Modelling the sewer-treatment-urban river system in view of the EU Water Framework Directive

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Contents

1 Introduction **11**
1.1 General Introduction 12
1.2 Outline of the Thesis 13

2 Context and State of the Art **15**
2.1 The EU Water Framework Directive (WFD) 16
2.2 The Integrated Urban Wastewater System (IUWS) 17
2.2.1 IUWS management 18
2.2.2 IUWS management in Luxembourg 22
2.3 Modelling within the WFD 23
2.3.1 Modelling and simulation 23
2.3.2 The WFD modelling context 24
2.3.3 IUWS modelling 27
2.4 New Developments in IUWS Management and Modelling 30
2.5 Aim and Challenges of the Thesis 31
2.6 Adopted Approach 33

3 Modelling & Simulation Tools **37**
3.1 Presentation of the Software WEST® 38
3.2 Modelling of Urban Drainage 40
3.2.1 Subsystems and processes 40
3.2.2 Hydrological versus hydrodynamic modelling of water 41
3.3 The KOSIM-WEST Modelbase 43
3.3.1 Urban catchment 44
3.3.2 Sewer transport 52
3.3.3 Comparison of KOSIM-WEST with KOSIM 57
3.3.4 KOSIM-WEST: The 'Bonnevoie' catchment 62
3.4 Integrated Modelling with WEST® 69
3.4.1 Models for WWTP and river 69
3.4.2 Connector models: The continuity-based interfacing method (CBIM) 70

4 The Case Study **73**
4.1 Description of the Case Study 74
4.1.1 The urban catchment 74
4.1.2 The Bleesbruck WWTP 77
4.1.3 The receiving water system 80
4.2 Measurement Campaigns 80
4.2.1 Planning 82
4.2.2 Materials and methods 84
4.2.3 Preliminary results 85

4.3 Pressures and Impacts 86

5 The Integrated Model 89
5.1 Model Construction and Calibration 90
5.2 Catchment and Sewer Network Model 95
5.2.1 KOSIM-WEST model formulation 95
5.2.2 Data collection 95
5.2.3 CSO reduction in subcatchments: Preliminary analysis 97
5.2.4 The Bleesbruck sewer model in WEST® 99
5.2.5 Hydraulic model calibration using InfoWorks™CS 99
5.2.6 Calibration using WWTP influent data 103
5.2.7 Conclusions 107
5.3 WWTP Model 109
5.3.1 ASM model formulation 109
5.3.2 On-site implemented 'LIFE' model calibration 109
5.3.3 'CD4WC' model validation 116
5.3.4 8-months calibration 117
5.3.5 Conclusions 123
5.4 River Model 124
5.4.1 RWQM model formulation 124
5.4.2 River model construction 128
5.4.3 River hydraulics and transport calibration 130
5.4.4 River biochemical processes calibration 133
5.4.5 Conclusions 138
5.5 The Integrated Model 139
5.5.1 Integrated simulations 139
5.5.2 Conclusions 140

6 Scenario Analysis 141
6.1 Scenarios 142
6.1.1 References 142
6.1.2 Source Control 142
6.1.3 Construction Measures 145
6.1.4 Operational Changes 148
6.1.5 River Measures 150
6.2 Costs 153
6.3 Evaluation Criteria 155
6.3.1 The evaluation matrix 155
6.3.2 Criteria and thresholds for the Bleesbruck case study 157
6.4 Scenario Analysis 1 159
6.4.1 WWTP discharges and effects 159
6.4.2 CSO discharges and effects 165
6.5 Scenario Analysis 2: Low Base Pollution in the River Alzette 171
6.5.1 Immission-emission 171
6.5.2 Event-based analysis 174
6.6 Discussion 177
6.7 Conclusions 179

7 Conclusions 181
7.1 Achievements 182
7.2 Prospects for IUWS Modelling 184
7.3 Future Directions 186

Symbols **189**

Acronyms **191**

Appendix **191**

A Default parameters in KOSIM-WEST **193**

Summary **195**

Bibliography **206**

Summary **207**

Samenvatting **213**

Curriculum Vitae **219**

Chapter 1

Introduction

1.1 General Introduction

During the finalisation of this dissertation, world news within environmental issues was led by the publication of the first, second and third reports of the Intergovernmental Panel on Climate Change (IPCC) (www.ipcc.ch): Results, although still with high uncertainty on the extent of the problem, show that climate change provoked by human activity on the planet as well as its consequences are real. The phenomenon of climate change was investigated by 2500 experts all over the world, and collected data fed models to test multiple scenarios. It is generally agreed that actions need to be taken to reduce the emissions of the so-called greenhouse gases and this requires constant development of 'new' technologies and methods, but certainly also a mentality change of the end-user.

Directly linked to climate change are world water reserves. They have never been evenly spread among world regions and people, but global warming will displace this equilibrium even further to the extremes. Water however, unlike any fossil fuel, is elementary for life and therefore demands highest attention regarding its use, to guarantee, through the most diversified solutions, clean water for everybody. Moreover, increased pollution and water overuse with industrialisation and population growth puts significant pressures on water resources (CEC (2007)).

