludge composition and therefore measurement of this information is not useful. However, when applying activated sludge modeling this information becomes useful because activated sludge models make predictions on the MLSS and biomass composition. Therefore activated sludge measurements can be used to validate the model calculations. Moreover, the composition of the activated sludge is useful information to calculate key plant performance indicators (e.g. SRT). In the model study, measurement of the activated sludge composition serves three main purposes:

1. Based on the activated sludge composition the model results can be validated; the activated sludge model predicts the activated sludge (MLSS) composition.
2. The activated sludge composition is used as a check on the calculated sludge retention time (SRT) and to verify the sludge production being key operational parameters.
3. The COD, TN and TP mass balances for the wastewater treatment plant can be derived from the activated sludge composition. Therefore, the COD:TSS, COD:N and COD:P composition ratios are used. These will be constant in the water line (e.g. tank reactors, WAS and RAS flows) within the time span equal to 2-3 times the actual SRT. When only the TSS is known of a measured flow (e.g. the WAS or RAS flow), e.g. COD can be calculated based on the TSS:COD ratio (this approach also applies to N and P). Thus, the opportunity arises to calculate all COD, N and P mass balances for the entire plant.

The activated sludge is measured based on measurements of the total mixed liquor suspended solids. Therefore one large MLSS sample is taken from which all measurements are analyzed (TSS, VSS, TCOD, TKN, TP). From the same sample, the filtrate is also analyzed thereby obtaining the soluble fractions. By subtracting both measurements, the particulate composition of the activated sludge can be calculated. It is important to note that all samples need to be taken from one grab sample to get the most accurate relative composition ratio (e.g. COD_X/VSS, TKN/COD etc.). In general, it is recommended that this measurement is performed on a monthly basis in order to provide a better process insight.

13.2.1 Activated sludge (MLSS) fractions

Table 13.1 shows an overview of the biomass composition expressed by its typical component ratios for the 5 project case studies. The biomass composition can be assumed to be more or less constant within a period of 2-3 times the SRT. This can be understood from the point of view of the SRT being the sludge mixing time. When an operational change is applied to the activated sludge process, as a rule of thumb, it will take 3 times the sludge retention (mixing) time for the activated sludge composition to change to a new dynamic steady state. Therefore the activated sludge composition is sensitive to how the plant is operated and especially to the SRT operation, but will only change slowly over longer periods of time. For example, the ash content (ISS) of the activated sludge typically increases with increasing SRT and vice versa. For systems with a short SRT (e.g. 3 days), the ash content of the sludge is typically in the range of 12-15% whereas for systems with a longer SRT (15-20 days) the ash content is typically in the range of 18-23%. However, this also depends on the influent ISS concentration. The influent composition and dosage of chemicals (metal-salts) for P removal will also influence the ISS activated sludge composition. For example, increasing inorganic suspended solids (ISS) in the influent will lead to a higher ash fraction of the activated sludge.

Table 13.1 Activated sludge composition and typical sludge composition ratios determined in 5 project case studies in 2011

Particulate components		Unit	Belišće	Čakovec	Varaždin	Vinkovci	Zagreb
TSS	Total suspended solids	mg/L	6,260	3,620	3,145	5,192	3,549
VSS	Volatile suspended solids (organic fraction)	mg/L	4,400	2,817	2,682	3,417	2,774
ISS	Inorganic fraction of total suspended solids	mg/L	1,860	803	464	1,775	775
CODx	Particulate COD	mgCOD/L	5,530	3,662	3,646	4,485	3,516
TKNx	Particulate total Kjeldahl nitrogen	mgN/L	291	141	255	250	248
TPx	Particulate phosphorus	mgP/L	26	65	44	39	48
Total (MLSS)							
TCOD	Total chemical oxygen demand	mgCOD/L	6,200	3,689	3,668	4,489	3,532
TKN	Total Kjeldahl nitrogen	mgN/L	291	143	262	250	249
TP	Total phosphorus	mgP/L	27	68	45	39	50
Soluble components							
CODgf	COD filtered (includes colloidal matter)	mgCOD/L	670	27	22	29	16
NH4	Ammonium	mgN/L	0.3	2.1	6.9	0.4	0.4
PO4	Ortho-phosphate	mgP/L	0.0	3.5	1.1	0.8	1.7
Composition							
Ash	Ash fraction of TSS	% of TSS	29.7	22.2	14.7	34.2	21.8
CODx/VSS	Organic weight to COD ratio	gCOD/gVSS	1.26	1.30	1.37	1.31	1.27
VSS/TSS	Organic fraction of total weight	gVSS/gTSS	0.70	0.78	0.85	0.66	0.78
N/VSS	Contribution of N to VSS	gN/gVSS	0.066	0.050	0.098	0.073	0.090
P/VSS	Contribution of P to VSS	gP/gVSS	0.006	0.023	0.017	0.011	0.017
N/COD	Contribution of N to COD	gN/gCOD	0.047	0.065	0.071	0.055	0.070
P/COD	Contribution of P to COD	gP/gCOD	0.004	0.030	0.012	0.008	0.014
Org. N/P	Organic composition	gN/gP	10.97	2.17	5.83	6.62	5.15

In a similar way, the influent particulate COD/VSS ratio determines the activated sludge (MLSS) COD/VSS ratio. In the model, the influent particulate COD/VSS ratio is used as a calibration parameter to fit the activated sludge concentration in the model to the actual measured MLSS in the reactors. More specifically, by adjusting different values for influent biodegradable and unbiodegradable COD/VSS (two separate parameters in the model), both influent TSS and MLSS in the tanks can be fitted simultaneously. Typically, the nitrogen content of activated sludge is in the range of 5-8% (COD:N mass-based). The phosphorus content of activated sludge is typically in the range of 1-3% (COD:P, mass-based). However, for a bio-P activated sludge process, the amount of phosphate in the sludge is higher (up to 5%) as the result of cell internal poly-phosphate storage.

13.2.2 Identifying relevant mass balances

Flow balance measurements are used to calculate all the unknown flows in the system. Additional mass balance measurements are used to check the measured (and calculated) flows for accuracy. This is an important step in the model design study for all processes depending on an accurate description of (mass) flows. This is also because mass balance measurements are used to calculate key operational parameters based on which the process can be assessed. A method by which all relevant flows and mass balances can be identified in one structured overview is explained in the following sections.

The first step is to identify all the relevant sub-systems in the overall process flow diagram to be studied. Therefore, the general process unit classification previously proposed can be used as a starting point (as in the following example). Generally, the mass balanced areas of the plant (Figure 13.1) correspond with the main areas of interest, also included in the model. Based on specific project goals (e.g. also assessing the operation of the anaerobic digester), more subsystems can be distinguished (e.g. Figure 13.1, the mass balance over the anaerobic digester should also be measured leading to two additional measurement points). In our example (Figure

13.1) three sub-systems are studied (3 mass balances): the primary treatment, the water line, and the sludge line. Moreover, the combination of these three mass balance results in the overall wastewater treatment plant mass balance adding up a total of four mass balances to be calculated.

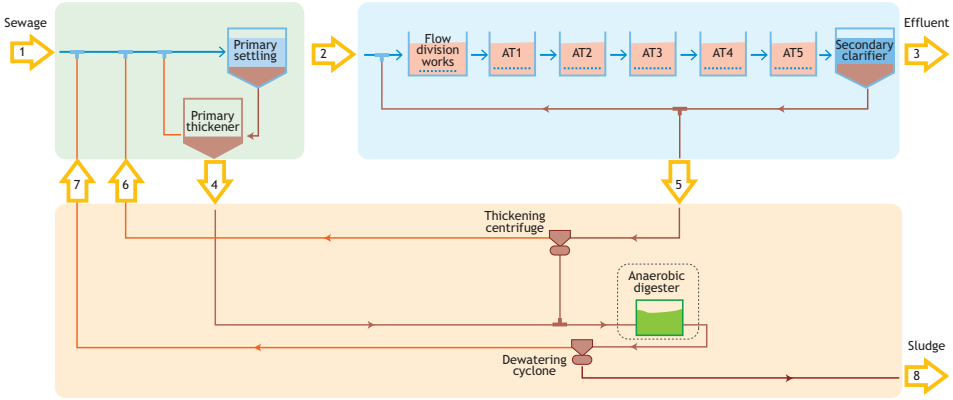

Figure 13.1 Simplified PFD indicating mass balances for the main process units (WWTP Zagreb). The yellow numbered arrows indicate in- and outflows of the sub-systems (primary treatment, water line and sludge line).

Each of the identified mass balances has incoming and outgoing flows. Some of the mass balances share mutual flows. In this example, there are in total eight flows that need to be identified to calculate all the flow and mass balances (indicated by yellow arrows and numbers 1 to 8). To be able to perform all our calculi a matrix system is defined according to Table 13.2.

Table 13.2 Matrix of flow balances (example WWTP Zagreb). Note: In-Out=0

		Flow balance →				
	Flow ↓	Primary treatment	Sludge line	Water line		Overall WWTP
Q1	Influent	+Q1	-	-	→	+Q1
Q2	Settled influent	-Q2	-	+Q2		-
Q3	Effluent	-	-	-Q3	→	-Q3
Q4	Primary sludge	-Q4	+Q4	-		-
Q5	Waste activated sludge	-	+Q5	-Q5		-
Q6	Thickening supernatant	+Q6	-Q6	-		-
Q7	Dewatering filtrate	+Q7	-Q7	-		-
Q8	Sludge	-	-Q8	-	→	-Q8
	SUM OF BALANCE	0	0	0		0

In the four vertical columns, all the identified mass balances are presented. In the eighth horizontal rows the flows are re-presented. It should be noticed that all the empty spaces in the matrix represent zeros. It should also be noticed that when summarizing the balances horizontally the result is the overall WWTP balance (influent – effluent – sludge = zero).

13.2.3 Evaluation of the process flow balances

The first step in the data evaluation is closing the flow (water) balance. Water is conserved (not disappearing) and therefore should always be fully accountable. Some wastewater will evaporate, however in practice this is negligible. Inconsistency in the flow balance indicates an error in (one or more) flow measurement(s) or it can indicate a misinterpretation of the process (thus an incorrect connection or unit in the PFD). A process description that is incorrect (e.g. misconnected or redirected flow) results in a gross error in the flow balance that needs to be corrected before data can be balanced. Calculation of the flow balance can be carried out in a spreadsheet environment (e.g. MS Excel) based on the process flow balance matrix (Table 13.2).

The flow balance matrix is a mathematical description of the PFD (Figure 13.1) and shows the flow direction (inflow is 1 and outflow is -1). The columns in the balance matrix (Table 13.2, 4 balances) represent the balances over the process units, and the rows are the in and outgoing water flows (Q1 to Q8). A vertical summation of the balances provides the possibility to check the system of flows for consistency. When no errors are present in the flow data, the flow balances should close, meaning the vertical summation should add up to zero. Horizontally summarized the 3 internal balances (Figure 13.2) result in the overall WWTP balance. Mathematically the balances can be written down according to the following for equations:

The flow balance over the primary treatment is given by:

$$Q1 - Q2 - Q4 + Q6 + Q7 \approx 0 \tag{13.1}$$

The flow balance over the sludge line is given by:

$$Q4 + Q5 - Q6 - Q7 - Q8 \approx 0 \tag{13.2}$$

The flow balance over the water line is given by:

$$Q2 - Q3 - Q5 \approx 0 \tag{13.3}$$

The flow balance over the overall WWTP is given by:

$$Q1 - Q3 - Q8 \approx 0 \tag{13.4}$$

13.2.4 Check on the process flows; introducing the TP balance

Similar to the flow balances, the conserved mass balances can also be calculated. Like the flow balance, the mass balances should add up to zero when no conversions are taking place. This is the case for the total phosphorus. All phosphorus entering the plant will also leave in one of the flows and no TP disappears to the gaseous phase. However, due to (small) measurement errors, balances often have residuals (the error in the sum of the balances).

The mass-flow (kg/d) is defined as the (average) flow (Q) multiplied by the (average flow weighed) concentration (C) according to:

$$Q\left(\frac{\text{m3}}{\text{d}}\right) \times C\left(\frac{\text{kg}}{\text{m3}}\right) = L\left(\frac{\text{kg}}{\text{d}}\right) \quad (13.5)$$

e.g. the TP mass balance over the water line is given by:

$$Q2 \cdot TP_2 - Q3 \cdot TP_3 - Q5 \cdot TP_5 \approx 0 \quad (13.6)$$

According to 13.6, the total phosphorus balances for the sludge line (13.2), the water line (13.3) and the overall wastewater treatment plant (13.4) can also be calculated.

In Table 13.2 the columns represent 4 mass balances (*n*), within each balance the flow elements *i* (Q1-8) are also indicated as the flow vector. All empty spaces in the balances represent zeros. Elements with negative signs are outflows, and positive elements are inflows. The balances correspond with the flow vector which consists of measured (*m*) and unknown (*u*) flows. The unknown flows *u* need to be calculated. The total amount of flows equals *m*+*u*=*i* elements (Q1-8). The mass balances may relate to individual reactors or groups of reactors (e.g. the sludge line or the water line).

A system with *n* balance equations (Table 13.3, *n*=4), can be solved when *n*=*u*. In our case, a system of 4 equations can be solved when 4 are unknown (*u*) and 8 – 4 = 4 are measured (*m*). For *n*>*u* the system is 'over determined'. This is expressed in the 'degree of redundancy', defined as *n*-*u*. It should be noticed that the degree of redundancy is calculated based on the number of independent balance equations. If the system is redundant, errors can be detected in the data.

Table 13.3 Matrix of conserved mass balances (example WWTP Zagreb).

		Flow balance →				
	Flow ↓	Primary treatment	Sludge line	Water line		Overall WWTP
Q1	Influent	+Q1*C1	-	-	→	+Q1*C1
Q2	Settled influent	-Q2*C2	-	+Q2*C2		-
Q3	Effluent	-	-	-Q3*C3	→	-Q3*C3
Q4	Primary sludge	-Q4*C4	+Q4*C4	-		-
Q5	Waste activated sludge	-	+Q5*C5	-Q5*C5		-
Q6	Thickening supernatant	+Q6*C6	-Q6*C6	-		-
Q7	Dewatering filtrate	+Q7*C7	-Q7*C7	-		-
Q8	Sludge	-	-Q8*C8	-	→	-Q8*C8
	SUM OF BALANCE	0	0	0		0

The higher the degree of redundancy, the higher the gain in accuracy. In our example this would mean that we can solve the flow balances when 4 out of 8 flows are measured. When 4 additional TP mass balances are calculated (*n*=8), we have over-determined our system of mass balances (*n*>*u*). This means we can solve the system of mass balances for more unknown

parameters (e.g. in the case less than four flows are measured *u*>4) and/or it will provide the possibility to check our flow and mass balanced data (SRT), thereby strongly improving the accuracy of the design study.

13.2.5 COD and N mass balances; calculating operational plant performance

Similar to the total phosphorus mass balance calculations, mass balances of COD and nitrogen can also be calculated. However there is an important difference between the two balance calculations; both COD and nitrogen are converted in the activated sludge plant and thereby disappear from the water flow via the gas phase. COD leaves the wastewater treatment plant via production of CO_2 (*g*) and CH_4 (*g*). Nitrogen leaves the wastewater treatment plant via the production of N_2 (*g*). Both COD and nitrogen are non-conserved mass balances where the sum of the mass balance represents the conversion term. As a result, these balances cannot be used to check the data for possible errors. On the other hand, the calculated conversions following from the COD and nitrogen mass balance provide valuable information on the plant operational state and can be used for plant assessment. Here we will present how to formulate the COD and nitrogen balance over the water line resulting in exact information on the amount of oxidized and denitrified organic material, the converted loads of nitrification and denitrification and reliable sludge production numbers.

For the COD and nitrogen (N) balances over the water line, the following balances are given:

The TKN balance over the water line calculates nitrification:

$$Q2 \times TKN_{Q2} - Q3 \times TKN_{Q3} - Q5 \times TKN_{Q5} \approx \text{ Nitrified load} \quad (13.7)$$

The TN balance over the water line calculates denitrification:

$$Q2 \times TN_{Q2} - Q3 \times TN_{Q3} - Q5 \times TN_{Q5} \approx \text{ Denitrified load} \quad (13.8)$$

The COD balance over the water line calculates the total organic conversion:

$$Q2 \times COD_{Q2} - Q3 \times COD_{Q3} - Q5 \times COD_{Q5} \approx \text{ total COD to } CO_2 \quad (13.9)$$

From the nitrified load (kg N/d), denitrified load (kg N/d) and oxidized organic load (kg COD/d) calculated from the mass balances (5, 6 and 7), the oxygen consumption can be calculated based on the following stoichiometric relations (in moles and in grams N, C and O_2).

Nitrification (in moles):

$$NH_4^+ + 2 \cdot O_2 \rightarrow NO_3^- + H_2O + 2 \cdot H^+ \quad (13.10)$$

Nitrification (in grams N and O_2):

$$NH_4^+ + 4{,}57 \cdot O_2 \rightarrow NO_3^- \quad (13.11)$$

From (13.11) and (13.7) OC_{NIT}, the stoichiometric oxygen consumption for nitrification (kgO_2/d) can be calculated according to:

$$\left(Q2 \times TKN_{Q2} - Q3 \times TKN_{Q3} - Q5 \times TKN_{Q5}\right) \times 4.57 \approx OC_{NIT} \quad (13.12)$$

Denitrification (in moles):

$$2 \cdot NO_3^- + 2 \cdot H^+ \rightarrow N_2\ (g) + 2.5 \cdot O_2 + H_2O \quad (13.13)$$

Denitrification (in grams N and O_2):

$$NO_3^- + 2.86 \cdot COD \rightarrow N_2\ (g) \quad (13.14)$$

From 13.14 and 13.8 COD_{DEN}, the denitrified load of COD (kg COD/d) can be calculated according to:

$$\left(Q2 \times TN_{Q2} - Q3 \times TN_{Q3} - Q5 \times TN_{Q5}\right) \times 2.86 \approx COD_{DEN} \quad (13.15)$$

The overall oxygen consumption for nitrogen removal is:

$$NH_4^+ \rightarrow NO_3^- \rightarrow N_2(g):\ -4.57 + 2.86 = -1.71\ \frac{gO_2}{gN-N_2} \quad (13.16)$$

For the COD calculation the following definition is used (e.g. oxidation of methane).

Organic oxidation (in moles):

$$CH_4^+ + 2 \cdot O_2 \rightarrow CO_2^- + 2 \cdot H_2O \quad (13.17)$$

Organic oxidation (in grams C and O_2):

$$CH_4^+ + 4 \cdot O_2 \rightarrow CO_2^- + 2 \cdot H_2O \quad (13.18)$$

Organic oxidation (in grams COD):

$$COD + O_2 \rightarrow CO_2^- \quad (13.19)$$

From 13.15 and 13.9 COD_{OX}, the aerobic oxidized load of COD (kg COD/d) can be calculated according to:

$$\left(Q2 \times COD_{Q2} - Q3 \times COD_{Q3} - Q5 \times COD_{Q5}\right) - COD_{DEN} = COD_{OX} \quad (13.20)$$

From 13.15 and 13.19 COD_{OX}, the (aerobically) oxidized organic load (kg COD/d) is calculated according to:

$$\left(Q2 \times COD_{Q2} - Q3 \times COD_{Q3} - Q5 \times COD_{Q5}\right) - COD_{DEN} = COD_{OX} \quad (13.21)$$

From 13.12 and 13.21 ThOD, the total theoretical oxygen demand (kg COD/d) is calculated according to:

$$OC_{NIT} + COD_{OX} = ThOD \quad (13.22)$$

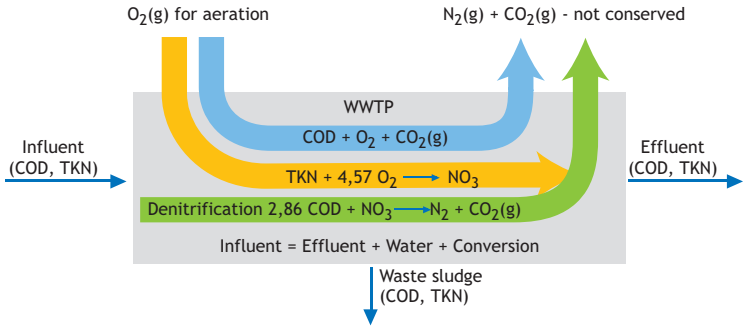

Figure 13.2 Schematic representation of the overall WWTP mass balances of COD and TKN including the conversions: nitrification, denitrification and COD conversion (aerobic and anoxic). Air is an input of the system, and CO2 and N2 are reaction products leaving via the gas phase. If influent, effluent and water are measured the balances (and internal conversions) can be calculated.

13.3 Mass balance assessment; an example

Instead of calculating all the above mass balance equations separately, a systematic method is presented to calculate the mass balance equations in one matrix table overview. Therefore, the same principles are applied as explained in Table 13.2 and Table 13.3. For the example of WWTP Zagreb, the mass balances were calculated over the water line (Table 13.4) and the overall wastewater treatment plant (Table 13.5)

Table 13.4. Flow and mass balances of TP, TKN, NO_3, COD and O_2 over the water line (WWTP Zagreb). All residues are zero in the model (no measurement errors exist).

		Q	TP	TKN	NO_3	COD	O_2
Flow	Direction (in/out)	m^3/d	kgP/d	kgN/d	kgN/d	kgCOD/d	kgO_2/d
Primary settlers (Q2)	1	336,522	1,463	15,991	159	79,193	55
Effluent (Q3)	-1	-331,170	-918	-949	-5,741	-5,882	-393
WAS splitter (U) (Q5)	-1	-5,325	-545	-2,510	-76	-35,791	0
Nitrification of ammonia				-12,132	12,132		-55,443
Denitrification of nitrate					-6,474	-18,579	
Aerobic COD conversion						-18,761	-18,761
Total theoretical O_2 demand							74,543
Residue sum of mass balance (~0)		0	0	0	0	0	0

In Table 13.4 the balances for Q, TP, TKN, NO_3, COD and O_2 are presented in the vertical columns. The flows (Q) over the water line are presented in the rows. The inflow (+1) is the outflow of the primary settler (Q2). The two outflows (-1) are the effluent Q3 and the waste activated sludge is Q5 (Figure 13.2).

Besides the (mass) flows leaving the water line, biological conversions also occur in the activated sludge tanks which are presented in the lower half of the table: nitrification of ammonia, denitrification of nitrate, aerobic COD conversion and finally the total stoichiometric oxygen demand.

Both flows and TP are conserved parameters (theoretically remain within the plant and can be measured). The balance for the flows and TP (first and second column) are closed which means that the sum of all input and output values for a particular parameter equals or is close to zero. From the TKN balance (13.7), it is calculated that 12,132 kg ammonia is removed (-) from the TKN balance in the nitrification process. From the nitrification process (13.11) a proportional (stoichiometric) amount (12,132 kg) of nitrate will appear (+) in the nitrate balance (Table 13.4). Based on the stoichiometric relation (4.57 kgO_2 per $kgNH_4$ converted), 55,443 kg of oxygen is consumed (-) in the nitrification process (13.12). Denitrification of nitrate is calculated from the result of the nitrate balance (after first calculating the nitrification). When 6,474 kg of nitrate is denitrified, the residue of some of the nitrate mass balance equals zero. Denitrification can also be calculated directly from the total nitrogen balance (13.8), being the sum of the TKN and NO_3 balances in Table 13.4 (i.e. TN = TKN + NO_3).