Hence the necessity to possess ways and tools to analyse, evaluate and optimise water supply and evacuation. It is stipulated by Wilderer & Odegaard (2006) that, indeed, we do not have a water crisis but a management problem. Especially in today's fast growing cities, good management plans are crucial to make sure that water resources are available and that rain and wastewater are brought out of the city without harming the population nor the ecological status of the receiving waters. Together with interdisciplinary collaboration and new technologies, a wide range of models, from river basin models to treatment process models to socio-economic models, can help to identify optimal, tailor-made solutions. Indeed, without interpreting outputs as accurate predictions, the objective of models, such as required in this context, should be to help understand the directions and the magnitude of different options (Jakeman & Letcher (2003)).

The EU Water Framework Directive (WFD) (CEC (2000)) also identifies modelling of systems as one of the tools for good implementation (a.o. Dorge & Windolf (2003), Rekolainen *et al.* (2003)). One overall objective of the WFD (presented in more detail in Chapter 2) is to obtain good chemical and ecological status of surface waters. Hence, if so far legislation focussed on emission limits into the receiving water by means of the Urban Wastewater Treatment Directive (CEC (1991)), the WFD displaces this approach to an immission-based approach. It is the status of the receiving water that will decide on whether or not pollution-abating measures are to be taken in the catchment.

The quality of the receiving waters is affected by two kinds of polluting sources: point and diffuse pollution. The former represents all point-like discharge locations of untreated wastewater, overflowing sewer systems or treatment works (municipal or industrial). The latter is closely linked to land use (agriculture, industry, urban activity, transport,...) and can be imagined as an accumulation of small point discharges from separate sources within the catchment. Especially for point sources from urban catchments, if data are available, emissions are fairly easily computed using models of differing complexity. With the immission approach, rivers will have to be included in the evaluation and decision processes, and therefore also in the models. However, so far, they are mostly omitted, certainly due to their complexity in terms of hydraulics and processes, but probably above all due to the lack of experience of engineers that are used to deal with urban drainage and wastewater treatment alone.

Hence, there is a need for more case study investigations and models, and the WFD should be seen as an opportunity for development in the field of urban water management and of tools for assessment of river water quality. The work presented in this thesis will concentrate on the integrated urban wastewater system (IUWS), i.e. sewer-treatment-urban river system, and uses a model to analyse various system configuration scenarios in view of WFD implementation.

1.2 Outline of the Thesis

The work described in this document was performed within the WFD context. Via model simulations, various management scenarios of the IUWS, i.e. sewer network, WWTP and urban river, are tested and compared. Such integrated modelling and simulation should be regarded as a tool to assess the impact of an urban catchment on receiving rivers and as a tool inside and part of the implementation process of the WFD. Application and illustration of the approach is done on a case study in Luxembourg.

The different chapters in the thesis are separately treating different steps and aspects of the work performed. Each of them should be readable on its own, although references to related information in other chapters are given.

Chapter 2 starts with a short description of the EU Water Framework Directive regarding its contents and the required tasks to be accomplished by the Member States. The next section gives a historical overview on urban wastewater management, reviews today's status and gives an overview on the situation of wastewater management in Luxembourg. The third section focuses on modelling, both in a WFD context and more specifically for the integrated urban wastewater system 'sewer-WWTP-river'. Then some of the new directions and needs for water resource management and modelling are given, before the aim and challenges of the thesis are presented. The Chapter finishes with an outline of the here-adopted approach.

Chapter 3 first describes the WEST® software platform used in this study, together with a new software kernel for better calculation performance, called Tornado. Thereafter, hydrological modelling as opposed to hydrodynamic modelling of sewer transport is briefly presented. The hydrological model KOSIM-WEST for catchment runoff and sewer transport, as implemented into WEST®, is presented. Simulation results from a hypothetical case study are compared to the original KOSIM model results to verify that the underlying models are the same. The model is also applied on a real urban catchment with storage tank. In the last part, model principles for WWTP and river systems as well as connector model principles used to link the submodels are explained.

Chapter 4 first characterises the integrated case study, situated in Luxembourg, by individually describing its 3 subsystems which are the urban catchment, the wastewater treatment plant 'Bleesbruck' and the receiving river system. A further section is devoted to the measurement campaigns conducted within the project. The last section summarises the deficits and pressures of the case study components.

Chapter 5 describes the construction and calibration of the integrated model of the sewer-WWTP-river system 'Bleesbruck'. First, a more general introduction to the here adopted approach for model construction and calibration is given. In the following sections, each submodel is described individually and calibration results are discussed. In the last section, integrated simulations are briefly introduced.