In the denitrification process 2.86 g of COD is removed per gram of nitrate according to the stoichiometric relation 13.14). Therefore, 18,579 kg of COD is removed from the COD balance as a result of denitrification of nitrate.

What remains in the COD balance is the aerobic COD conversion. The balance is closed by calculating 18,761 kg of aerobic COD conversion. Because the stoichiometric relation between COD and oxygen equals 1 according to the definition of COD (13.19), in the aerobic COD conversion the same amount (18,761 kg) of oxygen is consumed. Via the N and COD balance the kilos of COD converted in the denitrification process are already taken into account (13.20).

Finally, the total theoretical oxygen demand (ThOD) is the sum of the total oxygen used in the nitrification process (OC_{NIT}) and oxygen used in the aerobic COD conversion (COD_{OX}), adding up to 74,543 kg of theoretical oxygen demand (13.21). This is the amount of oxygen that is actually used by the biomass in the conversions. However, as a result of the oxygen transfer efficiency (k_La), approximately double this amount of air needs to be provided to the activated sludge system. ThOD can be used to design the blower capacity.

13.4 Operational assessment; an example

From the calculated conversions in Table 13.4 a good insight has already been obtained into how the plant operates. Also, from the ratio between anoxic and aerobic COD conversion (e.g. Table 13.4: 18,579 kg COD is denitrified and 18,761 kg COD oxidized) it can be seen how denitrification works; in this case 50% of the COD is oxidized under anoxic conditions while 50% of the COD is oxidized using oxygen. With the knowledge that the treatment process does not incorporate an anoxic activated sludge reactor it can be concluded that a considerable COD load is denitrified under simultaneous aerobic/anoxic conditions in the aerobic tanks (SND). This would be the result of anoxic (or anaerobic) pockets in the sludge and oxygen concentration gradients.

In addition the mass balance calculations show that when the amount of COD in the waste activated sludge becomes larger, at the same time the total theoretical oxygen consumption of the process becomes smaller, and vice versa. The mass balance calculations clearly show how a high SRT operation (i.e. low WAS production) via the COD mass balance directly results in higher oxygen consumption. It should be noted that in the calculation in Table 13.4 the possible production of hydrogen gas or methane in the water line is not taken into account. In the BioWin model, estimation is made of the hydrogen gas and methane production in the wastewater treatment process. Especially for anaerobically-operated process units this can be

considerable. However, some mass theme production will also be calculated in the model in the water line typically resulting in small calculation differences from what is presented here (<5%). This is demonstrated based on the same mass balance equations calculated for the overall process of a WWTP in Zagreb (Figure 13.2, Table 13.5). In this example the anaerobic digester is also part of the mass balance and it is expected that a considerable amount of COD entering the overall WWTP will leave the system (defined as the overall mass balance) as methane (CH_4 (*g*)) and hydrogen gas (H_2 (*g*)) via anaerobic digestion (the model calculated 43,013 kg COD/d). Based on the model calculations a considerable amount of ammonia evaporation was also calculated in the digesters. It should be noted that the net result of the two mass balances (Table 13.4 and Table 13.5) is the same regarding the ThOD, as it should be expected because no aerobic processes occur in other process units than covered by the water line.

Table 13.5. Flow and mass balances of TP, TKN, NO_3, COD and O_2 over the total WWTP (WWTP Zagreb). All residues are zero in the model (no measurement errors exist).

Flow	Direction (in/out)	Q m^3/d	TP kgP/d	TKN kgN/d	NO_3 kgN/d	COD kgCOD/d	O_2 kgO_2/d
Sewage	1	331,324	1,525	19,126	157	138,889	397
Effluent	-1	-331,170	-918	-949	-5,741	-5,882	-393
Dewatered sludge	-1	-155	-607	-4,339	0	-51,695	0
Methane, nitrogen, NH_3 biogas*				-1,690		-43,013	
Nitrification of ammonia				-12,148	12,148		-55,515
Denitrification of nitrate					-6,564	-18,839	
Aerobic COD conversion						-19,460	-19,460
Total theoretical O_2 demand							74,972
Residue sum of mass balance (~0)		0	0	0	0	0	0

*estimated from model calculations

13.5 SRT calculations

After closing the flows and TP balance and calculating all the conversions over the activated sludge plant, this information can also be used to accurately calculate the sludge retention time (SRT). The operated SRT is a key operational parameter which heavily determines the plant and model performance. Different calculation methods can be used to calculate the SRT. Traditionally SRT is calculated based on the mass of total suspended solids (TSS) in the activated sludge tanks and the mass of TSS wasted per day according to the equation:

$$SRT = (V_{AT} \times TSS_{AT}) \,/\, (Q_{WAS} \times TSS_{WAS} + Q_{EFF} \times TSS_{EFF}) \quad (13.23)$$

where:

V_{AT}	Activated sludge tank volume (m^3)
TSS_{AT}	TSS concentration in the aeration tanks (kg/m^3)
Q_{WAS}	Waste activated sludge flow (m^3/d)
TSS_{WAS}	TSS concentration in the waste activated sludge (kg/m^3)
Q_{EFF}	Effluent flow (m^3/d)
TSS_{EFF}	TSS measured in the effluent flow (kg/m^3)

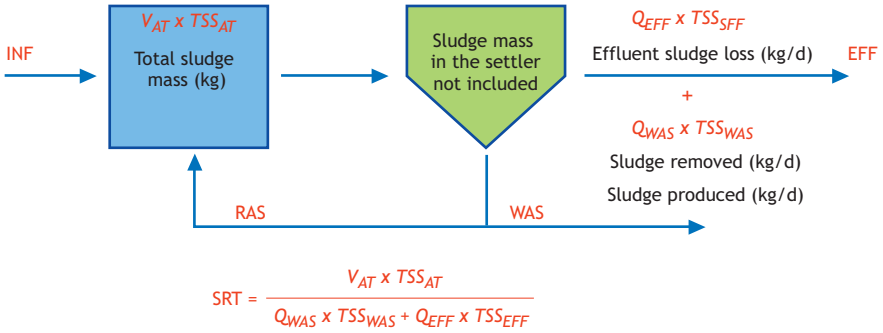

Figure 13.3 SRT calculation

The same calculation method can also be used for other particulate material measured in the activated sludge such as VSS, COD, TKN or TP. Therefore in 13.23, TSS is replaced by one of these measured parameters. Activated sludge consists of a mixture of TSS, VSS, COD, TKN and TP at a more or less constant ratio. Based on this fact, in the different calculations these elements become interchangeable and this is also the reason why it is useful to measured activated sludge characterization when performing a model design study.

13.6 An alternative SRT calculation: using the TP balance

A totally different method to calculate the SRT is based on the total phosphorus balance. This balance is maintained and therefore the TP that flows in the system is the same as the TP flowing out of the system. We also know from the activated sludge characterization that the relation between phosphorus and total suspended solids is a constant ratio (i.e. the particulate TP/TSS ratio is assumed to be constant over an estimated period of 2-3 times the sludge retention time). Practical advantages of using phosphorus as a 'tracer' compound are: (i) measuring total phosphorus is relative simple, (ii) in WWTP phosphorus concentrations are generally well above the detection limit and (iii) total phosphorus measurements are often readily available.

The TP balance over the water line is given by 13.6. When the influent and effluent TP are measured (i.e. TP in flows Q2 and Q3, Figure 13.1), TP in the waste sludge (kgP/d) can be calculated according to:

$$Q_{WAS} \times TP_{WAS} = Q_{INF} \times TP_{INF} - Q_{EFF} \times TP_{EFF} \quad (13.24)$$

From the activated sludge characterization the total amount of particulate phosphorus in the activated sludge tanks ($TP_{X,AT}$) can be calculated according to:

$$V_{AT} \times \left(TP_{AT} - TP_{S,AT}\right) = \text{total particulate TP mass in AT} \quad (13.25)$$

It can be assumed that soluble phosphorus in the effluent (ortho-phosphate) is the same as the concentration soluble phosphorus in the outflow of the activated sludge tanks and therefore:

$$V_{AT} \times \left(TP_{AT} - TP_{S,EFF}\right) = \text{total particulate TP mass in AT (kg P)} \quad (13.26)$$

Substituting 13.23 with 13.24 and 13.26 this results in:

$$SRT = \frac{V_{AT} \times \left(TP_{AT} - TP_{S,EFF}\right)}{\left(Q_{INF} \times TP_{INF} - Q_{EFF} \times TP_{EFF}\right) + Q_{EFF} \times \left(TP_{EFF} - TP_{S,EFF}\right)}$$

$$SRT = \frac{V_{AT} \times \left(TP_{AT} - TP_{S,EFF}\right)}{Q_{INF} \times TP_{INF} - Q_{EFF} \times TP_{S,EFF}} \quad (13.27)$$

The flow balance over the water line is:

$$Q_{EFF} = Q_{INF} - Q_{WAS} \quad (13.28)$$

And therefore:

$$SRT = \frac{V_{AT} \times \left(TP_{AT} - TP_{S,EFF}\right)}{Q_{INF} \times \left(TP_{INF} - TP_{S,EFF}\right) - Q_{WAS} \times TP_{S,EFF}} \quad (13.29)$$

Because Q_{WAS} is usually a small flow and also the effluent ortho-phosphate concentration of a wastewater treatment plant is usually low (typically in the range 1-3 mg P/L), soluble phosphate leaving the system via the waste sludge may be ignored (or estimated) in the case the waste sludge flow is not known and therefore the SRT (days) can be calculated according to:

$$SRT = \frac{V_{AT} \times \left(TP_{AT} - TP_{S,EFF}\right)}{Q_{INF} \times \left(TP_{INF} - TP_{S,EFF}\right)} \quad (13.30)$$

where:

V_{AT}	Activated sludge tank volume (m^3)
TP_{AT}	Total phosphorus concentration in the aeration tanks (kg/m^3)
Q_{INF}	Influent flow (m^3/d)
TP_{INF}	Total phosphorus concentration in the influent (kg/m^3)
$TP_{S,EFF}$	Soluble (ortho) phosphate concentration in the effluent and aeration tanks (kg/m^3)

In Table 13.6 the different calculation methods for the SRT for the example of WWTP Zagreb are presented. Based on the constant ratio between total phosphorus and TSS measured from the activated sludge characterization (Table 13.1), the sludge production in units TSS or COD can also be calculated.

In the above calculation it is assumed that ortho-phosphate in the WAS and AT is identical to ortho-phosphate in the effluent. In other words, it is assumed that no phosphate is released in the secondary clarifiers. However, sometimes this can happen especially with bio-P systems. In this case the assumed simplifications are not justified and these concentrations should be measured. Typically, it can be assumed that ortho-phosphate measured in the AT is the same as measured in the effluent. The SRT calculation method given as an example does not include internal phosphorus loads. These internal loads need to be included when relevant.

Table 13.6 Sludge Retention Time (SRT) calculations using different calculation methods (example WWTP Zagreb)

Parameter	Unit	Value
Particulate mass in activated sludge tanks (AT)		
TPx	kg	2,605
CODx	kg	177,057
VSS	kg	138,297
TSS	kg	178,403
Particulate mass in waste activated sludge (WAS)		
TPx	kg/d	529
CODx	kg/d	35,889
VSS	kg/d	28,081
TSS	kg/d	36,252
Particulate mass in effluent (EFF)		
TPx	kg/d	10
CODx	kg/d	709
VSS	kg/d	556
TSS	kg/d	718
Phosphorus mass flows for alternative SRT calculation		
TP (influent)	kg/d	1,463
TPs (PO_4 in effluent)	kg/d	908
SRT calculations		
Calculated on TSS	d	4.8
Calculated on COD	d	4.8
Calculated on TP	d	4.8
Calculated on TP influent	d	4.7

Chapter 14

Model protocol; step 5: secondary clarifier assessment

14.1 Introduction

When the operation of an activated sludge plant is assessed, the operation of the secondary settlers should also be taken in account because the functioning of these process units determines both the plant operation and the effluent quality. The models that are used to assess the operation of the secondary settlers are very different from the activated sludge model. Secondary settling is a very complex process that is not fully understood and therefore difficult to describe in a mechanistic model, inn contrary to the metabolism of activated sludge microorganisms. Instead, the models which are used for describing and designing secondary settlers are more empirical (heuristic), based on practical experience-based rules and tested on statistics. There are several (country-wide) design standards available for secondary settler design, for example, the UK standard (WRC), German standard (ATV) and the Dutch standard (STOWA). Besides these local (adapted) standards there is a general empirical design method and there is the Flux theory-based design method. All these guidelines and design methods can be used; however, some standards are more commonly used than others, mostly for practical reasons. The principles of these five most widely used design procedures are explained in this chapter. The objective of this chapter is to illustrate the variety of principles used, what data is used for the design and how the results relate. It is shown that different design methods for secondary clarifiers can result in (very) different designs. For all situations where the actual sludge characteristics (typically settleability indicators such as SVI) are not known, the designer's choice remains one of expert judgment and empirical knowledge. In this evaluation, all the design methods are used simultaneously to support this expert judgment. Using all the design methods simultaneously provides the best possible estimation of settler design within the context of uncertain sludge characteristics and the choice for typical design parameters (e.g. permitted overflow rates or surface loadings). To demonstrate the methodology for secondary settler design an example is presented based on the design retrofit performed at WWTP Vinkovci. This chapter starts by presenting the retrofit design results (Table 14.1 and Table 14.2). The first table presents the input parameters for the five different design methods (Empirical, the Flux method, STOWA, ATV and WRC). The second table presents the design results for several flow conditions (ADWF, PDWF and PWWF). In addition the mean of the five designs is calculated (in this study all the design methods are weighted equally). In the following sections the theory behind the different design methods is explained. To make it possible to calculate and compare the different design methods (e.g. $SSVI_{3,5}$, SVI, DSVI), the different sludge settling characterization methods are also compared. This is carried out in the section 'Alternative parameters for sludge settleability'. Here the empirical formulas are presented to explain the different types of sludge measurements.

14.2 Secondary settler design results; an example

From the results presented in Table 14.2 it can be observed that the different calculation methods give rather different results for the required settling area. These results were tested for their sensitivity to the different design input parameters. Therefore, several typical design input parameter values were tested (e.g. different SVI and MLSS inputs). Based on typical operation and practical experience, all the design inputs were considered within the empirical (realistic) range of operation.

Generally speaking, the German design standard ATV is the most conservative design method and therefore calculates the largest settler surface area. In this example the ATV method calculates an additional required surface of 170%. All the other methods are less conservative and indicate that the current settler surface is sufficient (i.e. <100% extension is required). The empirical design method and the British WRC methods both calculated the smallest settler area. The difference between the design methods is the result of the fact that the German effluent standards are much stricter than the British effluent standards. Whereas in Germany secondary treatment is generally applied, in the United Kingdom BOD-removing wastewater plants operated at short SRT are widely found. This fact was also recognized in the Netherlands and therefore STOWA proposes a different design method resulting in smaller, less conservative designs than the German ATV method. The mean design value of the five presented methods seems to provide a general and objective expert base for settler design. However, based on the required effluent design standards one (or more) of the five design methods can be given more weight in the design considerations.

Table 14.1 Clarifier design model input parameters

WWTP Vinkovci General Parameters	Code	Unit	Value
Measured SVI 30 min	SVI30	mL/g	150
Sludge concentration	MLSS	kg/m^3	5.2
Average dry weather flow	ADWF	m3/h	371
Peak flow % of ADWF	PDWF	%	150%
Peak dry weather flow	PDWF	m^3/h	557
Peak flow % of ADWF	PWWF	%	243%
Peak wet weather flow	PWWF	m^3/h	903
Flux theory			
Initial settling velocity	v0	m/h	6.1
Hindered settling parameter	phin	m^3/kg	0.42
Safety factor on area	Fcorr	-	1.0
WRC			
Stirred SVI 3.5 g/L	SSVI3.5	mL/g	84
Safety factor on area	F_{corr}	-	1.0
ATV-STOWA			
Diluted SVI	DSVI	mL/g	126
DSVI*MLSS	DSV30	mL/L	657

Table 14.2 Overview of the results of the clarifier design

WWTP Vinkovci		Calculation method					
Design extention	**Unit**	**Empirical**	**Flux**	**WRCm**	**ATV (1976)**	**STOWA**	**MEAN**
Designed surface	m^2	900.5	1,304.5	977.1	2,243.0	1,037.9	1,292.6
Current surface available	m^2	1,320.4	1,320.4	1,320.4	1,320.4	1,320.4	1,320.4
Additional surface required	m^2	-419.9	-15.8	-343.3	922.7	-282.5	-27.8
Extended surface	% extension	0.7	1.0	0.7	1.7	0.8	1.0
Diameter for additional tank	m	29.0	29.0	29.0	29.0	29.0	29.0
Number of tanks applied	tank units	-0.6	0.0	-0.5	1.4	-0.4	0.0
At ADWF							
Overflow, QADWF	m^3/h	371.1	371.1	371.1	371.1	371.1	371.1
Overflow rate to surface	m/h	0.4	0.3	0.4	0.2	0.4	0.3
Recycle flow (QRAS)	m^3/h	667.9	350.7	977.1	745.3	745.3	697.2
Recycle rate to surface	m/h	0.7	0.3	1.0	0.3	0.7	0.6
Recycle ratio (RAS/INF)	-	1.8	0.9	2.6	2.0	2.0	1.9
RAS TSS concentration	kg/m^3	8.1	10.7	7.2	7.8	7.8	8.3
Solids loading rate	$kg/m^2.h$	6.0	2.9	7.2	2.6	5.6	4.8
At PDWF							
Overflow, QPDWF	m^3/h	556.6	556.6	556.6	556.6	556.6	556.6
Overflow rate to surface	m/h	0.6	0.4	0.6	0.2	0.5	0.5
Recycle flow (QRAS)	m^3/h	667.9	350.7	977.1	745.3	745.3	697.2
Recycle rate to surface	m/h	0.7	0.3	1.0	0.3	0.7	0.6
Recycle ratio (RAS/INF)	-	1.2	0.6	1.8	1.3	1.3	1.3
RAS TSS concentration	kg/m^3	9.5	13.5	8.2	9.1	9.1	9.9
Solids loading rate	$kg/m^2.h$	7.1	3.6	8.2	3.0	6.5	5.7
At PWWF							
Overflow, QPWWF	m^3/h	902.9	902.9	902.9	902.9	902.9	902.9
Overflow rate to surface	m/h	1.0	0.7	0.9	0.4	0.9	0.8
Recycle flow (QRAS)	m^3/h	451.5	1,065.4	977.1	745.3	745.3	796.9
Recycle rate to surface	m/h	0.5	0.8	1.0	0.3	0.7	0.7
Recycle ratio (RAS/INF)	-	0.5	1.2	1.1	0.8	0.8	0.9
RAS TSS concentration	kg/m^3	15.6	9.6	10.0	11.5	11.5	11.6
Solids loading rate	$kg/m^2.h$	7.8	7.8	10.0	3.8	8.3	7.6

14.2.1 Empirical design of hydraulic surface loading

In the empirical model the input was used from typical surface loads and sludge loads, as found in literature and from familiar installations with satisfactory performance regarding the effluent quality. Because other methods have practical limitations the empirical method is often used. The design is straightforward. Empirical design rules are based on (local) engineering experience. The selection of the required settler area can be based on the maximum hydraulic loading specification and/or maximum solid loading specification, or on other criteria. In this case, maximum 1 m/h for PDWF and 2.0 m/h for PWWF is envisaged. In addition, the clarifier should not be loaded higher than 6 $kg/m^2.h$ during dry weather and 15 $kg/m^2.h$ temporarily during wet weather. Each of these specifications will lead to the required clarifier area as calculated. The largest area is selected. Recycling ratios between 0.5 and 1.0 are usually satisfactory.

14.2.2 ATV calculation method

Both the ATV and STOWA design are based on the diluted sludge volume index (DSVI) which is taken as 126 mL/g; a typical value average loaded WWTP. The diluted sludge volume is used as an additional design parameter together with empirical parameters. The input sensitivity of the DSVI is lower than in the WRC and Flux models, but nevertheless the design value remains an estimated parameter. In the end, also in the STOWA and ATV method the design is checked with several values for the DSVI to find out the difference regarding permissible surface loads, which brings us back to the empirical method.

The ATV 1976 (and STOWA) design principles are based on the DSVI test. The DSVI test is essentially an SVI test performed under more uniform conditions: diluting the sludge sample with effluent such that the settled volume falls into the 150-250 ml range. Based on the DSVI, two concepts related to settled sludge volume are introduced. DSV30 expressed in ml/l is the settled volume of the MLSS under the test conditions:

$$DSV_{30} = X_F \cdot DSVI \tag{14.1}$$

Sludge volume loading rate (q_{SV} in l/m^2/h) is

$$q_{SV} = Q_i / A \cdot DSV_{30} \tag{14.2}$$

which is the volumetric rate of settled sludge loaded in the clarifier, in the same way as solid loading, but expressed in volume instead of mass. The permissible overflow rate depends on DSV30. q_o must be smaller than 1.6 m/h.

$$q_o = 2400 \cdot DSV_{30}^{-1.34} \tag{14.3}$$

The required recycling flow is based on the compressibility of the sludge, and the maximum solid concentration it can reach under the given conditions, which is estimated from the DSVI test. Under average dry weather conditions this is:

$$X_{R,ADWF} = \frac{1200}{DSVI} \tag{14.4}$$

Under peak wet weather flow conditions:

$$X_{R,PWWF} = \frac{1200}{DSVI} + 2 \tag{14.5}$$

The necessary recycling flow is calculated based on the clarifier mass balance:

$$(Q_I + Q_R) \cdot X_F = Q_R \cdot X_R \tag{14.6}$$

where:

Q_I influent flow (m^3/h)
Q_R recycling flow (m^3/h)
X_F bioreactor mixed liquor suspended solids concentration (kg/m^3)
X_R return solids concentration (kg/m^3)

14.2.3 STOWA calculation method

Both the ATV and STOWA designs are based on the diluted sludge volume index (DSVI) which is taken as 126 ml/g; a typical value for average loaded WWTPs. The STOWA design procedure is closely based on the ATV design. The permissible overflow rate depends on DSV_{30}.

$$q_O = \frac{1}{3} + \frac{200}{DSV_{30}} \tag{14.7}$$

The sludge volume loading rate is calculated q_{SV} (m/h) according to ATV. This rate must be between 300 and 400 L/m^2.h. Also the recycling flow can be calculated in an identical way to the ATV method.

14.2.4 WRC design principle

The WRC design procedure is based on the $SSVI_{3.5}$ test, which in theory provides the most reliable measure of settleability. The $SSVI_{3.5}$ of the sludge used in this example was 84 ml/g (normal settling and compacting). In this example, the second lowest amount of tanks calculated was based on the WRC principle (Table 14.2). The WRC model is based on the statistical interpretation of 30 clarifier performances, resulting in an empirical formula to fit the model. The WRC model is, like the Flux theory, highly sensitive to changing sludge characteristics. The smallest parameter change already gives large differences in clarifier surface. Therefore, the sludge characteristics should be established several times in practice and over time, to obtain a reliable model.

In the WRC method the critical recycle ratio is calculated from:

$$q_{R,crit} = 1.612 - 0.00793 \cdot SSVI_{3.5} - 0.0015 \cdot max\left(0,\left(SSVI_{3.5} - 125\right)\right)^{1.115} \tag{14.8}$$

The required area is calculated from an empirical equation that is derived based on flux settling parameters measured at 30 plants in the UK and correlated to $SSVI_{3.5}$.

$$A = \left(\frac{X_F \cdot Q_{PWWF}}{306.86 \cdot q_{R,crit}^{0.68} \cdot SSVI_{3.5}^{-0.77} - X_F \cdot q_{R,crit}} \right) \tag{14.9}$$

From here, a 125% safety factor is applied from which the PWWF overflow rate is calculated. When evaluating the WRC method, it needs to be taken into consideration that in general British WWTPs are highly loaded (sludge load and hydraulic load). Therefore, these designs often result in relative high suspended solids in the effluent (20-30 mg/L). The derived WRC model is expected to perform in the same range. Moreover, the WRC model is based on batch settlement tests, which always perform better than under real-world conditions, where the sedimentation process deteriorates due to the hydraulics and mass density flows. In our calculations with the WRC method, reasonably accurate sludge characteristics are used. From the evaluation of the WRC method, it is concluded that this method can only be used if the appropriate sludge settling data is available. This becomes even more evident when considering the 125% safety factor in the design result! The practical application of the WRC method for design purpose is limited, especially for the design of low-loaded systems meeting EU standards.

14.2.5 Flux theory

The Vesilind settling parameters for the sludge were determined in a series of zone-settling experiments (results in Figure 14.4). In the practical application of the Flux theory, it is

recommended that the surface area is increased (the permissible flux is reduced) by a 25% safety factor compared to the theoretical value. This is to account for imperfections in the real clarifier structure, in contrast to the ideal 1-D approximation as laid out in the flux theory. Like the WRC method, the reliability of the Flux theory heavily relies on accurately measured sludge settling data. In this study typical design input values were assumed resulting in the calculations as presented in Table 14.1 and 14.2. For this reason, like the other methods presented here, the flux calculation method also has limited value for (exact) design evaluation. These calculations therefore should be considered as examples for the method. However, using these design criteria (all five together) gives the best possible guess on future settling performance. According to the Flux theory the settling velocity at the MLSS concentration is calculated as follows:

$$v_{s,MLSS} = v_0 \cdot e^{(phin \cdot MLSS)} \qquad (14.10)$$

The overflow rate during PWWF must not exceed this velocity. This can be graphically determined from the D&O and state point diagrams below (Figure 14.1 and 14.2).

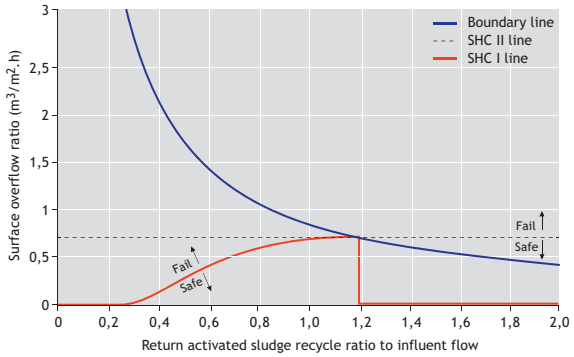

Figure 14.1 The Flux theory: the Ekama Design and Operation (D&O) Chart; determining the maximum settler surface overflow ratio as a function of the return activated sludge (RAS) flow (ratio to influent).

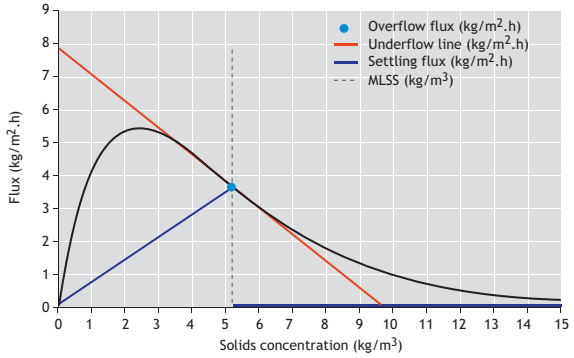

Figure 14.2 The Flux theory peak wet weather flow (PWWF) state point diagram; determining the maximum solid loading (flux in kg/m^2.h) at PWWF. The underflow line should not exceed the settling flux. The angle (tangent) of the underflow line is determined from the D&O Chart.

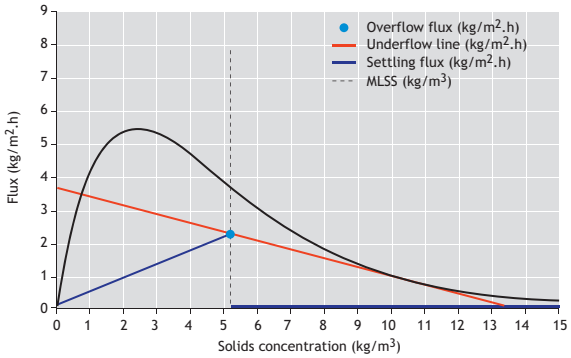

Figure 14.3 The Flux theory peak dry weather flow (PDWF) state point diagram; determining the maximum solid loading (flux in kg/m^2.h) at PDWF. The underflow line should not exceed the settling flux. The angle (tangent) of the underflow line is determined from the D&O Chart.

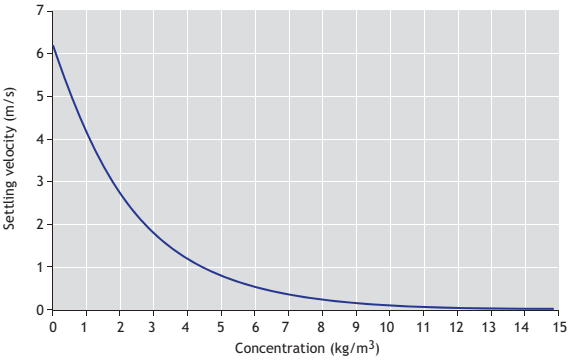

Figure 14.4 The Flux theory: Vesilind settling velocity; estimated input value.

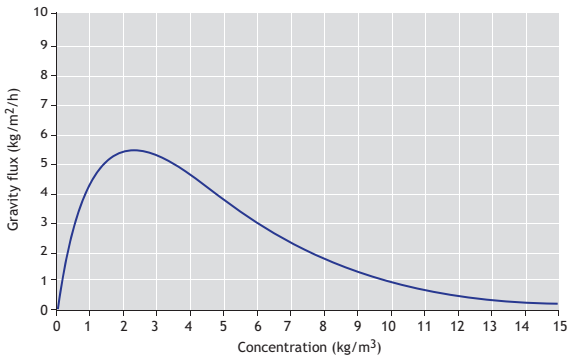

Figure 14.5 The Flux theory: Gravity settling flux as a function of sludge concentration.

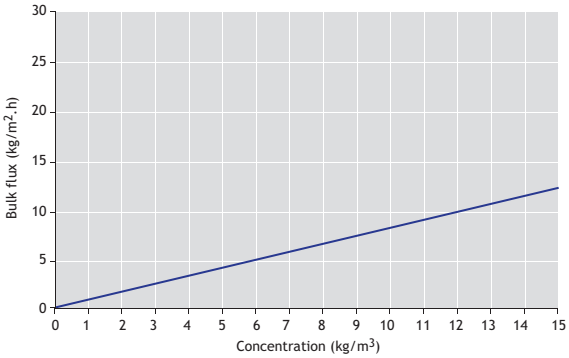

Figure 14.6 The Flux theory: Bulk flux (underflow RAS) removal as a function of the sludge concentration.

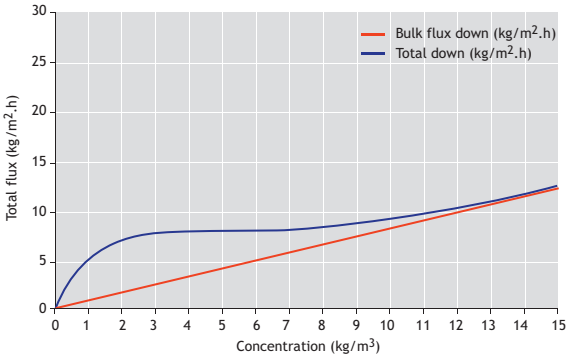

Figure 14.7 The Flux theory: Total flux including bulk (underflow RAS) removal as a function of the sludge concentration.

14.3 Alternative indicators of sludge settleability

The zone settling velocity test is not particularly suitable for routine use at wastewater treatment plants because it is very tedious and time consuming. For this reason many researchers have tried to find alternative ways to express sludge settleability in quantitative terms.

The Sludge Volume Index or SVI (Mohlman, 1934) is probably the best known and most widely applied test for sludge settling. In this test, a certain volume of mixed liquor (for example, 1 litre) is placed in a calibrated cone or cylinder. After a set time (for example, 30 minutes) the sludge volume is read off and, after determination of the initial sludge concentration, the volume of settled sludge per gram of solids is calculated. This number expresses the sludge volume index (SVI or SVI_{30}). As this test is extremely simple to carry out it has found wide application. Unfortunately, it is not a very useful test for quantitative work. Its principal shortcoming is that the numerical value of the test depends on the initial sludge concentration. A true parameter for sludge settleability should be independent of the sludge concentration.

In an attempt to eliminate the influence of sludge concentration, Stobbe (1964) developed the diluted sludge volume index or DSVI. This test is based on the observation that, when the sludge volume after settling is less than about 25 per cent of the initial volume, the calculated SVI value is practically constant and does not depend on the initial sludge concentration. Thus Stobbe (1964) suggested diluting sludge batches until the volume of the diluted suspension, after settling, has a volume of 200 ml or less per litre of initial volume.

White (1975) developed the stirred sludge volume index (SSVI), defined as the volume of a unit mass of suspended sludge solids after 30 minutes of settling in a cylinder while gentle stirring is applied. The SSVI is almost independent of the initial sludge concentration, unless the zone settling velocity is extremely low (< 1 m/h). To quantify settleability for these poorly settling sludges, White suggested using a standard concentration of 3.5 g/L, thus defining $SSVI_{3,5}$. Stofkoper and Trentelman (1982) determined both $SSVI_{3,5}$ and DSVI values in 25 activated sludge processes in The Netherlands. A proportional relationship between the two parameters was found such that:

$$I_{ssv} = cp \cdot I_{dsv} \tag{14.11}$$

where:
$I_{ssv} = SSVI_{3,5}$
I_{dsv} = DSVI
cp = proportionality constant (the average value of cp was determined to be 2/3)

Catunda *et al.* (1989) used sludge with a varying fraction of active sludge and showed that the value of the proportionality constant cp depends on the active fraction of the volatile sludge f_{av}:

$$cp = 1 - 0.35 \cdot f_{av} \tag{14.12}$$

Pitman (1984) developed an empiric relationship between the constants of Vesilind's equation and I_{ssv}. Analysing his data obtained during six years of full-scale investigation, the following correlation was established:

$$v_0/k = 68 \cdot exp(-0.016 \cdot I_{ssv}) \tag{14.13}$$

Ekama and Marais (1986) analyzed their data and that of others (White 1975, Rachwall *et al.* 1981, Koopman and Cadee, 1983) and concluded that Pitman's empiric expression gave a good description for all data. They also verified that there was a relationship between v_0/k and k:

$$k = 0.88 - 0.393 \cdot log(v_0 k) \tag{14.14}$$

Knowing the k-value, v_0 can now be calculated with the aid of 14.13 and 14.14.

$$v_0 = (v_0/k) \cdot k \tag{14.15}$$

Catunda *et al*. (1989) observed that by rearranging (14.13, 14.14 and 14.15), it is possible to express k and v_0 explicitly as a function of I_{ssv}. By substituting 14.13 in 14.14 one has:

$$k = 0.16 + 2.7.10^{-3} \cdot I_{ssv} \tag{14.16}$$

Substitution of 14.15 and 14.16 in 14.13 results in:

$$v_0 = (10.9 + 0.18 \cdot I_{ssv}) \cdot exp(-0.016 \cdot I_{ssv}) \quad (14.17)$$

In the range of I_{ssv} values that are of practical interest for the activated sludge process, 14.17 is almost linear and can be reasonably well described by equation below:

$$v_0 = 11.2 - 0.06 \cdot I_{ssv} \quad (14.18)$$

Equations (6.7 and 6.8) can be used to calculate the constants k and v_0 directly from the I_{ssv} value, without carrying out the zone settling velocity test. The constants can also be calculated from I_{dsv} data using 14.12. It must be remembered that the empiric relationships of this section are all based on experimental work with municipal sewage as wastewater. It is possible that the relationships do not hold for predominantly industrial wastewaters.

Catunda *et al*. (1989) carried out an experimental investigation to evaluate the influence of sludge concentration and the composition of activated sludge generated from municipal wastewater on the values of the settleable settleability constants: k, v_0, I_{dsv} and I_{ssv}. The investigation was carried out on a pilot scale, by operating an aerated lagoon (Rs = 2 d) and a series of 4 aerobic digesters digesting the excess sludge from the aerated lagoon. The active sludge fraction in the sludge was varied between 83% (in the aerated lagoon) and 14% (in the last digester of the series). The main results of investigation are listed below:

$$(1)\ I_{ssv} = 25 + 25 \cdot f_{av} + 5 \cdot X_t \quad (14.19)$$

$$(2)\ I_{ssv}/I_{dsv} = 1 - 0.35 \cdot f_{av} \quad (14.20)$$

$$(3)\ k = 0.16 + 0.003 \cdot I_{ssv} \quad (14.21)$$

$$(4)\ v_0 = 16 - 0.1 \cdot I_{ssv} \quad (14.22)$$

When the results by Catunda *et al.* (1989) and those from Ekama and Marais (1986) are compared, the following three points are concluded:

The relationships between k and I_{ssv} (14.16 and 14.21) and v_0 and I_{ssv} (14.17 and 14.22) found by Catunda *et al.* (1989) do not differ significantly from those suggested by Ekama and Marais (1986);

The relationship between I_{ssv} and I_{dsv} as described by Van Haandel and Catunda (1992) is comparable with the results by Stofkoper and Trentelman (1982), when the active sludge fraction is high ($f_{av} \rightarrow 1$). This is not unexpected, as in the period between 1970 and 1985 nutrient removal was not yet required and most activated sludge processes in Holland were therefore operated using a short sludge age;

The sludge concentration and composition influence the values of the settleability constants. While the influence of the sludge concentration is small, the sludge composition (active fraction) has a very marked effect on I_{ssv} and hence on the values of k and v_0: a lower active sludge fraction results in a decrease in sludge settleability.

14.4 Conclusions on design of clarifiers

It is clear from the above calculation that significantly different clarifier design principles are used around the world and this demonstration leads to different overflow and underflow rates (Table 14.2). The clarifiers designed based on empirical design, the Flux theory and the WRC principles have a relatively smaller surface area, and larger pumps are used to remove the settled sludge from the bottom, at a lower concentration. The ATV and STOWA guidelines lead to building larger settlers, and count on good sludge compatibility (as is the case with the sludge used in this demonstration) which requires lower recycle pumping rates. An objective and reliable design method is using all of the above calculation methods and from there indicating a reasonable average design value. This average value may be based on equal weights, depending on the required effluent quality selected weights resulting in a weighted average design result (equally weighted in the last column of Table 14.2 – Mean).

Chapter 15

Modeling protocol; step 6: sampling program design

15.1 Introduction

A sampling program (or campaign) is primarily carried out to (i) collect routine operating data concerning the overall WWTP performance, (ii) acquire information that can be used to document the performance of a given treatment operation or process, (iii) collect data that can be used to implement proposed new programs for sewerage and WWTP, and (iv) obtain records needed for reporting regulatory compliances. At this stage, we have gone through all the steps of the design and have defined all the possible plant information that has to be acquired to perform a model design study. In general this information is:

- Influent characterization measurements (see chapters 3, 4 and 5);
- Mass balance measurements (see the chapters on plant assessment);
- Activated sludge characterization measurements (see the chapters on plant assessment);
- Effluent characterization (see the chapters on influent characterization);
- All the other necessary information (see the chapter on historical data acquisition).

Following the steps in the modeling protocol, after making the plant inventory, constructing the process flow diagrams (PDF) and defining the area of the plant to be studied, the measurement plan (campaign) can be developed. The required plant measurements follow on from the proposed modeling methodology and from the methodology for plant assessment as was presented in the previous chapter.

A general overview of the measurements is presented in the list above. Based on this, it can be concluded that the majority of the required plant measurements will be identical for all modeling studies; e.g. influent, effluent and activated sludge measurements will be standard for all design studies. Therefore, a large part of the sampling program can be pre-assembled and is generic for all model design studies. The sampling plan is pre-assembled in this chapter and also, based on the example of WWTP Zagreb, the plant-specific information required for the operational assessment is included in the plan. Thus this plan (found in the appendix of this report) can serve as an example for other modeling design studies.

15.2 Example of a measurement plan

The measurement plan as presented in the appendix consists of one table overview of all the measurements and all the measurement points. Information is also added on the type of sampling (grab samples or 24-hour composite samples), the number of samples, and how they should be prepared for lab analysis (filtered or total) and the weighted sample is analyzed. Also indicated in the table are references to indication codes used in the process flow diagrams together with indication codes either used by operators or from typical SCADA data acquisition systems. The measurement table is presented as a matrix and the number of samples collected

for every sample and sample point is automatically counted. This information can help to check if all the required samples were taken. Finally, in addition, the costs of each sample and the totals are calculated in the table for easy budget management. The following tables present the most common parts of the total measurement program.

Table 15.1 Sample table showing PFD and measurement locations

Location	Code	Location description	Comment
1	Q1	Raw sewage	Determined according to influent characterization method
2	Q2	Influent to primary settlers	Check internal flows
3	Q3	Settled sewage	Determined according to influent characterization method
4	Q4	Activated sludge tanks	TSS, ISS/VSS, COD, TKN, TP; check equal loading tanks
5	Q5	Effluent	BOD_5, TSS, COD, COD_F, TKN, NH_4, NO_3, TP, PO_4
6	Q6	Dewatered sludge	TSS, VSS, COD, TKN, TP
7	Q7	Internal flows	NH_4, PO_4

Table 15.2 Summary of water quality parameters required for model design studies

Parameter	Code	Unit	Comment
Total suspended solids	TSS	mg/L	20 µm coarse paper filtrated, oven dried (105°C)
Inorganic suspended solids (ash)	ASH	% of TSS	Incineration (550°C) of filtered and dried TSS
Dry matter	DM	mg/L	Total sample evaporated in glass bottle; includes dissolved salts
Inorganic fraction of dry mass	IM	% of DM	Incineration (550°C) of evaporated DM; includes dissolved salts
pH	pH		pH electrode measurement
Alkalinity	Alk	$mgCaCO_3/L$	
Calcium	Ca	mg/L	
Magnesium	Mg	mg/L	
Dissolved oxygen	DO	mgDO/L	DO electrode measurement
Total iron (full recovery)	TFe	mgFe/L	Total sample (solids and liquid)
Total phosphorus	TP	mgP/L	Total sample (solids and liquid)
Ortho-phosphate	PO_4	mgP/L	1,2 µm glass-fibre filtered (soluble fraction)
Total COD	TCOD	mgCOD/L	Total sample (solids and liquid)
COD glass-filtered	COD_{GF}	mg OD/L	1,2 µm glass-fibre filtered (soluble fraction)
COD micro-filtered	COD_{MF}	mgCOD/L	0.45 µm (membrane) filter (soluble fraction)
COD as VFA	VFA	mgCOD/L	Acetate + propionate, No poly-acetate filters or vacuum filtration
Acetate	HAc	mgCOD/L	Do not use poly-acetate filter or vacuum filtration
BOD_5	BOD	mgBOD/L	Total sample (solids and liquid)
BOD_5 glass-filtered	BOD_{GF}	mgBOD/L	1,2 µm glass-fibre filtered (soluble fraction)
Total Kjeldahl nitrogen	TKN	mgN/L	Total sample (solids and liquid)
Ammonium	NH_4	mgN/L	1,2 µm glass-fibre filtered (soluble fraction)
Nitrate and nitrite	NO_X	mgN/L	1,2 µm glass-fibre filtered (soluble fraction)

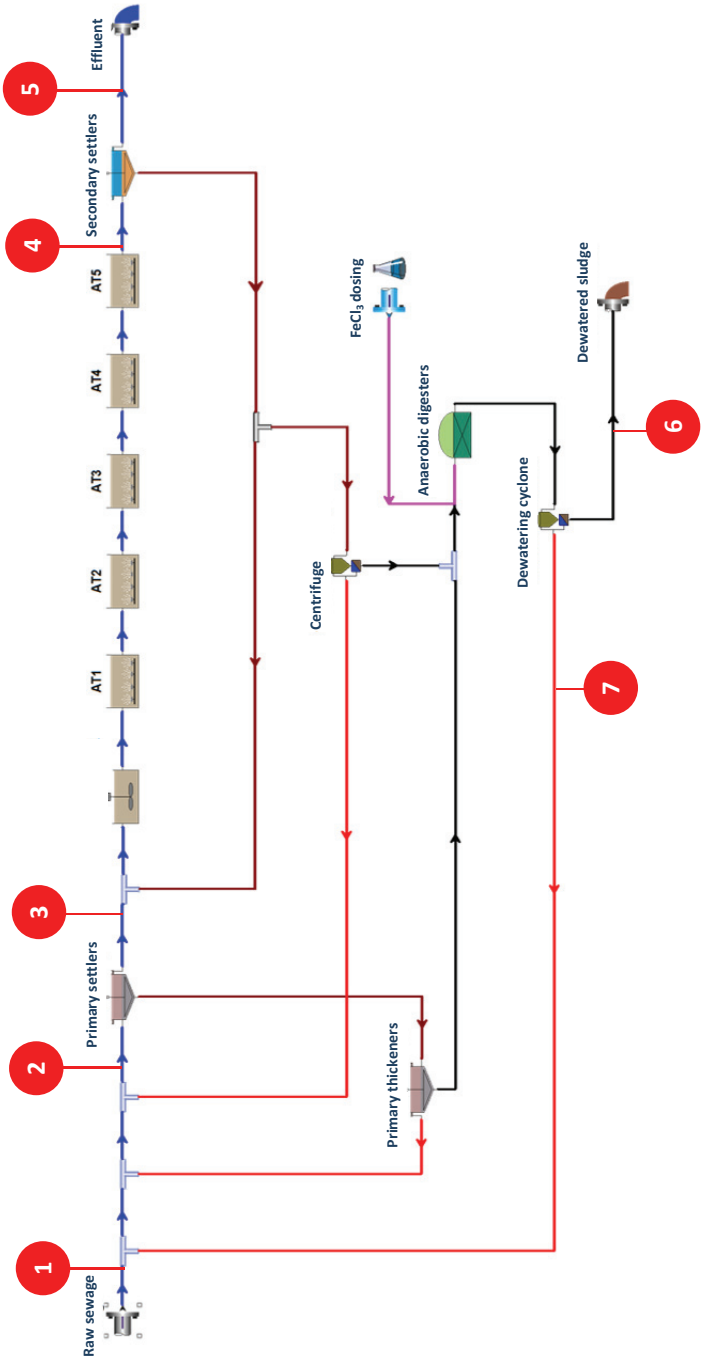

Figure 15.1 PFD of the WWTP indicating the measurement points

Table 15.3 An example of a sampling schedule for the WWTP Zagreb modeling study

		Description	Raw sewage	Influent to primary settlers	Settled sewage	Activated sludge tanks	Effluent	Dewatered sludge	Internal flows	Total additional analytical measurements per day	Total additional analytical measurements	Total Indicative costs (EUR)
		PFD code	Q1	Q2	Q3	Q4	Q5	Q6	Q7			
		WWTP code	na	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.			
		SCADA code	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.			
		Sample type	24h flow proportional	Grab sample	Grab sample	Grab sample	Grab sample	24h flow proportional	Grab sample			
Code	Volume	Preparation at sampling location										
TSS		Paper filtration (solid fraction)	**1**	**1**	**1**	**1**	**1**	**1**		6	6	**57.6**
ASH		Paper filtration (soluble fraction)	**1**		**1**	**1**		**1**		4	4	**24.0**
DM		Total sample								0	0	**0.0**
IM		Total sample								0	0	**0.0**
pH		Total sample	**1**				**1**			2	2	**12.0**
Alk		Total sample	**1**							1	1	**7.2**
Ca		Total sample	**1**							1	1	**7.2**
Mg		Total sample	**1**							1	1	**7.2**
DO		Total sample	**1**			**1**	**1**			3	3	**21.6**
TFe		Total sample								0	0	**0.0**
TP	100 mL	Total sample	**1**	**1**	**1**	**1**	**1**	**1**		6	6	**79.2**
PO_4	100 mL	Paper filtration (soluble fraction)	**1**		**1**		**1**		**1**	4	4	**28.8**
TCOD	100 mL	Total sample	**1**		**1**	**1**	**1**	**1**		5	5	**66.0**
COD_{GF}	200 mL	Paper filtration (soluble fraction)	**1**		**1**					2	2	**45.6**
COD_{MF}	200 mL	Paper filtration (soluble fraction)	**1**		**1**		**1**			3	3	**68.4**
VFA		Paper filtration (soluble fraction)								0	0	**0.0**
HAc		Paper filtration (soluble fraction)	**1**		**1**					2	2	**28.8**
BOD	500 mL	Total sample	**1**		**1**		**1**			3	3	**43.2**
BOD_{GF}	500 mL	Paper filtration (soluble fraction)	**1**		**1**					2	2	**28.8**
TKN	200 mL	Total sample	**1**	**1**	**1**	**1**	**1**	**1**		6	6	**79.2**
NH_4	50 mL	Paper filtration (soluble fraction)	**1**		**1**		**1**		**1**	4	4	**28.8**
NO_X	50 mL	Paper filtration (soluble fraction)	**1**		**1**		**1**			3	3	**21.6**
			18	3	13	6	11	5	2	**58**		**655.2**
			18	3	13	6	11	5	2		**58**	

15.3 General recommendations on sampling program design

A well-prepared measurement plan should not only give the type (e.g. COD, TKN, TP) and location (e.g. influent, effluent or aeration tank) of the measurements, but should also indicate what the measurement frequency is and how the sample should be taken, handled, stored, and processed in the laboratory for analytical measurements. The goal of the measurement planning is to communicate all this information to the staff responsible for acquiring the samples (the operators or plant technical personnel) and the laboratory personnel measuring the samples. Tables should be developed indicating all this information, preferably in one table overview. Also, a dedicated process flow diagram should be developed indicating all the precise measurement points related to the plan.

While designing the sampling campaign the following practical questions must be addressed as a minimum:

- WHY What is the purpose of taking the samples?
- WHAT Which parameters are to be sampled?
- WHERE Where are the sampling locations?
- WHEN When are the samples to be taken?
- HOW OFTEN What is the frequency for each sampled parameter?
- HOW How will the sampling be executed?
- BY WHOM Who will take the samples and who will analyse them?
- EQUIPMENT What will be used for collecting, storing and analysing the samples?

The 6 criteria for quality data are:

1. Collecting representative samples
2. Formulating the objectives of the sampling program
3. Proper handling and preservation of the water samples
4. Proper chain-of-custody and sample ID procedures
5. Field quality assurance
6. Proper analysis

The sample, in order to serve its purpose, has to be representative, reproducible, defensible and useful. It is also important to realize that proper understanding of sampling procedures is crucial for the design and success of the sampling program. This should not only be realized by the designer but also by all the others involved in the project. Therefore, it is recommended to organize a pre-operational meeting to communicate the measurement plan to all the personnel involved. During this meeting, it should be clearly stated what the goal of the project is, what will be done with the specific measurements and also the importance of acquiring accurate information. It might also be necessary to give (short practical) training in (grab) sampling techniques.

To ensure that the measurement plan is properly understood and executed, it should relate as much as possible to the actual routines of the plant and laboratory personnel. Where possible, references should be made to standard (practical) routines. For example, measurement locations on the PFD should be indicated in the same way as the operators give them. The designer often has his/her own preferred way of indicating the flows (e.g. in a flow diagram

using a logical numbering Q1 to Q8 to indicate flows in the mass balance), but in order to communicate this information effectively, the actual names of process units and flows should also be indicated in all the diagrams and tables.

Also, for analytical measurements the designer can have different names that do not always correspond with the names and practical methods applied in the laboratory. Therefore the measurement plan should also be checked with the laboratory personnel and the applied measurements routines (analytical methods and equipment).

For the purpose of a model design study, the plan measurements should be taken in dry weather conditions (ADWF). In practice it is not always clear if and when these conditions are met. Typically it is recommended to plan a day of sampling after three days without (major) rain. This means that the measurement plan can never be planned for an exact date but, however, incorporates a period of several days in which the plan can be executed depending on the weather conditions.

Some of the samples can be done automatically with 24-hour continuous (or quasi-continuous) flow proportional composite measurement devices. These automatic samplers collect samples over a 24-hour period and have to be started 24 hours ahead of time to provide a collected sample on the desired sampling day. It should be borne in mind that operators have to be notified the day before in order to be able to collect the samples at the given time. All the other samples taken on the day of sampling will likely be spot samples (also called grab, random, catch, individual, instantaneous, or snap-samples). Because the sampling program contains multiple measurement points and for each measurement points multiple parameters have to be measured, the amount of containers required to store the samples will rapidly increase; e.g. the amount of samples needed for the relatively small example of WWTP Zagreb presented in the previous chapter already adds up to 58 different samples. Also, it should be realized that for each sampling point the sampled volume should allow analytical measurements, washing of filters and vessels, possible spills and possibly duplicate or triplicate measurements in the laboratory.

In addition, the laboratory should be prepared to handle this large amount of samples in one day. Some of the samples can easily be stored to be measured at a different time. However, for other measurements it is critical that the samples are pre-treated (e.g. filtered) before they can be stored. Again, other samples cannot be stored at all (e.g. as the result of biological conversions which occur in the sample vessel). Also, this should be planned in cooperation with the laboratory personnel to avoid a possible work overload or inaccurate measurement results.

15.4 Sample identification and data sheet

As was previously pointed out, in the sample measurement plan there are 58 samples that need to be analyzed for one sampling day. Clearly this requires good administration and sample identification (sample ID). A list of the samples collected should be kept. The samples should be listed in one overview and include the information below. The individual sample containers should also be properly labelled.

The container label requirements are:

1. Sample site location
2. Container number if more than one container/sample
3. Name of sample collector

4. Facility name/location
5. Date and time of collection
6. Identification as a grab or composite
7. Test parameters - list analysis to be done
8. Preservative used

Proper administration of the sampling program is necessary for quality assurance. The following checklist can be used for making forms supporting the measurement campaign. Forms of this kind are often already available if they are used for the regular sampling program. It can be helpful to find out whether these forms and also the forms in the measurement campaign are available instead of constructing new ones.

• Project number	Assign a number to the sampling episode that can be used on the bottles to track the samples.
• Sampler	The sampler should print and/or sign their name.
• Project name	The project name, and if necessary, the project address.
• Contact person	The name of the person who is in charge of the project.
• Telephone number	The phone number where the project contact person can be contacted.
• Sample date	List the dates for grab and composite samples.
• Sample time	List the sample time for each sample.
• Sample type	Composite: collected over a 24-hour period or grab: taken as grab samples.
• Station location	The site where the sample was taken.
• Number of containers	List the number and type of containers for each sampling event.
• Analysis requested	List the analysis needed that is compatible with reference to official methods.
• Split samples	When a sample is split into different proportions for analysis, the receiving party can accept or reject the samples and they must sign this box.
• Comments	Note any special requirement for the samples for the laboratory.

15.5 Factors that can limit the validity of the results

When evaluating the results of the sampling program, there are several factors that can limit the validity of the results, such as:

- Missing values;
- Sampling frequencies that change over the period that is being recorded;
- Multiple observations within the sampling period;
- Uncertainty in the sample preservation and measurement procedures;
- Censoring the measurement signals;

- Small sample size;
- Improper data handling;
- Equipment inaccuracies.

It is always useful to check the sampling procedure and the measurement results against the above criteria.

15.6 Practical recommendations for analytical measurements

From the case studies carried out (measurement programs were executed at WWTP Varaždin, WWTP Zagreb, WWTP Čakovec, WWTP Vinkovci and WWTP Belišće), some typical recurring errors in the sample procedure or analytical measurements were documented and are listed below:

- The method used for analytical measurements of suspended solids is different for concentrated sludge samples and diluted MLSS samples; if the concentration is above a certain range, filtration, drying and weighing the solids will no longer be possible. Under these conditions the dry mass methods is often used (evaporation of the sample). This includes the soluble salts which for lower concentrated sludge samples in particular will not give representative results for total suspended solids. This should be taken into account especially when measuring activated sludge, thickened sludge and dirty water flows. For the more concentrated sludge flows (>20%) the dry mass analysis can be used.
- When measuring the filtrate of the sample containing suspended solids, the filter paper should not be washed other than with the sample itself and not with demineralized water, which will cause dilution of the measured filtrate.
- When handling samples containing suspended solids, it is important to take well-stirred homogeneous samples when taking the samples or pouring a sample into another vessel. It should be noted that TSS measurements are essential for the model design study but at the same time are very difficult to sample accurately. It is recommended to instruct all the personnel in how to take reliable TSS samples.
- When measuring the activated sludge characterization, all the samples should be taken out of the same vessel because it is important what the relative concentration of TSS, VSS, COD, TKN and TP in the sample is. Usually this is not a standard procedure and therefore all the personnel, especially those working in a laboratory who are responsible for analysing, should be properly instructed on this matter.
- A commonly known problem is the destruction of suspended solids which is necessary to analytically measure TP (and also COD and TKN). When the destruction procedure is not long enough, phosphorus measurements will be under-estimated. Especially when there are chemical precipitates (e.g. as a result of dosing of iron-salts) including phosphorus in the sample, destruction can be difficult.
- Measurements of nitrate in samples containing activated sludge (denitrifying heterotrophic bacteria) should be carried out quickly before denitrification occurs.
- Measurements of volatile fatty acids in samples containing activated sludge (denitrifying heterotrophic bacteria) is practically impossible for the same reason; VFAs are almost immediately oxidized by microorganisms. These types of samples are highly unreliable and should not be used. Measurement of volatile fatty acids in the influent is possible, but however should be immediately filtrated after sampling and stored under prepared conditions.

15.7 General methodology for quality monitoring

In general, there is a standard approach developed regarding the design of a water quality monitoring system from which the approach for a sampling program at a WWTP can be derived and which consists of 6 distinct steps:

Step 1: Evaluation of the existing information

- Wastewater collection and treatment;
  - Domestic wastewater;
  - Industrial effluents;
  - Combined sewer overflows and pumping stations;
  - Wastewater treatment plants.
- Lab-scale and pilot plants investigations;
  - Wastewater treatment plants;
  - Wastewater collection networks.

Step 2: Evaluation of the information expectations

- Water quality goals and objectives;
- Water quality problems and issues;
- Management goals and strategy;
- Monitoring role in plant management;
- Monitoring goals (as statistical hypotheses).

Step 3: Establishment of statistical design criteria

- Statistically characterise the population to be sampled;
  - Variation in quality;
  - Seasonal impacts;
  - Correlation (independence);
  - Applicable probability distributions;
  - Select the most appropriate statistical tests from those available.

Step 4: Design monitoring network

- Where to sample (from monitoring role in management);
- What to measure (from water quality goals and problems);
- When to sample (from specific circumstances);
- How frequently to sample (from needs of statistical tests).

Step 5: Develop operating plans and procedures

- Sampling routes and points;
- Field sampling and analysis procedure;
- Sample preservation and transportation;
- Laboratory analysis procedures;
- Quality control procedures;
- Data management and retrieval hardware and data base management systems;
- Data analysis software.

Step 6: Develop reporting procedures

- Type of format of reports;
- Frequency of report publication;
- Distribution of reports (information);
- Evaluation of report ability to meet initial information expectations.

Based on the above, a general wastewater and sludge sampling program can be carried out following the steps as listed below:

Step A: Definition of the purpose of sampling.

Step B: Determination of the type, scope and required accuracy of the analyses to be carried out.

Step C: Definition of the character of the samples to be collected.

Step D: Selection of the localities and sources to be sampled, and of the sampling points at these localities.

Step E: Determination of the hydraulic and other parameters relating to the subject of the sampling program.

Step F: Consideration of the occupational safety and hygiene of those collecting the samples.

Step G: Preparation of an optimal sampling programme.

Step H: Selection of sampling and measuring equipment suited to the sources to be sampled, and determination of its state.

Step I: Selection of the most suitable sampling technique in line with a fixed programme and selected equipment for the given source/site, including preparations, subsidiary measurements and observations. When implementing a sampling programme the techniques used should comply with a number of general requirements such as reliability, economy, repeatability and conservation.

Step J: Selection of appropriate procedures, sample-handling equipment and tools, transport from site to laboratory, storage after delivery.

Step K: Consideration of the most suitable methods of analysis on site and in the laboratory, the quickest possible interpretation of the analyses including reliability checks, and possible repetition of the sampling, and other factors likely to influence the accuracy or representativeness of the analyses.

Step L: Use of feedback whenever possible to modify a programme to an optimal sampling programme.

Step M: Establishment of the conditions necessary for the immediate use of the results and for their storage as primary sources of information in the future, whether for short- or long-term use.

Step N: Selection of the appropriate system for data management (i.e. handling, processing, transfer and manipulation)

Step O: Selection of the method of documentation to be used throughout the programme.

15.8 Final comments

Logistically, performing the measurement campaign in the design study is a complicated task, but at the same time it is the most important and critical step. Therefore it is recommended to carefully prepare the measurement campaign in all possible aspects. Also it is advised to accompany the first rounds of measurements together with all the personnel involved to make sure that all the instructions in the measurement plan are correct and as intended and properly understood by all the personnel. The analytical results of a sample are only as accurate as the quality of the sample taken. If your technique for collecting samples is poor, then no matter how accurate your lab procedures are, the results will be poor. By sampling according to set procedures, you reduce the chance of error and increase the accuracy of your sample results. It should be borne in mind that for a lot of samples the measurement campaign is a one-off opportunity; redoing samples at a different stage will often not lead to satisfactory results

because the activated sludge plant is a dynamic system continuously changing over time. With this in mind, for the design study the samples are collected simultaneously in such a way that their relative information can be used for plant assessment. Therefore, individual grab samples will often not be representative meaning that (missing or incorrect) measurement can often not be redone. In conclusion data collection through sampling at WWTP is a critical activity that needs a systematic and professional approach, as the design of sampling program is often a task for an experienced specialist.

Chapter 16

Activated sludge modeling protocol

16.1 General introduction

The following sections present the STOWA protocol for modeling activated sludge systems. This protocol was published by Hulsebeek *et al.* (2002) and is the generally applied modeling practice, as depicted by Figure 16.1.

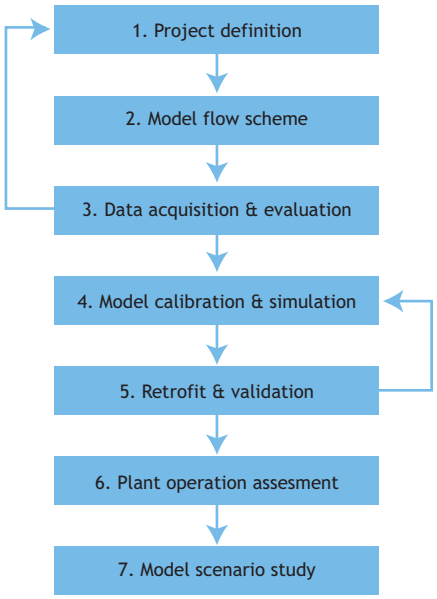

Figure 16.1 General modeling protocol for plant-wide wastewater treatment modeling as described in this guideline report. The colours in the scheme correspond with the detailed modeling protocol included in this report. For Section 4 the main structure of the STOWA protocol for modeling of activated sludge plants can be fully applied.

It has been extended here with new insights and a more detailed protocol for model calibration. Altogether, this revised protocol gives a logical sequence of the activities needed to model a wastewater treatment plant, such as the formulation of modeling objective(s), the plant description, data collection, construction of the model, flow characterization (using specific

protocols also presented in this report), model fine-tuning (calibration), model verification using other sets of data (validation), and reporting. This approach has been applied and verified in a number of modeling studies and widely adopted as good modeling practice.

In its form, the STOWA protocol may look different from what is proposed in this guideline report. However, it should be taken into account that these guidelines cover a much broader scope than only activated sludge modeling. These guidelines also cover information on data collecting, planning time and resources, presenting information, and methods for data evaluation, etc. Also, on a technical level, these guidelines have a broader scope; not only do they describe the activated sludge process (as does the STOWA protocol), but they focus on describing the full plant model (WWTP model or plant-wide model).

Whole plant (or plant-wide) modeling becomes possible when the appropriate modeling software is used. In this particular case the BioWin whole plant simulator is used. This software not only utilizes the activated sludge model, but also includes the Anaerobic Digestion Model (ADM) and several sub-models for widely-used process units such as centrifuges, thickeners, settlers, etc. From the perspective of plant-wide modeling, collecting plant data and calibrating the model is much more complicated. In that respect, the STOWA methodology presented here does not cover all aspects of plant-wide modeling. Besides the activated sludge system, the other WWTP process units should also be described and calibrated. In general, plant-wide modeling is more difficult because in the plant-wide model the error of one model (e.g. primary settling) is passed on to the other models (e.g. the activated sludge model and to the primary sludge). This can make the (precise) calibration of plant-wide models very difficult. If the design questions are purely related to the activated sludge plant, it is therefore not advisable to use the plant-wide model. In that case, it is better to focus on the description of the activated sludge system and measure all the incoming and outgoing flows. Thus the STOWA method presented here can be used effectively. On the other hand, the advantage of applying plant-wide models is that the overall information coming from the plant operation is much more valuable for troubleshooting, plant upgrade and design. A much broader understanding of the process is obtained via the plant-wide modeling in comparison with modeling of only the activated sludge plant. From the perspective of model stoichiometry and kinetics, the ASM is still the most complex model (e.g. more complex than the ADM or the other process units models typically used for description of a wastewater treatment plant). Also for that reason, placing emphasis on the correct modeling and calibration of the activated sludge model is well justified. The STOWA protocol is based on previously published work. It consists of a series of diagrams, some of which are in the form of decision charts (algorithms). The charts have a distinct order of approach; the topics are handled stepwise according to Figure 16.1. In the following sections the charts will be briefly discussed where the presented information is not self-explanatory.

16.2 Inventory of practical issues in the protocol

The development of the protocol was carried out with a special focus on the aspects of influent characterization, model structure and model calibration. Influent characterization is discussed extensively elsewhere in this guideline report. When developing the protocol a problem area was identified in the uncertainty about the required monitoring frequency and duration of the monitoring for characterization of the different water and sludge flows. Practical experience has shown that under typical Dutch conditions (flat sewers → long hydraulic retention time → low variability → daily averages sufficient), 24-h flow proportional samplings are sufficient for most purposes. In the original STOWA protocol, modeling and measuring rain events is mentioned and advice is given on how to measure these events for modeling purposes. However, this advice

was given based on very narrow experience of such projects so far. Reality shows that reproducing dynamic events in wastewater treatment, in particular rainwater events, is practically impossible for the simple reason that the causes for the event will never reoccur in the same way, e.g. each rain event is unique. From a modeling point of view, this further means that the model can never be validated (reproduced) in regard to such conditions. Accepting this, it is useless trying to measure (describe) these events and further calibrate a model based on such dynamic events. Practical experience has shown that much better and more reliable and reproducible results can be obtained when the efforts are focused on accurately calibrating the steady-state wastewater treatment model. When dynamics are applied to such a steady-state model, in most cases representative dynamics can be reproduced, which is useful for model design, plant upgrade and troubleshooting (Meijer *et al.* 2001a,b).

The duration of the measurement campaign necessary for the development of a properly working model depends on the accuracy of the required results (linked to a purpose for modeling, as described earlier in this report). In general 1 to 3 days of sampling is considered sufficient for standard situations, whereas for the development of models used in optimization studies and for control strategies, a longer measuring campaign is advised (3 to 7 days and at least 7 days, respectively).

Nowadays, even for the less experienced modeller, it is quite straightforward to set up a model for an existing WWTP, but it has been demonstrated that a visit to the plant and critical evaluation of collected data are essential for the success of the study. This is also because the plant operators have often modified the process configuration to a varying degree so that the supposed (assumed) operational conditions as designed no longer hold true. Often errors in flows or control set points are proven to be more sensitive to the model simulations then the model parameters themselves (Meijer *et al*., 2001b). Furthermore, in setting up the hydraulics of process units there is great uncertainty regarding the choice of the number of compartments and their configuration in the model. For example, to simulate the operation of a 'plug flow' pre-denitrification tank it was shown that only two compartments are sufficient. On the other hand, for an oxidation ditch type AS plant (Carrousel© type, often applied in the Netherlands), the number of compartments can easily exceed ten. The number of compartments may become even higher if the vertical oxygen gradient needs to be incorporated (e.g. when surface aerators are applied). From the modeling process control perspective it is important that the control structure is implemented in such a way that the process dynamics are properly described. It is not necessary that the control algorithm is identical to the one in practice; for example, one could increase the DO in the aeration tank in the model to simulate (match) the measured (observed) DO. Only if the process control itself is the main subject of the study, is it necessary for the control structure to be in accordance with the operations of the process control implemented in practice.

A typical step in activated sludge modeling is tuning the model parameters until the model results fit to the measured data (reality). This activity is known as model calibration. In many scientific publications and practical studies, calibration is used to make the model produce the desired (measured) results. The applied parameter changes found in the literature are highly variable, generally depending on individual choices, and do not give a unified idea of which parameters need to be changed and under which conditions. In the STOWA protocol it is proposed to calibrate the model in a stepwise procedure: (i) sludge composition and production, (ii) nitrification and (iii) denitrification, and (iv) (biological) phosphate removal. Parameters to be given preference for fine-tuning were also identified in the STOWA protocol. Meijer *et al*. (2001a,b) further improved this method according to Figure 16.2.

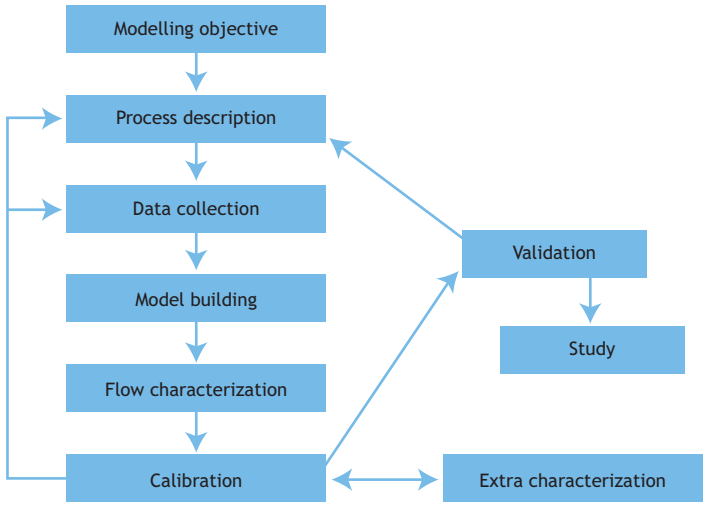

Figure 16.2 The main structure of the STOWA protocol for modeling of activated sludge plants. This protocol can be used within the framework of this guideline report to describe the plant-wide model.

In this procedure a unique set of model parameters (5 to 7 parameters were selected from the total of 52 parameters of the ASM TUDP model) is used to fit each of the four steps to the measured data (in terms of TSS, COD, TKN, NO_3 and TP mass balances). In this chapter this fitting will be further explained.

Quite often the duration of the calibration takes more time than expected. In general, this will happen when an attempt is made to calibrate the model based on raw unprocessed plant data. Raw data is not checked in a mass balance evaluation (for possible errors) and therefore has not been verified (whether the input and outflow measurements correspond). However, it is not confirmed if the model can be calibrated using the raw data in the first place, as the model itself is in fact a set of mass balance equations that can only reproduce correct flow and mass measurements. As a consequence of directly introducing raw data and thus not applying the mass balancing and data evaluation step during the calibration, it has been demonstrated that this approach is not possible and, therefore, either the input measurements need to be adapted or a set of new measurements needs to be obtained. When this is the case, the model calibration has to be carried out all over again. Therefore, it is strongly recommended to pay additional attention to the set-up of the plant model before starting a detailed measurement and calibration program. With regard to the validation, sometimes data is used from a period highly comparable with the period from which data for input and calibration was obtained (e.g. 1 week later). Of course this is not the best practice; data from a clearly different period (e.g. summer and winter conditions) should be preferably used for validation.

16.2.1 Protocol structure

The main structure of the protocol is shown in Figure 16.1. The protocol is based on the results of the inventory of the existing protocol structures. In the next sections the protocol structure is further detailed per phase. The figures also give the amount of time required for the different phases, this time based on the time taken by users who had calibrated at least one WWTP

model before. The protocol structure is dependent on the objectives of the study. In the protocol three different phases are distinguished:

- System choice during the basic design;
- Optimization studies for existing WWTP;
- Development of control strategies for existent and new WWTP.

The required level of specification of the model and the necessary monitoring frequency of the relevant flows are dependent on the objectives mentioned above. For the development of control strategies the level of specification is the highest.

16.2.2 Modeling objectives

In the five case studies carried out in Croatia, the general modeling objectives were; (1) describing the plant performance, (2) optimization of plant operation and, (3) design-based support for plant upgrade. While the first step can be characterized as largely identical for all model-based design projects (and also the scope of the STOWA guidelines), steps two and three are much more plant-specific and therefore will be different for each design case. The modeling protocol therefore is mainly focused on the steps leading to a calibrated model that can be used to reliably describe the plant performance.

16.2.3 Process description

Once the objectives of the study have been set, a definition of the relevant process components can be made. In the guidelines presented here it is proposed to construct a process flow diagram (PFD) outside of the modeling environment (typically not in the software for reasons described elsewhere in this document). Often it is not required to model the complete WWTP. Only those parts that fit within the described process are useful for consideration in the model (Figure 16.3).

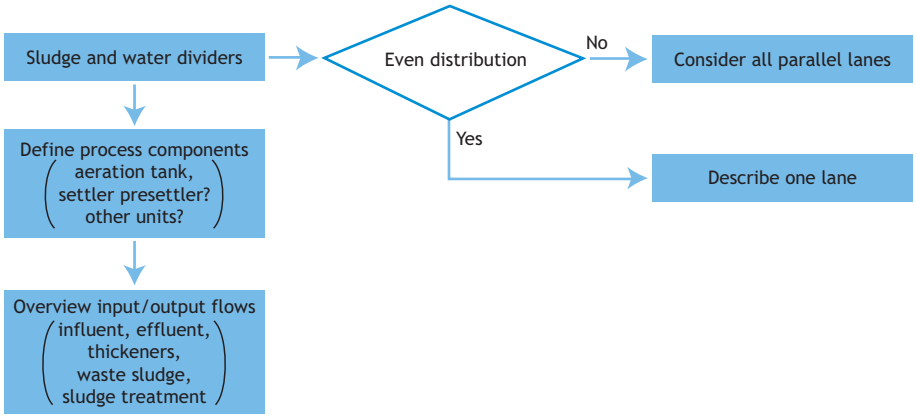

Figure 16.3 Process description

In this document it is recommended to use a systematic approach to divide a wastewater treatment plant in sub-systems (general subsystems are preliminary treatment, primary treatment, the water line and the sludge line). The water line can be modelled as described in

this chapter. If the distribution of sludge/water in the activated sludge system with parallel lanes is well balanced, all the treatment lines can be modelled in the same way. If the distribution is not well balanced, each line has to be modelled separately. This has to be checked at the treatment plant itself. The same issue needs to be addressed regarding the collection and redistribution of return activated sludge coming from the settlers to the beginning of the aeration tanks. Also here, an uneven redistribution will result in a considerable difference between the operation of the different parallel activated sludge units. Typically, this information will be included in a properly described PDF. When modeling the water line, it should be realized that when the two lines need to be modelled separately this also means twice the amount of plant data to be measured (basically two simulation studies). In general it is a simpler approach to only measure one of the two (or more) parallel lines within the main water line, if possible.

16.2.4 Data collection and data verification

The composition and the volume of the flows entering the different process units, as well as the volume of the process components, are defined in this phase (Figure 16.3). These guidelines give considerable attention to the collection of the required plant information. In general, a lot of the WWTP data is readily available (daily average concentrations, flow patterns etc.). It is recommended in the protocol to start by deriving the water quantity and quality of the streams that are of interest from the available data. If necessary, relevant missing data can then be obtained via additional observations. To be able to construct an efficient measurement plan it is recommended to make a preliminary simulation based on raw plant data. After a first set of (non-calibrated) simulations a better observation (monitoring) program can be developed. Using mass balances (flows, COD, dry matter, nitrogen and phosphorus) the available data should be checked (Meijer *et al*., 2001b). If the mass balances are not correct, it is necessary to perform additional monitoring of the plant. If these mass balances in the primary data are not correct (do not close), calibration will by definition lead to erroneous results (Meijer *et al*., 2001a; Nowak *et al*., 1999). Meijer *et al*. (2001b) presented a detailed overview of this procedure including the use of data reconciliation techniques. Mass balances for phosphorus (or iron if chemical P-removal is applied) can be made simply by measuring influent and effluent total phosphate concentrations, flows and by measuring the phosphate in the (excess) sludge. Based on this, the mass balance for phosphorus in the mass of excess sludge can be checked or obtained if proper data is unavailable. Since the assumed SRT is highly sensitive in the simulation model it is essential that this mass balance is scrutinized, even if one is not interested in modeling P-removal. The (measured) flows can be checked and unknown flows can be calculated by using a combination of the measurements of the concentration of suspended solids in the reactor or incoming flows and the flows that are known. The STOWA protocol proposes a system of checks and balances based on the total phosphorus balance. This approach is also explained in the guidelines presented here. The method is used to accurately identify the sludge retention time of the activated sludge, based on the sludge production. For accurate activated sludge modeling the sludge production should be known within a margin of 5% accuracy. To accomplish this, it should be possible to close the phosphate balances over the influent of the aeration tanks, the effluent and the waste activated sludge production. One practical problem related to the measurements of the sludge production is related to the fact that the operation of secondary settling tanks can be very dynamic. As a result, the concentration of return activated sludge and waste sludge varies considerably over the course of the day. To overcome this inaccuracy, the phosphate balance and thereby the sludge production (expressed in kilograms of phosphorus) can be calculated from the relatively stable influent and effluent flow. By also measuring the phosphorus content of the activated sludge, the TSS concentration of the waste sludge can be calculated backwards. How to apply mass balances is explained earlier in these guidelines.

16.2.5 Some examples for data evaluation

The STOWA protocol recommends that the TSS, COD and TN mass balances are calculated over the activated sludge system. The (measured) flows can be checked and unknown flows can be calculated by using a combination of the measurements of the concentration of suspended solids in the reactor or incoming flows and the flows that are known:

$$\mathrm{Ga} = \frac{(\mathrm{Qrs} * \mathrm{Grs}) + (\mathrm{Qi} * \mathrm{Gi})}{\mathrm{Qi} + \mathrm{Qrs}} \quad (16.1)$$

where:

- Ga = suspended solids in a reactor (kg MLSS/ m^3)
- Qrs = flow return sludge (m^3/d)
- Grs = suspended solids in return sludge (kg MLSS/m^3)
- Gi = suspended solids in the influent of a reactor (kg MLSS/m^3)
- Qi =influent flow (m^3/d), depending on the plant configuration.

For nitrogen, the mass balance is more complicated,

$$Ni = Ne + Ns + Nd \quad (16.2)$$

where:

- Ni = total nitrogen load in the influent (kg/d)
- Ne =total nitrogen load in the effluent (kg/d)
- Ns =total nitrogen load in the excess sludge (kg/d)
- Nd = total denitrified nitrogen load (kg/d)

For nitrification the following equation is valid:

$$Nki = Nke + Ns + Nn \quad (16.3)$$

where:

- Nki = total Kjeldahl-nitrogen load in the influent (kg/d)
- Nke = total Kjeldahl-nitrogen load in the effluent (kg/d)
- Nn = total nitrified nitrogen load (kg/d).

In practice Ni and Nki are often the same.

The COD balance gives the oxygen uptake in the treatment plant:

$$CODi = CODe + OUR + (Nd \times 2.86) + (Qs \times Gs,org * 1.42) - (4.56 \times Nn) \quad (16.4)$$

where:

- CODi = COD load in the influent (kgO_2/d)
- CODe = COD load in the effluent (kgO_2/d)
- OUR = oxygen uptake rate (kgO_2/d)
- 2.86 = oxygen reduction-equivalent (1 kg NO_3-N is equivalent to 2.86 kgO_2)
- Gs,org = the concentration of volatile suspended solids in the excess sludge (kgVSS/m^3)
- Qs = the daily volume of excess sludge (m^3/d)
- 1.42 = the factor to convert the amount of VSS to COD equivalents (kgCOD/ kgVSS)
- 4.56 = conversion factor for the calculation of oxygen uptake during nitrification.

The OUR obtained can be used to check the supposed aeration efficiency of the plant (kgO_2/kWh), to detect possible errors, or can be used in the model simulation set-up of the aerators. Further explanation on these balance equations can be found earlier in this guideline report.

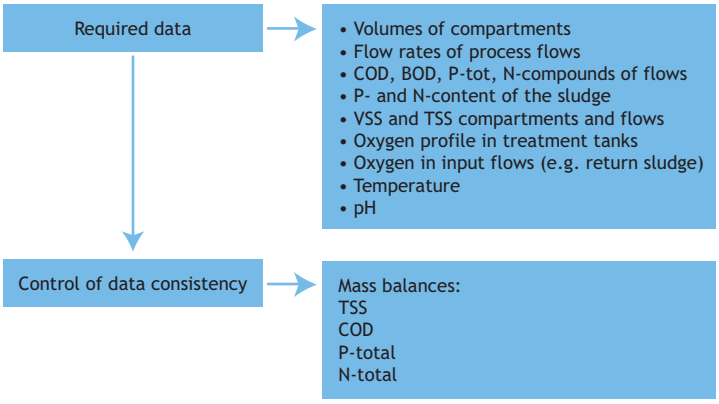

Figure 16.4 Data collection and data verification

The list of measurements that are required to perform a simulation study in Figure 16.4 serves as a general indication only. This list is used to create a simplification of the reality and is not useful for direct practical application. The measurement campaign should always be tailor-made and based on the actual conditions at the wastewater treatment plant. Therefore the guidelines presented here propose a structured methodology based on which it is possible to construct a tailor-made measurement campaign.

16.2.6. Model structure

Figure 16.5 describes the main components of the model structure.

The model structure for the hydraulics of the WWTP is based on the process description. The definition of the model structure is an essential phase of the modeling study. There is no point carrying out calibration if the model is not properly defined. Errors in e.g. flow rates of recirculation pumps are more sensitive than errors in model parameters (Meijer *et al.*, 2001a). The model definition describes the different process components. Aspects such as the number of compartments, aeration configuration, settling and control will be taken into account during the definition of the model structure. In this phase it is essential to evaluate the oxygen gradients in the aerated tanks in a horizontal and vertical direction. Certainly for surface aeration this needs to be checked in order to set up a proper compartmentation. For bubble aeration a visual inspection of the operation of the aeration is recommended. It is equally important to control all the flow splitters at the plant for exact operation. The modeling of the settler depends heavily on the use of the model. The amount of suspended solids in the effluent should be made equal to the measured values (thus fitted in the model). Denitrification in the sludge line can be checked by comparing the nitrate in the return sludge and effluent. Denitrification can be best calibrated by applying a virtual (modelled) tank in the return sludge

line. It should be noted that, in general, the denitrification in the settler is more important than the exact height (level) of the sludge blanket in the settler as a function of time; a rough estimate of this value is often sufficient. However, the settler models should also take into account the biological processes. Otherwise, when a settler model is used which has no volume, an extra tank in the return sludge line for denitrification can be used, although it should not be larger than the estimated volume occupied by sludge in settler tanks as this would lead to a too high amount of total sludge in the activated sludge system. Multi-layer secondary sedimentation and conversions in the sludge blanket can be included in the model. Due to general shortcomings in the exact understanding of the sedimentation process, suspended solids in the effluent are in general set (fixed) in the model by the user and cannot be predicted. Based on empirical models (extensively described in these guidelines) it is possible to assess the secondary tank design outside of (separately from) the activated sludge model.

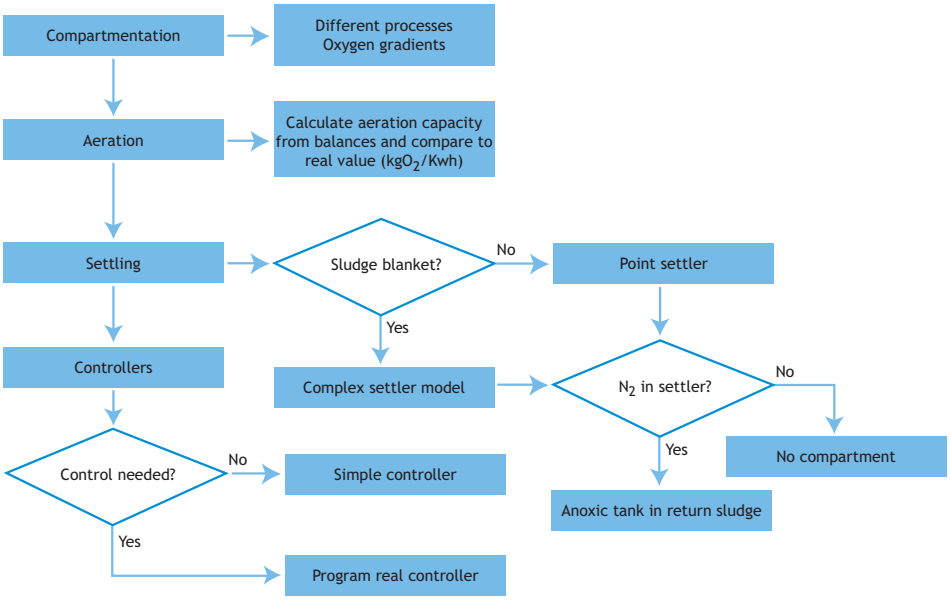

Figure 16.5 Model structure

In general it is required that the effects of the control incorporated in the model are in accordance with the operations of the process control in practice. One could e.g. define flow rates or DO based on the actually measured values instead of implementing the controller. In theory, for steady-state simulations, introducing controllers in the model is not necessary. All average conditions can be reached by setting up the required flows and oxygen set points. However, in the process of calibrating the model, it can be convenient to have certain parameters controlled against their measured value (e.g. TSS in the aeration tank). This avoids the situation where when calibrating one aspect the other aspect has to be changed and so forth; a controller can help do this automatically. Therefore in general, in most cases the introduction of simple controllers can satisfy modeling tasks (e.g. maintaining the SRT). However, if the goal of the modeling study is to investigate the process control, then a real

controller has to be implemented in the model (e.g. TSS control). When building an activated sludge model the aeration tanks are the primary focus. Mostly, compartmentation is used to distinguish between different processes of the activated sludge (e.g. anaerobic, and anoxic or anaerobic). In the case of plug-flow systems, the actual tank will be described as a number of smaller tanks in series. Also the presence of oxygen gradients (e.g. in carousel type or aeration controlled tanks) can require different compartments in the model. For completely mixed reactors, one should check whether the reactor is indeed fully mixed in reality. Dissolved oxygen (DO) profiles are used to evaluate whether more than one completely mixed reactor is needed (over length and depth of the reactor). In the case that the concentration is near substrate affinity constant (Cs≈Ks), the rate becomes concentration-dependent and the need for 'compartmentation' needs to be evaluated.

When aeration tanks are modelled, an oxygen model is also used (often working in the background). Aeration can be built into the model relatively easily by the introduction of a DO control(ler). In order to fix the COD mass balance, it is better to use the exact air input based on kgO_2/kWh and measured energy usage (kWh). In the case of tanks that have not been well mixed, oxygen affinity is used to calibrate the aerobic (and anoxic) conversions. The information to carry out this calibration accurately is obtained from the previously discussed mass balances. For different aeration types (e.g. surface aeration, bubble aeration) there are different oxygen models. To apply the oxygen model, it is required to have an energy input or blower capacity. To estimate this value for application in the model, it is recommended to calculate the COD and N balances, and from there to estimate the required oxygen uptake rate. This value can be recalculated as an aeration capacity to be compared with the model calculations. These guidelines apply the same methodology. However, in most simulation studies the exact aeration capacity is not of interest, in contrast to, for example, when studying the sludge production or the effluent quality of an activated sludge plant. Therefore most simulation software has the option to turn off the oxygen model and implement a direct oxygen set point. By doing so, blower aeration capacity limitations are no longer possible and the model is also easier to solve without running the oxygen model in the background. From the point of view of model solvability (this is the result of mathematics and not the capacity of the CPU), only when a steady-state solution is obtained is it advisable to simulate aeration capacity. A decade ago the calculation power of personal computers limited the model calculations and the amount of compartments (more compartments means more differential equations = more CPU) introduced determined the speed of the simulation. However, with the increased calculation capacity of personal computers this is no longer a problem. Therefore, it is recommended to introduce compartments for all potential conditions (within reasonable assumptions). This avoids the risk that later on in a modeling project the model has to be reconstructed.

16.2.7 Characterization of the main flows

Wastewater characterization is a decisive activity in this approach (Figure 16.6). It largely determines the model input and thereby the model results. Besides the influent data, some effluent data also needs to be measured. The data collection is structured based on a general protocol and specified in detail for each of the studied plants. It is advisable to get a good quality data set routinely measured by the staff of the plant. By using historical data and/or specific measurements, the important process flows (influent, effluent and the activated sludge) can be characterized. However, it is often necessary to reform the specific influent and effluent and activated sludge measurements needed for the characterization. How this can be done efficiently is explained elsewhere in this guideline report. The guidelines for influent characterization are the basis for the flow characterization (Roeleveld, 2001). When the model is used for a system choice, daily average concentrations of influent and effluent are sufficient. If

the model is used for optimization or the development of control strategies, specific data (2 or 4-hour composite samples) is required. In this case other flows in addition to the influent and effluent (e.g. recirculation flows) should also be sampled. Figure 16.6 gives the procedure for the characterization of the main flows.

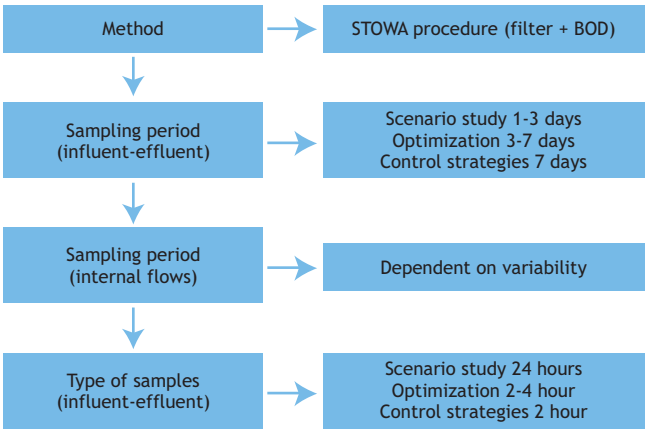

Figure 16.6 Procedure for the characterization of the main flows



Figure 16.7 UNESCO-IHE participants during the influent and sludge characterization, which was part of the model-based study on improvement of the WWTP of the Shell oil refinery in Godorf, Germany.

Sludge characterization based on laboratory bench scale batch tests can be performed as a part of the sampling program, preferably using fresh sludge from the treatment plant. Care should be taken that the sludge is taken from the locations at the treatment plant which best correspond with the purpose of the batch test. If possible, batch tests should be performed at the treatment plant itself; if not, sludge can be transported and stored in a dark and cold place for up to a couple of days. It should be checked to see if there is any difference using the stored and fresh sludge. Batch tests should be performed at the same temperatures and pH as observed in the process. Once influent and sludge characterization has been carried out, the simulation and calibration steps can be performed.

16.2.8 Calibration

When the model set up is properly defined and the required model input data has been implemented, the first simulation can be done with the aim of checking to see if the model can be fitted to the measured data. If the results of this initial test simulation indicate that a major adjustment of the model parameters is necessary, experience has shown that it is most likely that a structural error (gross error) is present. Typically, this error is either in the model setup of the flow scheme, or in the data to which the model needs to be fitted. In that case, a monitoring program should not be started before the model structure has been checked. This is done by checking the flow and TP mass balances as described elsewhere in these guidelines.

If the effluent quality is initially not well predicted by the model, a sensitivity test for several of the process parameters (flow rates, set points etc.) can be carried out. A sensitivity analysis can be done manually by changing various parameters (e.g. by increasing the parameter value by 10-15% of the original value) and repeating simulations by the model. The change in the model results as the result of the parameters variation indicates whether is this parameter sensitive towards the specific process that needs to be fitted to the measured data. Comparison of the sensitivity of different model parameters shows which parameter is best suited for calibrating the desired model process (e.g. nitrification or waste sludge production). Testing the sensitivity of the preselected set of the model parameters aims to detect where potential errors in the process have been made. By this method, the number of components that need to be analyzed and the number of analyses can be reduced. In the protocol it is recommended to make a calibration according to the procedure given in Figure 16.8.

Further it is indicated which model parameters can be adapted in order to calibrate the model properly:

- Activated sludge composition and waste sludge production: influent X_S and X_I, iN_X, iN_I;
- Ammonium concentration in the effluent: K_{O2}, k_{NH4}, b_A;
- Nitrate concentration in the effluent: η_{N03}, b_H, K_{O2}, K_{NO3}, K_{OH}.

Firstly the N content of the sludge in the model has to be made equal to the measured activated sludge characteristics. If not, the error will accumulate in the nitrification/denitrification process. Since the N content of biomass is approximately known, it is recommended to adjust the N content of the inert particulates (X_I). This will also affect the influent composition (characterization). If the SRT is checked and is accurate (by closing the total phosphorus balance as discussed earlier in this report), the sludge content in the activated sludge system can be calibrated. This is influenced mostly by the yield and decay rates and also by the assumed fraction of influent inert particulates. It is recommended to use the influent inert particulates (thus not the yield or decay parameters in the model) as the only calibration factor to fit the sludge production, since this inert influent fraction is usually the most uncertain one.

The nitrification process is calibrated against the ammonium effluent content. After checking that a wrong prediction of the ammonium is not caused by a wrong DO in the model or by too low alkalinity, the affinity coefficients for oxygen or ammonium can be used to adjust the ammonium in the effluent. For adjusting the ammonium in the effluent at different temperatures it is recommended to use the decay factor rather than the growth rate, because of the larger uncertainty associated with the former parameter. It is important to pay attention not to calibrate the model further than the accuracy of the actual measurements.

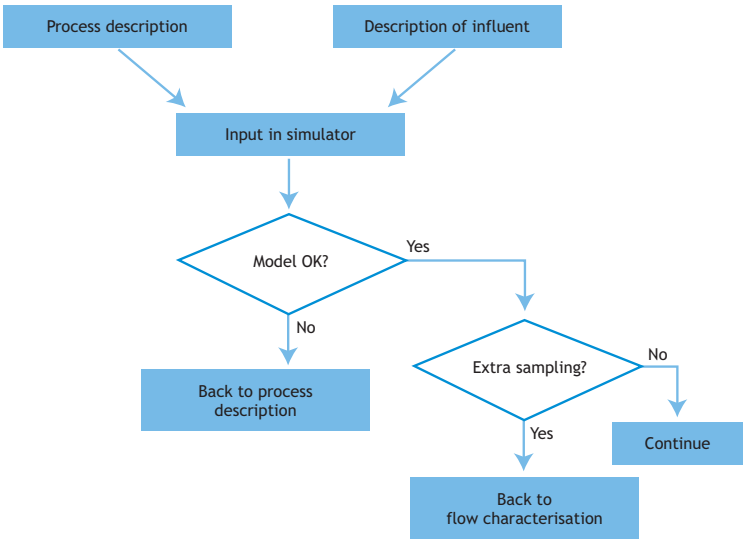

Figure 16.8 Calibration protocol

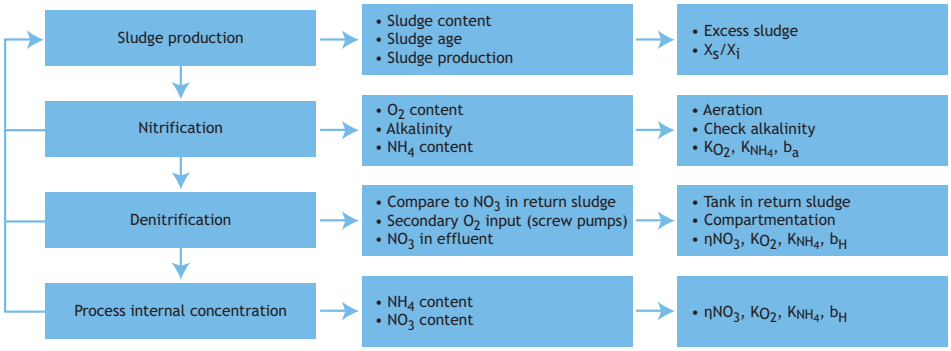

Figure 16.9 Order of steps during the calibration

The denitrification process can be calibrated against the nitrate in the effluent. First it should be checked whether the model set up is correct (denitrification in the return sludge, aeration by e.g. screw-pumps or overflow weirs). The most appropriate parameter to calibrate the

denitrification is the anoxic reduction factor for anoxic growth and hydrolysis processes, or possibly the heterotrophic decay factor. It is best to select the most sensitive parameter out of those suggested. If in the nitrification/denitrification calibration parameters are changed, the sludge production will also probably be slightly changed. Therefore an iterative approach needs to be used. Usually one iterative loop is sufficient. In order to further calibrate the model, internal concentrations in the different process units can be calibrated.

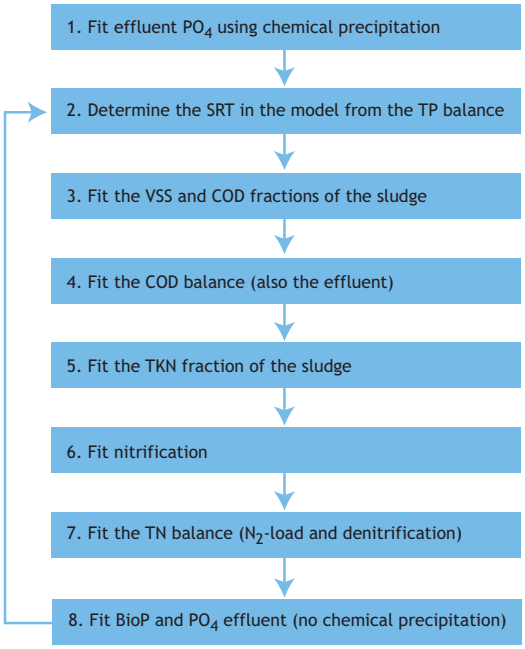

Figure 16.10 Model protocol for calibration of the activated sludge plant (Meijer *et al.,* 2001). This protocol is an updated version of the STOWA protocol and suitable for ASM2d, ASM3 and TUDP type models (including BioWin).

16.2.9 Introducing a stepwise calibration for nutrient removal

Step 1. Fitting the TP balance

In the model the activated sludge system needs to be balanced for total phosphorus according to the measurements. Therefore, the incoming and outgoing flows and mass loads need to be reproduced based on the total phosphorus balance. At this stage of the calibration, the model does not yet predict effluent orthophosphate (PO_4). Therefore, as a temporary measure in the model, orthophosphate in the effluent can be artificially controlled against the measured effluent value by introducing chemical precipitation. Total phosphorus in the effluent can be calibrated by adjusting the loss of solids from the clarifier. When regulating the waste flow, total phosphorus in the system can be introduced into the mass balances, thereby fitting the model. By doing this, the SRT is also fixed in the model (waste activated sludge – WAS, is fixed). By fitting the total phosphorus balance and setting the SRT the first step of the calibration is completed.

Step 2. Fitting the solids COD_X balance

Because the COD_X balance is a non-conserved balance, an incorrect influent COD load will generally be compensated for by the oxygen consumption of the activated sludge process. In the previous step the SRT was fixed according to the TP balance. Hence, the total amount of MLSS in the WWTP is (mainly) determined by the influent X_I/X ratio as inert COD (X_I) accumulates in the WWTP. By adjusting the influent X_I/X ratio the model can be fitted to the measured TP_X/COD_X ratio coming from the activated sludge characterization (described earlier in this report). Hereby, all model uncertainties related to the production of X_I and the influent characterisation are lumped together in the influent X_I/X ratio. This is justified because the amount of inert COD in the influent cannot be analytically measured and therefore is based on estimation.

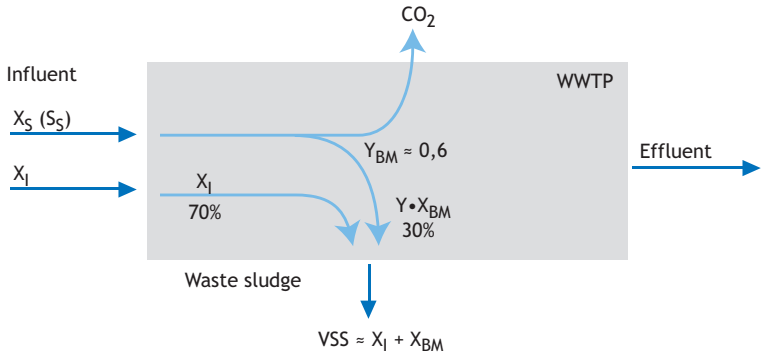

Figure 16.11 Schematic relation between the influent COD load (X_S and X_I) and sludge production (VSS or COD). Inert particulate material is the main contributor to waste sludge production and therefore fitting the fraction X_I/X_S in the model immediately affects waste sludge production.

Step 3. Fitting soluble COD_S in the effluent

COD_S in the effluent is fitted by adjusting the influent ratio S_I/S_F. Increasing S_I in the influent directly affects the effluent concentration, as S_I is not converted. In doing so, S_A in the influent is not changed, as this model component is directly measured from the influent VFA.

Step 4. Fitting the TKN balance

Like COD, TKN is a non-conserved compound. An incorrect (influent) TKN load is generally compensated for by the oxygen consumption of the activated sludge process via the nitrification process, and N_2 production via denitrification (as explained earlier in this report).

The TKN_X/COD_X fraction of the activated sludge can be obtained from the activated sludge characterization. TKN_X is fitted by increasing the fractions iN_{XI} and iN_{XS} thereby obtaining the measured sludge fraction (typically in the range 0.06 gN/gCOD).

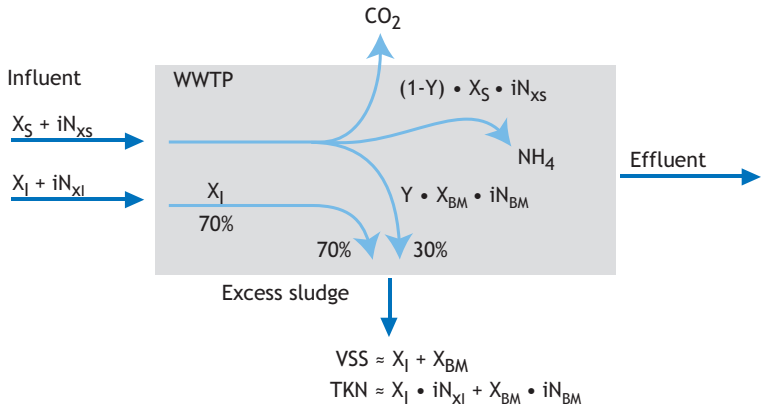

Figure 16.12 Schematic relation between the influent organic particulate TKN_x load (iN_{xs} and iN_{xi}) and sludge production (VSS or COD). By increasing iN_{xi} in the influent, the mass of particulate nitrogen in the sludge also increases and thereby the activated sludge fraction TKN_x/VSS increases. To maintain the same TKN influent load, at the same time iN_{xs} needs to be reduced. This means slightly less ammonia is released and therefore the oxygen consumption for nitrification is less.

Step 5. Calibrating the net nitrified load

To simulate ammonium in the effluent, DO set points in the aeration tank are usually adjusted. Thus the original aerated volume is not changed. Otherwise it is permissible to adjust the growth rates of nitrifying organisms. Usually calibrating nitrification is straightforward.

Step 6. Calibrating the net denitrified load

The net denitrification in the activated sludge process is fitted by increasing the parameter K_{O2}. This model parameter limits the anoxic process in the presence of oxygen (according to a Monod equation). By increasing this value the anoxic process does not 'see' that oxygen is present in the aeration tank and will continue to denitrify and therefore simultaneous nitrification and denitrification is stimulated in the activated sludge process. A practical reason that anoxic bacteria in a anaerobic tank do not 'see' oxygen can be explained by the fact that oxygen can be depleted inside sludge flocks (Figure 16.13) as the result of an oxygen gradient towards the inside of a flock. The same effect can also occur on a larger scale, when an aeration tank, often aerated from the surface, is not properly mixed. Under these conditions sludge will settle to the bottom of the tank where anoxic and even anaerobic conditions can occur. To a lesser extent this also happens in carousel-type reactors; by artificially introducing an oxygen gradient in one tank, zones that include and exclude oxygen are created, thereby simultaneously nitrifying and denitrifying. Figure 16.14 shows how the typical Monod equation responds when increasing or decreasing the K_{O2} value; when the value is increased the relative reaction rate (0-1) increases speeding up the anoxic process while the oxygen concentration remains the same. The same principle also accounts for anaerobic processes which are suppressed by increasing K_{O2}.

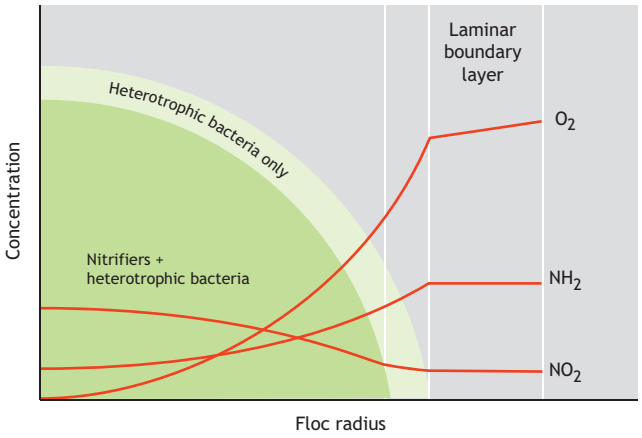

Figure 16.13 Diffusion limitation in activated sludge flocks (Siegrist et al., 2006)

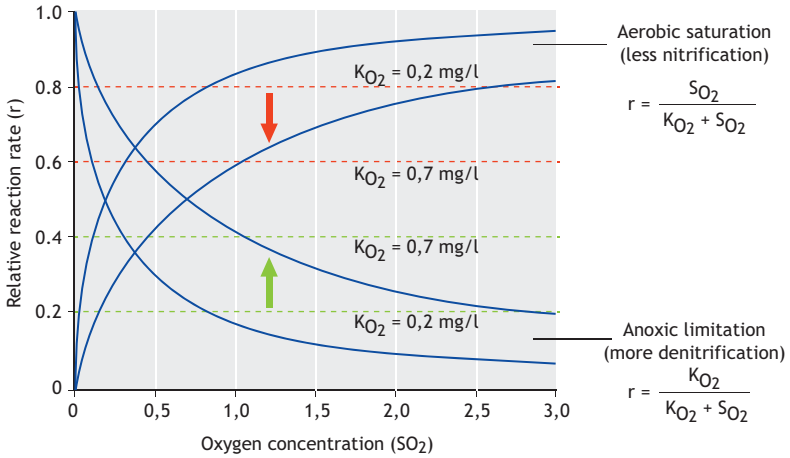

Figure 16.14 Fitting model kinetics simulating diffusion limitation in activated sludge (Meijer et al., 2001)

The model can be used beforehand to evaluate which internal process concentrations are sensitive. These can then be measured. Also, if necessary for the calibration of the internal flows, the effluent calibration has to be checked again in an iterative loop. Table 16.1 shows the suggested range of the values and the default value for the different model parameters for ASM 1.

Table 16.1 Suggested range of values and the default value for the different model parameters for ASM 1

Parameter	Description	Default	Range	Unit
YH	Heterotrophic yield	1	0.46–0.69	gCOD/gCOD
YA	Autotrophic yield	0	0.07–0.28	gCOD/gN
iNX	Mass of N per mass of COD in biomass	0		gN/gCOD
iNI	Mass of N per mass of COD in inert material	0	0.02–0.10	gN/gCOD
μH	Maximum specific growth rate for heterotrophic biomass	6	3.0–13.2	L/d
KS	Half saturation coefficient for heterotrophic biomass	20	10–180	$gCOD/m^3$
KOH	Saturation/inhibition coefficient for O_2 and for het. biomass	0	0.01–0.20	gO_2/m^3
KNO	Saturation/ inhibition coefficient for nitrate	1		$gNO_3\text{-}N/m^3$
bH	Decay rate for heterotrophic biomass	1	0.05–1.6	L/d
bA	Decay rate for autotrophic biomass	0		L/d
ηg	Reduction factor for anoxic growth	1		–
ηH, ηNO3	Reduction factor anoxic hydrolyses	0	0.6–1.0	–
kH	Maximum specific hydrolysis rate	3	1.0–3.0	L/d
KX	Saturation coefficient for particulate COD	0	0.01–0.03	gCOD/gCOD
μ	Maximum specific growth rate for autotrophic biomass	1	0.34–0.8	L/d
KNH4	Saturation coefficient for ammonium	1		$gNH_4\text{-}N/m^3$
KOA	Saturation coefficient for O_2 for autotrophic biomass	0		gO_2/m^3

In specific cases kinetic rates of the activated sludge (growth rates) need to be measured in order to accurately calibrate an activated sludge plant. This can be the case for industrial wastewater with atypical composition on which only specific bacteria can grow which have adapted to the specific substrate (or e.g. inhibitory) conditions. Often such organisms will grow at different growth rates to those modelled in the ASM. Lab scale batch tests that can be performed to identify dissimilar bacterial growth are the following:

- Respirometry test;
- Nitrification test;
- Denitrification test;
- Anaerobic P-release;
- Aerobic P-uptake;
- Anoxic P-uptake;
- Endogenous P-release;
- PAO denitrifying activity.

A final comment is that the calibration procedure is an iterative process that should terminate when the required results are more or less obtained. The model will never completely reproduce the measurement data (reality). The required accuracy of the predictions will be determined by the purpose of the modeling and the usual margin of error in the input data (e.g. measurement or analytical inaccuracies). There is no need to aim at unnecessarily high accuracy as this will increase both the efforts and costs substantially and will not bring any added value to the project.

16.2.10 Model validation

After successful calibration, the model can be validated prior to the actual model-based investigation of 'what if' scenarios. Therefore, sufficient data from different monitoring periods should be available. Typically one set of data is used for the calibration and another is used for

the validation of the calibrated model results. It is e.g. possible to use plant data from a different temperature period for validation, or use data from a completely different set of conditions (e.g. when primary settlers are out because of maintenance).

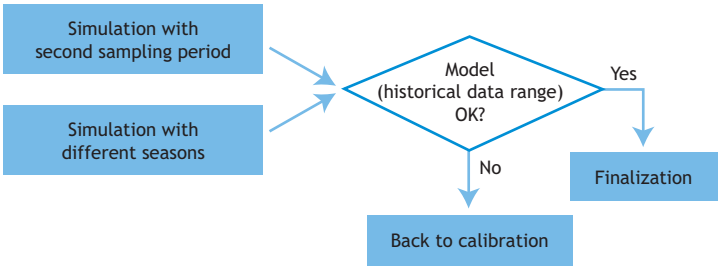

Figure 16.15 Validation procedure

This test (validation) is carried out to evaluate whether the model applicability can be extended to a reasonable range of dynamic operating conditions within the regularly experience, such as wet weather conditions and/or seasonal climate changes. Dynamic validation only is used for plants that have been selected to be optimized in considerable detail (e.g. designing dynamic process control). It often happens that the initial calibrated model does not satisfactorily match the tested dynamic operating conditions. In this case a second calibration would be necessary. Usually one additional iterative fine-tuning procedure is sufficient to make a model satisfactorily describing the selected conditions. It should be noted that models are not designed to describe extreme conditions that may occur at the treatment plant and such conditions (if any) will not be considered in this project.

16.2.11 Scenario investigation

Based on the conclusions reached from the mass balance data analysis (as presented earlier in this report) and the calibrated model results, the operational assessment of the plant can be carried out and improvements on the performance can be accurately proposed based on the calibrated model results. In itself, this is a good achievement for the modeling technology; it is possible to describe the (measured) operation of a wastewater treatment plant with a high level of accuracy. However, the true power of the application of models lies in the extrapolation of the designed to non-existing operational conditions. By using the model it is possible to predict future performance and performance under different operational conditions that cannot be measured. Moreover, this can be done within an acceptable level of accuracy, but in any case a higher rate than all the other design methods available. Scenario analysis studies are always specific to the plant and research objectives and no standards are available. A practical limitation however is the amount of data that is produced by the model which rapidly accumulates. Therefore a scenario study should not exceed a maximum of 5 to 8 scenarios. Above this number it will be very difficult to distinguish between the simulations and make well-informed choices about the design.

16.3 Conclusions

The STOWA protocol was developed based on the experience of a large and diverse group of wastewater and modeling practitioners. It proposes a uniform protocol for static and dynamic modeling of activated sludge systems. This STOWA protocol is a guideline for improving the quality and controllability of the simulation studies for activated sludge processes. Some problem areas have been identified and analyzed in the calibration procedure. For these bottlenecks practical solutions were formulated and incorporated in the protocol. In the protocol it is recommended to use the following sequence for the calibration phase:

- Fit the total phosphorus balance and fix SRT;
- Fit the TSS balance and fix the sludge production;
- Fit the particulate nitrogen balance and the nitrogen content of activated sludge;
- Calibrate the ammonium concentration in the effluent (nitrification);
- Calibrate the nitrate concentration in the effluent (denitrification).

This approach has been tested on various occasions and has become standard in good modeling practice.

Chapter 17

Planning

17.1 Planning the steps in the guidelines

This short chapter proposes a sample plan from the guidelines. In Chapter 16 (and 18), each step (phase) was estimated regarding the estimated time investment. Based on practical experience, the guideline phases were further divided into practical steps and an estimated time investment. Typically, dedicated office software is used (e.g. MS Project) to save time in making an efficient plan and, if necessary, adjusting it during the progress of the project. A detailed sample plan in MS Project is presented in the appendix to this book. Table 17.1 further specifies the steps in the guidelines. It also indicates deadlines for reports and models.

Table 17.1 Overview of planned tasks in the general guidelines and estimated time investment.

	Name of task in planning	Time invested in days
	Total Guidelines	**46.0**
1	**Project kick-off**	**2.0**
2	Kick-off meeting	0.5
3	Definition of the objectives	0.5
4	Planning of time and resources (report)	1.0
5	*Project kick-off report*	
6	**Inception phase**	**5.0**
7	Visiting plant location	1.0
8	Making PFD	1.0
9	Reporting initial information (report)	3.0
10	*Inception report*	
11	**Initial model**	**2.0**
12	Model construction	1.0
13	Initial simulations	0.5
14	Testing results	0.5
15	**Project evaluation**	**1.0**
16	Evaluation meeting	0.3
17	Defining starting points of the project	0.3
18	Updating project definition	0.3
19	Updating planning of time and resources (report)	0.3
20	*Updated project progress report*	
21	**Data acquisition phase**	**25.0**
22	Definition of mass balances	1.0
23	Making sampling program	1.0
24	Practical preparation of sampling	1.0
25	Sampling	3.0
26	Laboratory work	5.0
27	Idle time	7.0
28	Reporting measurement analysis	2.0
29	Checking measured data	4.0
30	Reporting final data set	1.0
31	*Report on collected and processed data*	
32	**Model calibration and simulation**	**4.0**
33	Model completed	
34	**Model validation and design calculations**	**4.0**
35	Traditional design calculations	2.0
36	Model validation	1.0
37	*Reporting*	1.0
38	Model validated	
39	**Final plant assessment**	**3.0**
40	Determining safety factors	0.5
41	Evaluating design	0.5
42	Formulating advice	0.5
43	Quality control on the design	0.5
44	final report on plant assessment	1.0
45	*Full plant assessment report*	
46	**Finalization guidelines**	

The time plan is made immediately at the start of the project. During the first project evaluation, following the initial simulations, the plan can be updated as at this stage of the project a complete project overview is available. Based on this overview, the initial objectives can be validated and the project planning for the given time and project budget can be further detailed. Because the model design project relies on actual measured data, idle time needs to be reserved for when waiting for the measurement results.

17.2 Risks in time and budget management

The main risk involved in model-based project is the acquisition of all the required (often not routinely measured or readily available) information on time as well as sufficiently accurate to be used for plant assessment and for the model study. Additional attention should be given to the project plan and specifically to the plan for the sampling program (see Chapter 15). Because measuring and processing plant data are time consuming activities as well as costly, it is worthwhile investing time in a good program preparation, thereby also reducing the risk of exceeding the budget and the available time for the project execution. One of the issues in executing sampling campaigns on site is effective communication with the laboratory staff and plant operational personnel involved regarding what, where and how samples should be collected and analyzed. To facilitate this communication, the protocol includes a sampling program plan and design together with information on measurement types, sample handling, sample location in reactors and flows, sampling frequency and the analytical method and sample preparation. For the budget management, a cost calculation is also included for analytical work.

17.3 Estimated time investment for technical tasks

The time chart presented in Figure 17.1 gives a general indication of the time allocation for modeling a complex WWTP (in this case the WWTP Haarlemwaarderpolder near Amsterdam, Brdjanovic *et al*., 2000). The chart is made on the assumption that the designer (modeller to be) has a good knowledge of underlying wastewater and sludge treatment processes, has good understanding of the ASM and ADM models, has no previous practical experience with modeling, and has the support of an experienced modeller. The total time investment assuming the above is approximately 4 to 6 months including the execution of an additional sampling program and reporting.

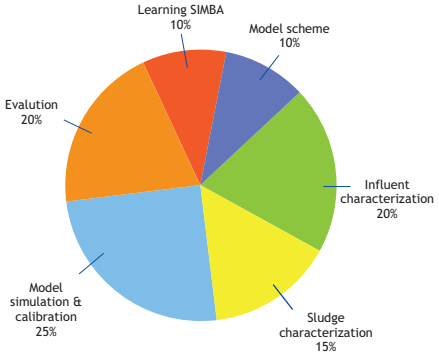

Figure 17.1 Estimated relative time investment for performing the technical tasks in a model-based project.

The study of Brdjanovic *et al*. (2000) indicated that the model simulation and calibration is the most difficult and time-consuming step in the process of modeling a wastewater treatment plant. Another relatively time-consuming activity is the evaluation of the simulation results. As for each wastewater treatment design project, irrespective of whether the modeling is used or not, the assessment of design information and drawing conclusions requires an expert's knowledge. It is further indicated that the influent characterization also takes approximately 20% of the total invested time; however, like the sludge characterization, this also includes sampling and analytical measurements work. In themselves, the calculation procedures (model simulations) are not complicated and can be performed in less than a day. Finally, it is important to realize that for each new technology/approach, time should be reserved for training and introduction. Also when the technology/approach is not regularly used, the knowledge level will deteriorate in time and some additional time for refreshing it and possibly additional reading or training should be planned.

Reference

Brdjanovic D., van Loosdrecht M.C.M., P. Versteeg, Hooijmans C.M., Alaerts G.J. and Heijnen J.J. (2000). Modeling COD, N and P Removal in a Full-scale WWTP Haarlem Waarderpolder. *Water Research,* Vol.34, No. 3, pp. 846-859.

Chapter 18

Conclusions

The application of mathematical modeling in the wastewater sector started in the early 1980s. Several models surfaced and each model used a different nomenclature and different approach. At that time, the former International Water Quality Association (IAWQ), which was originally formed as the International Association for Water Pollution Research (IAWPRC) in 1965 and which in 1999 became the International Water Association (IWA), played a pivotal role in the development of wastewater modeling by establishing the 'IAWPRC Task Group on Mathematical Modeling for Design and Operation of Biological Wastewater Treatment' in 1986. A year later, this group produced the Activated Sludge Model Nr 1 (ASM1) and this year (1987) is considered 'Year Zero' for wastewater modeling. ASM1 is in essence a consensus model – comprising the results of discussions at the time between different modeling groups, most prominently from South Africa, the U.S.A., Switzerland, Japan and Denmark. Many of the basic concepts of ASM1 were adapted from the activated sludge model defined in 1980 by the group from the University of Cape Town, S.A. In the 1990s we witnessed sudden bursts and a gradual increase in the use of mathematical models. This evolution was undoubtedly supported by the availability of more powerful computers. Nowadays models are regularly used at almost every plant in the US, Australia, Canada, Japan and many EU countries. In 1994 in the Netherlands, for example, Dutch practitioners started to feel that models could be useful. Soon after (1995), STOWA recommended the use of the SIMBA simulator as a national standard. In 1996 the STOWA modeling and characterization protocol was published. By 1997 approximately half of the WWTPs in the Netherlands were modelled. In 1999 a new protocol for model calibration was introduced. Nowadays more than 90% of the new WTTPs in the Netherlands are modelled. At the moment, looking from a practical perspective, models are considered to be sufficiently developed and reliable for most of the purposes for which they are used. Modellers have become a standard asset of any serious consulting company in the country and even specialist modeling companies have emerged in the sector. This was accompanied by global developments such as the establishment of the IWA Specialist Group on Good Modeling Practice and the biannual specialist IWA conference on mathematical modeling that started in 1995 in Denmark. The latest encouraging developments demonstrate that modeling is no longer the privilege only of the developed world, as the applications in developing countries (e.g. India) and countries in transition (Korea, Bosnia and Herzegovina) are increasing.

In Croatia new legislation is being introduced which stipulates that for agglomerations larger than 10,000 PE located in environmentally sensitive areas (in total 54 in the country are declared as such), it is necessary to apply a tertiary level of wastewater treatment. For full application of this directive, Croatia has requested an implementation period of up until 2023 in order to complete the projects on wastewater collection and treatment. It has been estimated that approximately 3.15 billion EUR will be needed to make this all happen. Given the extent of the investment, as well as the tremendous present and future efforts that the Republic of Croatia is undertaking regarding the control of man-made water pollution, it is clearly a necessity that the Croatian wastewater sector is reinforced and that capacity building in the sector takes place at an increased speed. This is not only a prerequisite for sustainable development and safeguarding the above investments, but also an opening window into the world of innovations in the

wastewater sector, being not only of a technological and technical nature, but also institutional, financial, social and economical. So far in Croatia, modeling has been applied extensively in the wastewater sector only to sewage collection and urban drainage systems. Therefore, this innovative modeling project is considered as a pioneering application of mathematical modeling on Croatian wastewater treatment practice. This project introduces a model-based approach to project preparation, design and operation concerning both new and existing plants. It includes the evaluation of all the 26 plants in the Black Sea river basin and demonstrates successful application of mathematical modeling on 5 selected large WWTPs in that basin. In addition, tailor-made guidance for the application of mathematical modeling of activated sludge plants in Croatia has been prepared as presented in this report, and also translated into the Croatian language to make it more accessible to the wider wastewater treatment public. Given the fact that the geographic area concerned in this project (part of the Black Sea river basin belonging to Croatia) has some very different characteristics to the Croatian Adriatic coast (temperature, climate, effluent recipient, tourism, islands, etc.), it is strongly recommended that the scope of this project is extended to the Croatian coastal areas, in order to cover the entire territory of the Republic of Croatia. The two new elements in the modeling approach regarding the coastal pollution control are description of the links between the (level of) wastewater treatment and seawater quality, and the inclusion of options concerning short and long sea outfalls. The objective of this project is to demonstrate the use of modeling in practice on a full scale and, based on the local (Croatian) experience gained through the project, to present a methodology (a general protocol) that can be used for WWTP modeling and the application of wastewater treatment assessment, upgrade and design.

The developed guidelines originate from the Dutch experience for model design in activated sludge modeling (Model Based Design or MBD). The aim of the guidelines (the model protocol) is to reduce the complexity of model design projects. Therefore, the model design is divided into smaller sub-tasks. Seven different phases in wastewater treatment plant modeling are distinguished according to the general scheme in Figure 18.1, and in more details in Figure 18.2.

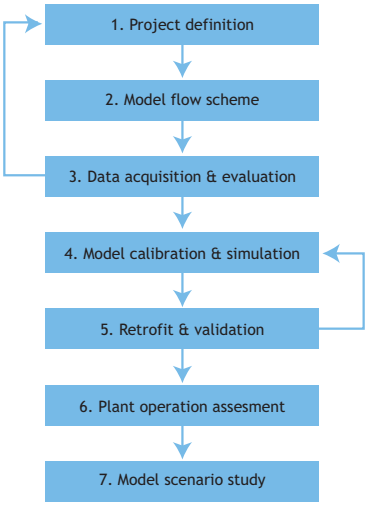

Figure 18.1 General structure of the guidelines presented in this report depicting the seven phases of modeling an activated sludge plant.

In the protocol each step is addressed in a standard, practical, feasible and realistic manner. The method was demonstrated in practice in the Croatian context in five model design cases (WWTP), namely: Varaždin, Čakovec, Vinkovci, Zagreb and Belišće. In the guideline report the applied methodology is presented in a generic and explanatory way. The aim for the guidelines is to be used as a knowledge base and in practice a proven manual (check list) for new design projects thereby providing a quality basis and standard for new model design studies in the future development of wastewater treatment in Croatia towards EU regulations and improved environmental safety standards.

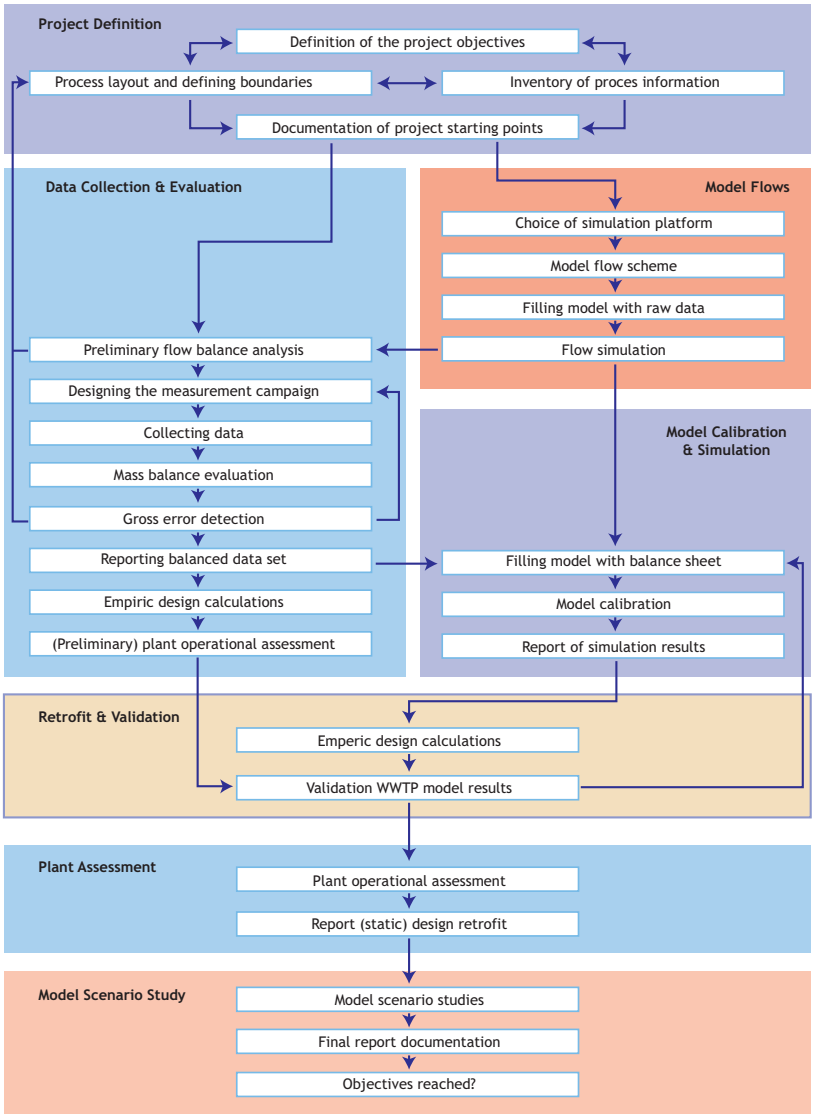

Figure 18.2 Generic scheme of the activated sludge modeling guidelines in this report.

The proven key factor for the successful introduction of model-based wastewater treatment design is the practical standardization of the methods, models and the simulation software. These guidelines are the foundation for a univocal approach and application in wastewater treatment and have been put in practice in a Croatian context and thereby further developed into a more tailor-made practical document. Emphasis was given in the protocol to the local conditions and, where needed, translations were made of key technical documentation published elsewhere. The guidelines provide a knowledge base and practical advice useful for project management, logistical organization, quality control and technological development. The guidelines provide managers with a rapid project overview and guidelines for quality control, consultants with a knowledge base for design projects and developments, and modellers, operators and laboratory personnel practical with 'hands on' advice on how to improve their operations towards modern standards in the context of wastewater treatment model design. Based on the information presented, the training program was started in 2011 on Croatia-wide capacity building regarding the wastewater modeling for which the foundations were laid down by this project.

The structure of the guidelines is a seven-step generic approach in preparation for a location-specific wastewater treatment design study. The first step is the inception phase where the main goal is getting a complete project overview. It contains the visits of the plant location collecting all kinds of information based on the checklists provided in the guidelines. The results of the inception phase are a definition of the project objectives and realistic planning of available time and resources. The estimated invested time for this phase is 4 to 5 days including plant visits and writing a short report on the conclusions.

Following the inception phase, the modeling work is carried out. In a desk study lasting an estimated 1 to 2 days, using the preliminary collected information, an initial model is constructed and the first simulations take place. Thus an early overview of the project is obtained and the modeling requirements can be overseen resulting in further elaboration of the project objectives as well as time and resource planning. For the modeller who already has hands-on experience with the (general) modeling software, this stage also provides the possibility to refresh his/her modeling skills through further training.

From the initial desk study, the focus is then redirected to acquiring the operational and design plant data. Therefore, a comprehensive sampling and measurement plan is designed which should be communicated to all the personnel involved during a new (repeated) plant visit(s). Again, the guidelines here serve as a checklist for all the steps to be taken in the preparation of data acquisition. At this stage of the project, there is a substantial investment in financial resources for sampling and analytical analysis as well as a personnel time investment. Therefore efficient planning will definitely save some costs. At the same time, additional data acquisition will ensure that all the required information necessary to bring the project to a successful completion (to get reliable results) is collected. To obtain the plant measurements, an estimated 2 to 3 weeks are needed to cover the complete procedure of sampling and analytical work. When this data is made available, the project can proceed, and therefore, this project stage is critical for the time planning. At this stage the collected information is checked for easily observable errors and (in)consistency. Because actual (real) plant measurements are used, errors will frequently occur. Therefore, the protocol provides a set of necessary steps to check and correct the collected plant information, thereby providing the highest reasonably possible accuracy for the model results. The data acquisition phase is finalized by drafting a report presenting all the raw and corrected plant information in a single tabular overview. This table serves as the starting point for the model simulations and is also a reliable reference point

regarding the operational performance of the plant as it is based on measured (observed) information. The total time investment in this phase depends on the amount of data collected, and is estimated for a small project as 3 weeks and a larger project as 4 weeks (including preparations and practical execution).

In the model calibration and simulation phase that follows, the guidelines focus on the more technical aspects of the activated sludge modeling protocol. They explain step-by-step how the model should be fitted to the measured data (reality). In scientific literature the model calibration is often indicated as a complicated and time-consuming step in the model application. These conclusions are however nowadays considered out-dated as the guidelines presented recommend a methodology for calibration that is not only practical and easy to apply, but also results in a highly accurate model description. By using these guidelines, a modeller can obtain a fully calibrated model in within 2 to 5 days, depending on the experience of the modeller. For pioneering studies it is advisable to reserve 2 to 3 additional weeks, so that the user (modeller to be) can get acquainted with the software, the background theory of ASM models, and model calibration. If possible, he/she should follow specialist training on this topic.

The model simulations are followed by a traditional design phase where all the results from the model and plant measurements are calculated and reported, leading to a traditional wastewater treatment plant design. Thereby, state-of-the-art wastewater treatment modeling is embedded in the traditional design practice whereby model-based design and traditional design are compatible. This next step in the guidelines provides the tools to present model results in such a way that others (without modeling skills) can understand and judge them. Thus, obviously the wastewater treatment modeling supports traditional design and operation of WWTP. Templates of the summary tables are presented in the guidelines to compare the model results to the measured data and thereby the model is validated for use in a scenario study analysis. Preparing the traditional WWTP design and validating the activated sludge model is a desk study that will take approximately 2 to 5 working days.

In the concluding phase a final operational plant assessment is made and disseminated. Interpreting design data requires an expert's judgment. Making conclusions on the plant performance and the results of the model application will take approximately 1 to 2 days, depending on the experience of the designer. The model design accuracy is also evaluated and design safety factors are introduced. The accuracy of the model predictions can be determined from differences between the evaluated measured plant data and the results obtained by the calibrated model. A valuable aspect of modeling is the statistical accuracy that is available regarding the plant design and which is the basis for determination of more realistic safety factors. From the financial perspective, this is very relevant - safety incorporated in the design procedures directly increases the construction and operation cost of wastewater treatment and therefore the determination of realistic and acceptable safety factors often delivers financial benefit.

From this point on the added value of the model can be demonstrated by using it for various types of extrapolations, e.g. simulating conditions that do not (yet) exist. These are often called 'what-if scenario' studies. The model can be used to check how the plant will perform under full load (design capacity); in other words, whether the plant will indeed be capable of reaching the designed effluent standards under full load. If the WWTP is already overloaded, the model can be used to alter the design and extend the plant (volumes and capacities) to test the effect of upgrade or retrofit on the effluent quality. Simulations may also reveal the possibility for operational improvements, savings (energy and chemicals), and improvements in efficiency in

terms of effluent quality, biogas production, sludge minimization, and aeration requirements. The power (possibility) of performing realistic (dynamic and static) scenario studies is the unique feature of the modeling approach; there is no other technique/methodology by which these sorts of predictions can be carried out within the given margin of accuracy. On the other hand, these results can only be achieved if all the recommended steps are systematically and accurately followed according to the modeling protocol presented in the guidelines.

Appendix 1

Checklist - influent (table 1 of 6)			
Sewer system			
1	Max. pumping capacity	m^3/h	Overflow capacity
2	Rain catchment area	m^2	Total paved surface
3	Rain storage capacity	mm/h	Rainfall intensity
4	Rain storage capacity	m3	Sewer storage capacity
5	HRT	h	Estimation of DWF hydraulic residence time
Raw sewage			
6	ADWF	m^3/d	Average dry weather flow
7	Q365	m^3/d	Average mixed weather flow (DWF and WWF)
8	PDWF	m^3/d	Peak dry weather flow
9	AWWF	m^3/d	Average wet weather flow
10	PWWF	m^3/d	Peak wet weather flow
11	MWWF	m^3/d	Maximum wet weather flow - maximum sewage capacity
Raw sewage origin			
12	PE	136 gTOD/cap.d	Total population equivalents at 136 g TOD per person
13	PE	54 $gBOD_5$/cap.d	Total population equivalents at 54 g BOD_5 per person
14	PE (industry)	% of PE	Population equivalents contributed by industry
15	PE (urban)	% of PE	Population equivalents contributed by urban population
16	PE (rural)	% of PE	Population equivalents contributed by rural population
17	PE (served)	% of PE	Population equivalents connected to sewer system
18	Water use	L/person.d	Average drinking water use per inhabitant
Raw sewage temperature			
19	Tmin	˚C	Minimum temperature sewage
20	T365	˚C	Yearly average temperature sewage
21	Tmax	˚C	Maximum temperature sewage
Raw sewage composition (after grit and sand removal)			
	Total		
22	TSS	mg/L	Total suspended solids (after grit/sand removal)
23	ISS	mg/L	Total inorganic suspended solids (after sand removal) - ash
24	BOD_5	mgBOD/L	Total carbonaceous BOD_5
25	TCOD	mgCOD/L	Total COD
26	TKN	mgN/L	Total Kjeldahl nitrogen
27	TP	mgP/L	Total phosphorus
	Soluble		
28	BOD_F	mgBOD/L	Filtered (fiberglass or paper 1.2 µm) BOD
29	COD_F	mgCOD/L	Filtered (fiberglass or paper 1.2 µm) COD
30	COD_{MF}	mgCOD/L	Flocculated and micro-filtered (0.45 µm) COD
31	VFA	mgCOD/L	Volatile fatty acids or acetate
32	NH_4	mgN/L	Ammonium
33	NO_3	mgN/L	Nitrate - often assumed zero
34	PO_4	mgP/L	Phosphate
35	DO	mgO_2/L	Dissolved oxygen - often assumed zero
	Other parameters		
36	pH	-	
37	Alkalinity	$mgCaCO_3/L$	Alkalinity in $CaCO_3$ equivalents
38	Calcium	mg/L	
39	Magnesium	mg/L	

Checklist - primary treatment (table 2 of 6)			
Raw wastewater treatment			
	Grid and sand removal		
40	Designed solids loading	$kg/m^2.h$	Maximum design TSS loading
41	Maximum hydraulic capacity	m^3/h	Overflow / bypass capacity
	Screens		
42	Type		
43	Maximum hydraulic capacity	m^3/h	Overflow / bypass capacity
	Influent pumping station		
44	Pump type		Screws, pumps
45	Installed pumping capacity	m^3/h	Maximum capacity
Primary treatment			
	Primary settling tanks		
46	Type	-	Round, rectangular
47	Maximum hydraulic loading	m^3/h	Max. flow for primary bypass conditions
48	Volume	m^3	Total volume including cone
49	Surface	m^2	Total surface for loading calculation
50	Depth	m	Side-wall depth
51	Diameter	m	For round tanks half the radius
52	Slope settling cone	-	Meter decline per meter diameter
53	Designed solids loading	$kg/m^2.h$	Maximum design TSS loading
54	Type of chemicals dosed	-	Polymers, aluminium, iron
55	Quantity of chemicals dosed	kg/d	Pure product
	Primary sludge		
56	Average pump capacity	m^3/d	
57	Installed pump capacity	m^3/d	
58	TSS	mg/L	Primary sludge TSS concentration
	Primary sludge thickening		
59	Volume	m^3	Total volume including cone
60	Surface	m^2	Total surface for loading calculation
61	Depth	m	Side-wall depth
62	Diameter	m	For round tanks half the radius
63	Slope settling cone	-	Meter decline per meter diameter
64	Maximum hydraulic loading	m^3/h	Max. flow for primary bypass conditions
65	Maximum solids loading	$kg/m^2.h$	Maximum design TSS loading
Primary settling solids removal efficiency			
66	TSS removal efficiency	% removal	Solids settling efficiency
67	ISS removal efficiency	% removal	Solids settling efficiency
68	Particulate BOD_5 removal efficiency	% removal	Solids settling efficiency
69	Particulate COD removal efficiency	% removal	Solids settling efficiency
70	Particulate TKN removal efficiency	% removal	Solids settling efficiency
71	Particulate P removal efficiency	% removal	Solids settling efficiency
Settled sewage			
	Totals		
72	TSS	mg/L	Total suspended solids
73	ISS	mg/L	Total inorganic suspended solids - ash
74	BOD_5	mgBOD/L	Total carbonaceous BOD_5
75	TCOD	mgCOD/L	Total COD
76	TKN	mgN/L	Total Kjeldahl nitrogen
77	TP	mgP/L	Total phosphorus
	Soluble		
78	BOD_F	mgBOD/L	Filtered (fiberglass or paper 1.2 µm) BOD
79	COD_F	mgCOD/L	Filtered (fiberglass or paper 1.2 µm) COD
80	COD_{MF}	mgCOD/L	Flocculated and micro-filtered (0.45 µm) COD
81	VFA	mgCOD/L	Volatile fatty acids or acetate
82	NH_4	mgN/L	Ammonium
83	NO_3	mgN/L	Nitrate - often assumed zero
84	PO_4	mgP/L	Phosphate
85	DO	mgO_2/L	Dissolved oxygen - often assumed zero
	Other parameters		
86	pH	-	
87	Alkalinity	$mgCaCO_3/L$	Alkalinity in $CaCO_3$ equivalents
88	Calcium	mg/L	
89	Magnesium	mg/L	

Checklist - water line (table 3 of 6)			
Activated sludge process			
	Selector tank		
90	Mixing type		None, mixer, horizontal flow
91	Volume	m^3	Reactor water volume
92	Depth	m	Reactor depth
93	Width	m	Reactor with
94	Surface	m^2	Reactor surface
	Anaerobic tank		
95	Mixing type		None, mixer, horizontal flow
96	Volume	m^3	Reactor water volume
97	TSS operation	g/L	Activated sludge concentration
98	Depth	m	Reactor depth
99	Width	m	Reactor with
100	Surface	m^2	Reactor surface
	Anoxic tank; denitrification		
101	Mixing type		None, mixer, horizontal flow
102	Volume	m^3	Reactor water volume
103	Depth	m	Reactor depth
104	Width	m	Reactor with
105	Surface	m^2	Reactor surface
	Facultative aerobic tank; on-off controlled aeration		
106	Mixing type	-	None, propeller, bubbles, surface aeration
107	Volume	m^3	Reactor water volume
108	TSS operation	g/L	Activated sludge concentration
109	Depth	m	Reactor depth
110	Width	m	Reactor with
111	Surface	m^2	Reactor surface
112	Aeration type	-	Bubble-diffuser, brush, point surface
113	Aeration set-point	mgO_2/L	Aeration control set-point
114	Installed capacity	kW/h	Installed power input surface aeration
115	Surface aeration efficiency	$kgO_2/kW/h$	Factory design oxygen production efficiency
116	Installed capacity	Nm^3/h	Installed blower air input
117	Bubble aeration efficiency	$gO_2/Nm^3.m$	Factory design oxygen transfer efficiency
118	Aeration depth	m	Bubble rise height
	Aerobic tank - aeration		
119	Mixing type	-	None, propeller, bubbles, surface aeration
120	Volume	m^3	Reactor water volume
121	TSS operation	g/L	Activated sludge concentration
122	Depth	m	Reactor depth
123	Width	m	Reactor with
124	Surface	m^2	Reactor surface
125	Aeration type	-	Bubble-diffuser, brush, point surface
126	Aeration set-point	mgO_2/L	Aeration control set-point
127	Installed capacity	kW/h	Installed power input surface aeration
128	Surface aeration efficiency	$kgO_2/kW/h$	Factory design oxygen production efficiency
129	Installed capacity	Nm^3/h	Installed blower air input
130	Bubble aeration efficiency	$gO_2/Nm^3.m$	Factory design oxygen transfer efficiency
131	Aeration depth	m	Bubble rise height
Internal recycle flows			
132	RAS recycle	m^3/d	Return activated sludge
133	Anaerobic recycle	m^3/d	Activated sludge recycled to anoxic tank
134	Anoxic recycle	m^3/d	Activated sludge recycled to anaerobic tank
135	Recycle to facultative tank	m^3/d	Activated sludge recycled to facultative tank
136	Internal Carrousel recycle	m^3/d	Activated sludge recycled in Carrousel
137	Horizontal flow velocity Carrousel	m/s	Horizontal water flow inside Carrousel
Activated sludge composition (measured outflow aerobic tank)			
138	TSS	g/L	Total suspended solids activated sludge
139	COD	mgCOD/L	Total COD activated sludge
140	Ash content	% of TSS	Inorganic sludge fraction
141	VSS	g/L	Organic sludge fraction
142	% organic bound TP	m^3	Total volume including cone
143	% organic bound TKN	m^2	Total surface for loading calculation
144	SVI	mL/g	Measured sludge volume index

Checklist - clarifiers (table 4 of 6) Secondary settling			
	Secondary settling tanks		
145	Type	-	Round, rectangular
146	Maximum hydraulic loading	m^3/h	Maximum flow for bypass conditions
147	Volume	m^3	Total volume including cone
148	Surface	m^2	Total surface for loading calculation
149	Depth	m	Side-wall depth
150	Diameter	m	For round tanks half the radius
151	Slope settling cone	-	Meter decline per meter diameter
152	Designed solids loading	$kg/m^2.h$	Maximum design TSS loading
153	Designed SVI	mL/g	Designed sludge volume index
154	Type of chemicals dosed	-	Polymers, aluminium, iron
155	Quantity of chemicals dosed	kg/d	Pure product
	Activated sludge setting characteristics		
156	TSS	g/L	Total suspended solids activated sludge to settler
157	Measured SVI	mL/g	Measured sludge volume index
	Waste activated sludge (WAS)		
158	Type of pump	-	Screws, pumps
159	Type of pump control	-	e.g. influent flow proportional
160	Installed pump capacity	m^3/d	Total pump capacity
161	Average pump capacity	m^3/d	average applied pump capacity
162	TSS WAS	mg/L	Primary sludge total suspended solids concentration
	Return activated sludge (RAS)		
163	Type of pump	-	Screws, pumps
164	Type of pump control	-	E.g. influent flow proportional
165	Installed pump capacity	m^3/d	Total pump capacity
166	Average pump capacity	m^3/d	average applied pump capacity
167	TSS WAS	mg/L	Primary sludge total suspended solids concentration

Checklist - effluent quality (table 5 of 6)			
Effluent			
	Effluent discharge requirements		
168	TSS	mg/L	EU standard: 30
169	TCOD	mgCOD/L	EU standard: 125
170	BOD_5	mgBOD/L	EU standard: 20
171	TKN	mgN/L	EU standard: 5
172	TN	mgN/L	EU standard: 10
173	TP	mgP/L	EU standard: 1
	Activated sludge process effluent measurements		
174	TSS	mg/L	Total suspended solids
175	TCOD	mgCOD/L	Total COD
176	COD_F	mgCOD/L	Filtered (fiberglass or paper 1.2 µm) COD
177	BOD_5	mgBOD/L	Total carbonaceous BOD_5
178	TKN	mgN/L	Total Kjeldahl nitrogen
179	NH_4	mgN/L	Ammonium
180	TN	mgN/L	Total nitrogen
181	NO_3	mgN/L	Nitrate
182	TP	mgP/L	Total phosphorus
183	PO_4	mgP/L	Phosphate
	Sewage bypass measurements		
184	TSS	mg/L	Total suspended solids
185	TCOD	mgCOD/L	Total COD
186	COD_F	mgCOD/L	Filtered (fiberglass or paper 1.2 µm) COD
187	BOD_5	mgBOD/L	Total carboneous BOD_5
188	TKN	mgN/L	Total Kjeldahl nitrogen
189	NH_4	mgN/L	Ammonium
190	TN	mgN/L	Total nitrogen
191	NO_3	mgN/L	Nitrate
192	TP	mgP/L	Total phosphorus
193	PO_4	mgP/L	Phosphate
	Total effluent discharge measurements		
194	TSS	mg/L	Total suspended solids
195	TCOD	mgCOD/L	Total COD
196	COD_F	mgCOD/L	Filtered (fiberglass or paper 1.2 µm) COD
197	BOD_5	mgBOD/L	Total carboneous BOD_5
198	TKN	mgN/L	Total Kjeldahl nitrogen
199	NH_4	mgN/L	Ammonium
200	TN	mgN/L	Total nitrogen
201	NO_3	mgN/L	Nitrate
202	TP	mgP/L	Total phosphorus
203	PO_4	mgP/L	Phosphate

Checklist - sludge treatment (table 6 of 6)			
Primary sludge thickening			
204	Thickening unit type	-	Gravity, filter press
205	Unit capacity	m^3/d	Maximal processing capacity of primary sludge
206	Primary sludge concentration	mg/L	Thickened primary sludge concentration
207	Thickened sludge concentration	% dry mass	Thickened primary sludge concentration in dry mass
208	Thickening factor	%	Factor by which the sludge is concentrated
Secondary WAS sludge thickening			
209	Thickening unit type	-	Gravity, filter press, centrifuge
210	Unit capacity	m^3/d	Maximal processing capacity of primary sludge
211	Thickened sludge concentration	mg/L	Thickened primary sludge concentration
212	Thickened sludge concentration	% dry mass	Thickened primary sludge concentration
213	Thickening factor	%	Factor by which the sludge is concentrated
Digestion			
214	Volume	m^3	Volume digester
215	Inflow digester	m^3/d	Average flow into digester
216	Inflow digester	% DM	Dry mass loading digester
217	Ash inflow digester	% of DM	Ash fraction of inflow digester
218	Digested sludge	% DM	Dry mass of digested sludge
219	Ash digested sludge	% of DM	Ash fraction of digested sludge
220	Gas head space	m^3	Volume of gas headspace in digester
221	SRT	d	Sludge retention time in the digester
222	Gas production	m^3/d	Methane/biogas production
Digested sludge dewatering			
223	Dewatering unit type	-	Filter press, centrifuge
224	Capacity	m^3/d	Maximal processing capacity of primary sludge
225	Thickened sludge concentration	% dry matter	Thickened primary sludge concentration
226	Thickened sludge concentration	mg/L	Thickened primary sludge concentration
227	Thickening factor	%	Factor by which the sludge is concentrated
Processed sludge production			
228	Dry mass	% DM	Dry mass sludge
229	Volume	m^3/d	Volume sludge
230	Weight	kg/d	Total weight sludge
231	% TKN in sludge	% of DM	Fraction TKN in dry mass
232	% TP in sludge	% of DM	Fraction TP in dry mass
233	% COD in sludge	% of DM	Fraction COD in dry mass
234	% Ash in DM	% of DM	Fraction ash/inorganic in dry mass

Appendix 2

Id	Taaknodi	Taaknaam	Duur	Begindatum	Einddatum
1		Guidelines	**46 dagen**	**vri 28-10-11**	**vri 30-12-11**
2		**Project kick off: 2 days**	**2 dagen**	**vri 28-10-11**	**maa 31-10-11**
3		Kick-off meeting	0,5 dagen	vri 28-10-11	vri 28-10-11
4		definition of the objectives	0,5 dagen	vri 28-10-11	vri 28-10-11
5		planning of time and resources (report)	1 dag	maa 31-10-11	maa 31-10-11
6		Project kick-off report	0 dagen	maa 31-10-11	maa 31-10-11
7		**Inception phase: 4-5 days**	**5 dagen**	**din 1-11-11**	**maa 7-11-11**
8		visiting plant location	1 dag	din 1-11-11	din 1-11-11
9		making PFD	1 dag	woe 2-11-11	woe 2-11-11
10		reporting initial information (report)	3 dagen	don 3-11-11	maa 7-11-11
11		Inception report	0 dagen	maa 7-11-11	maa 7-11-11
12		**Initial model: 1-2 days**	**2 dagen**	**din 8-11-11**	**woe 9-11-11**
13		Model construction	1 dag	din 8-11-11	din 8-11-11
14		Initial simulations	0,5 dagen	woe 9-11-11	woe 9-11-11
15		Testing results	0,5 dagen	woe 9-11-11	woe 9-11-11
16		**Project evaluation**	**1 dag**	**don 10-11-11**	**don 10-11-11**
17		Evaluation meeting	0,25 dagen	don 10-11-11	don 10-11-11
18		Defining starting points of the project	0,25 dagen	don 10-11-11	don 10-11-11
19		Updating project definition	0,25 dagen	don 10-11-11	don 10-11-11
20		Updating planning of time and resources (repor	0,25 dagen	don 10-11-11	don 10-11-11
21		Updated project progress report	0 dagen	don 10-11-11	don 10-11-11
22		**Data acquisition phase 18 days + 7 days**	**25 dagen**	**vri 11-11-11**	**don 15-12-11**
23		Definition of mass balances 1 day	1 dag	vri 11-11-11	vri 11-11-11
24		Making sampling program 1 days	1 dag	maa 14-11-11	maa 14-11-11
25		Practical preparation of sampling 1 day	1 dag	din 15-11-11	din 15-11-11
26		Sampling 3 days	3 dagen	woe 16-11-11	vri 18-11-11
27		Lab work 5 days	5 dagen	maa 21-11-11	vri 25-11-11
28		Idle time 7 days	7 dagen	maa 28-11-11	din 6-12-11
29		Reporing measurement analysis 2 days	2 dagen	woe 7-12-11	don 8-12-11
30		Checking measured data 3-4 days	4 dagen	vri 9-12-11	woe 14-12-11
31		Reporing final data-set 1 day	1 dag	don 15-12-11	don 15-12-11
32		Report on collected and processed data	0 dagen	don 15-12-11	don 15-12-11
33		Model calibration and simulation 4 days	4 dagen	vri 16-12-11	woe 21-12-11
34		Model compleated	0 dagen	woe 21-12-11	woe 21-12-11
35		**Model validation and design calculations 4 day**	**4 dagen**	**don 22-12-11**	**din 27-12-11**
36		Traditional design calculations 2 days	2 dagen	don 22-12-11	vri 23-12-11
37		Model validation 1 day	1 dag	maa 26-12-11	maa 26-12-11
38		Reporing 1 day	1 dag	din 27-12-11	din 27-12-11
39		Model validated	0 dagen	din 27-12-11	din 27-12-11
40		**Final plant assessment 3 days**	**3 dagen**	**woe 28-12-11**	**vri 30-12-11**
41		Determining safety factors	0,5 dagen	woe 28-12-11	woe 28-12-11
42		Evaluating design	0,5 dagen	woe 28-12-11	woe 28-12-11
43		Formulating advice	0,5 dagen	don 29-12-11	don 29-12-11
44		Quality control on the design	0,5 dagen	don 29-12-11	don 29-12-11
45		final report on plant assessment	1 dag	vri 30-12-11	vri 30-12-11
46		Full plant assessment report	0 dagen	vri 30-12-11	vri 30-12-11
47		Finalization guidelines	0 dagen	vri 30-12-11	vri 30-12-11

Appendix 3

WWTP Zagreb Costs indication for measurements			
	1.20	**€ / ILOW	
Total and solid fractions		***ILOW-points**	**Cost**
Total suspended solids	TSS	8	€ 9.60
Inorganic suspended solids (Ash content)	ASH	5	€ 6.00
Dry mattererial	DM	5	€ 6.00
Inorganic fraction of dry mass	IM	5	€ 6.00
BOD_5	BOD	12	€ 14.40
Total COD	TCOD	11	€ 13.20
Total Kjeldahl nitrogen	TKN	11	€ 13.20
Total iron (full recovery)	TFe	41	€ 49.20
Total phosphorus	TP	11	€ 13.20
Soluble fractions (including filtration)		***ILOW-points**	**Cost**
COD micro-filtered	CODMF	19	€ 22.80
COD glass-filtered	CODGF	19	€ 22.80
COD as VFA	VFA	20	€ 24.00
Soluble fractions		***ILOW-points**	**Cost**
Nitrate and nitrite	NOX	6	€ 7.20
Ammonium	NH4	6	€ 7.20
Ortho-phosphate	PO4	6	€ 7.20
Alkalinity	Alk	6	€ 7.20
Calcium	Ca	6	€ 7.20
Magnesium	Mg	6	€ 7.20
*Sterlab-WGS 01-07-2009 **DSBV ILOW price 2009			

WWTP Zagreb

Example sampling program for plant modeling

Description	Code	Unit	Preparation	NEN-ISO	Volume	Preparation at sampling location	Q1 Raw sewage	Q2 Influent to primary settlers	Q3 Settled sewage	Q4 Activated sludge tanks	Q5 Effluent	Q6 Dewatered sludge	Q7 Internal flows	Total additional analytical measurements per day	Total additional analytical measurements	Total indicative costs (EUR)
						PFD code	Q1	Q2	Q3	Q4	Q5	Q6	Q7			
						WWTP code	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.			
						SCADA code	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.			
						Sampling method	24h flow-proportional	Grab sample	Grab sample	Grab sample	Grab sample	24h flow-proportional	Grab sample			
Total suspended solids	TSS	mg/L	20 µm coarse paper filtrated, oven dried (105ºC)	6499, 872		Paper filtration (solid fraction)	1	1	1	1	1	1		6	0	57.60
Inorganic suspended solids (ash)	ASH	% of TSS	Incineration (550ºC) of filtered and dried TSS	NA		Paper filtration (soluble fraction)	1		1	1		1		4	0	24.00
Dry matter	DM	mg/L	Total sample evaporated in glass bottle; includes dissolved sa	NA		Total sample								0	0	0.00
Inorganic fraction of dry mass	IM	% of DM	Incineration (550ºC) of evaporated DM; includes dissolved sal	NA		Total sample								0	0	0.00
pH	pH		pH electrode measurement	NA		Total sample	1				1			2	0	12.00
Alkalinity	Alk	mgCaCO$_3$/L		NA		Total sample	1							1	0	7.20
Calcium	Ca	mg/L		NA		Total sample	1							1	0	7.20
Magnesium	Mg	mg/L		NA		Total sample	1							1	0	7.20
Dissolved oxygen	DO	mgDO/L	DO electrode measurement	NA		Total sample	1			1	1			3	0	21.60
Total iron (full recovery)	TFe	mgFe/L	Total sample (solids and liquid)	15587-1		Total sample								0	0	0.00
Total phosphorus	TP	mgP/L	Total sample (solids and liquid)	6646, 15681-2	100 ml	Total sample	1	1	1	1	1	1		6	0	79.20
Ortho-phosphate	PO$_4$	mgP/L	1,2 µm glass-fibre filtered (soluble fraction)	15681-2	100 ml	Paper filtration (soluble fraction)	1		1		1		1	4	0	28.80
Total COD	TCOD	mgCOD/L	Total sample (solids and liquid)	6633.00	100 ml	Total sample	1		1	1	1	1		5	0	66.00
COD glass-filtered	COD$_{GF}$	mgCOD/L	1,2 µm glass-fibre filtered (soluble fraction)	6633.00	200 ml	Paper filtration (soluble fraction)	1		1					2	0	45.60
COD micro-filtered	COD$_{MF}$	mgCOD/L	0.45 µm (membrane) filter (soluble fraction)	6633.00	200 ml	Paper filtration (soluble fraction)	1		1		1			3	0	68.40
COD as VFA	VFA	mgCOD/L	Acetate + propionate, No poly-acetate filters or vacuum filtrat	NA		Paper filtration (soluble fraction)								0	0	0.00
Acetate	HAc	mgCOD/L	Do not use poly-acetate filter or vacuum filtration	NA		Paper filtration (soluble fraction)	1		1					2	0	28.80
BOD$_5$	BOD	mgBOD/L	Total sample (solids and liquid)	1899-1	500 ml	Total sample	1		1		1			3	0	43.20
BOD$_5$ glass-filtered	BOD$_{GF}$	mgBOD/L	1,2 µm glass-fibre filtered (soluble fraction)	1899-1	500 ml	Paper filtration (soluble fraction)	1		1					2	0	28.80
Total Kjeldahl nitrogen	TKN	mgN/L	Total sample (solids and liquid)	6646.00	200 ml	Total sample	1	1	1	1	1	1		6	0	79.20
Ammonium	NH$_4$	mgN/L	1,2 µm glass-fibre filtered (soluble fraction)	NA	50 ml	Paper filtration (soluble fraction)	1		1		1		1	4	0	28.80
Nitrate and nitrite	NO$_X$	mgN/L	1,2 µm glass-fibre filtered (soluble fraction)	6604.00	50 ml	Paper filtration (soluble fraction)	1		1		1			3	0	21.60
						Additional samples per measurement point per day	18	3	13	6	11	5	2	58		655.20
						Total additional samples per measurement point	0	0	0	0	0	0	0		0	



Dr. Sebastiaan Meijer graduated as an Environmental Engineer from Wageningen University under the supervision of Prof. Lettinga. His scientific career continued at the Delft University of Technology (TU Delft), where in 2004 he was awarded a PhD degree for his thesis entitled 'Theoretical and practical aspects of modelling activated sludge processes' under the supervision of Prof. van Loosdrecht. In 2005, after working as a consultant for Grontmij for a couple of years, Dr. Meijer founded the company ASM Design B.V. with the goal of advising governments and providing training to wastewater professionals on modelling and design. He has carried out in excess of 50 modelling assignments in a wide spectrum, ranging from classical design examples up to the most technologically complex and specialised model design projects. Dr. Meijer is recognized as one of the leading specialists in the field of wastewater treatment model-based design and has a sound publication record. Since 1998, Dr. Meijer has been a lecturer in engineering and activated sludge modelling subjects at TU Delft and UNESCO-IHE, educating undergraduate and postgraduate students and sector professionals.

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