Chapter 6 first presents the 15 scenarios that, in the context of a combined immission-emission approach, have been tested via simulations of the integrated urban river system model described before. Costs of scenarios are discussed and an evaluation method, dealing with the large amount of simulated data, is presented. The first scenario analysis, done for immission and emissions at different locations, assesses the impact of the urban catchment Bleesbruck on the receiving rivers and identifies more and less suitable scenarios. In a second scenario analysis, a semi-hypothetical case study is analysed where the quite high original background concentrations of the Bleesbruck receiving waters is set to comply with WFD requirements.

Chapter 7 gives some general conclusions by summarising the achievements of this thesis, identifying the prospects for improvement within the here discussed case study and integrated urban wastewater system modelling in general. The Chapter ends on some general thoughts for future directions within urban wastewater management.

Chapter 2

Context and State of the Art

This Chapter starts with a short description of the EU Water Framework Directive regarding its contents and the required tasks to be accomplished by the Member States. The next section gives a historical overview on urban wastewater management, reviews today's status and gives an overview on the situation of wastewater management in Luxembourg. The third section focuses on modelling, both in a WFD context and more specifically for the integrated urban wastewater system 'sewer-WWTP-river'. Then some of the new directions and needs for water resource management and modelling are given, before the aim and challenges of the thesis are presented. The Chapter finishes with an outline of the here-adopted approach.

2.1 The EU Water Framework Directive (WFD)

The Water Framework Directive of the European Union (CEC (2000)) is the most important piece of legislation in the context of water in Europe and somewhat summarises the previous European directives on water by suggesting consideration of the water cycle as a whole, including groundwater and surface waters. It asks for '*good*' quantitative and qualitative status of water reserves and introduces the concept of *integrated river basin management*. This will require new planning and management practices and push forward research in the field (Griffiths (2002)). For good implementation on a Community level it also requires 'a transparent, effective and coherent *legislation*', evoques the *polluter-pays principle* and the involvement of the *general public*. In an urban drainage context, additionally to the Urban Wastewater Treatment Directive (CEC (1991)) setting the minimum level of treatment, and the EU Integrated Pollution Prevention and Control Directive (CEC (1996)), which is a regulatory instrument controlling emissions from major industrial sectors to all environmental media, the WFD shifts the focus from a purely source emission approach to a *combined approach* with 'control of pollution at source through the setting of emission limit values and of environmental quality standards' (Article 40, WFD).

In order to address the challenges in a co-operative and coordinated way, the Member States, Norway and the Commission agreed on a Common Implementation Strategy (CIS) for the WFD after the entry into force of the Directive. The aim of the CIS (CEC (2001), CEC (2003)) is to allow, as far as possible, the coherent implementation of the WFD in Member States and fixes a time schedule for realisation of certain tasks (see Table 2.1).

Table 2.1: Member States tasks and deadlines to be met for implementation of the EU WFD.

Year	**Issue**	**Reference**
2000	Directive entered into force	Art. 25
2003	Transposition into national legislation	Art. 23
	Identification of river basin districts and authorities	Art. 3
2004	Characterisation of river basin: pressures, impacts and economic analysis	Art. 5
2006	Establishment of monitoring network	Art. 8
	Start public consultation (at the latest)	Art. 14
2008	Present draft river basin management plan	Art. 13
2009	Finalise river basin management plan including programme of measures	Art. 13 & 11
2010	Introduce pricing policies	Art. 9
2012	Make operational programmes of measures	Art. 11
2015	Meet environmental objectives	Art. 4
2021	First management cycle ends	Art. 4 & 13
2027	Second management cycle ends, final deadline for meeting objectives	Art. 4 & 13

Within the CIS context, several guidance documents have been produced from expert meetings and pilot studies to help stakeholders to implement the individual steps. Of primary interest in the context of this thesis is the guideline on analysis of pressures and impacts (IMPRESS (2003)), which should help Member States to assess the human activity pressures on a water body and to estimate the likelihood of the water body to achieve good ecological status by 2015.

A first analysis should have been completed by the end of 2004, with subsequent refinement (through further monitoring campaigns) to produce a programme of measures, which can be expected to improve the current status. The guideline on reference conditions (REFCOND (2003)) asks that for every water body type conditions are established, where any human pressure has no or only minor effect on the ecological status. To do this, tools include pressure analysis to screen for sites or values representing such conditions, monitoring data, modelling or historical data or paleoreconstruction.

Establishment of monitoring networks and management plans for river basins will both require a wide variety of modelling (see section 2.3) and monitoring tools (Allan *et al.* (2006)).

2.2 The Integrated Urban Wastewater System (IUWS)

One important puzzle piece within the integrated river basin is the *integrated urban wastewater system (IUWS)*. As in the here presented context, it includes the following: rainwater runoff from pervious or impervious surfaces and domestic or industrial wastewaters, altogether transported to a wastewater treatment plant (WWTP), treatment of the combined sewage at the WWTP, discharges of these waters after treatment, and/or via combined sewer overflows (CSOs) during rain events, and the receiving water itself. Figure 2.1 illustrates the IUWS.

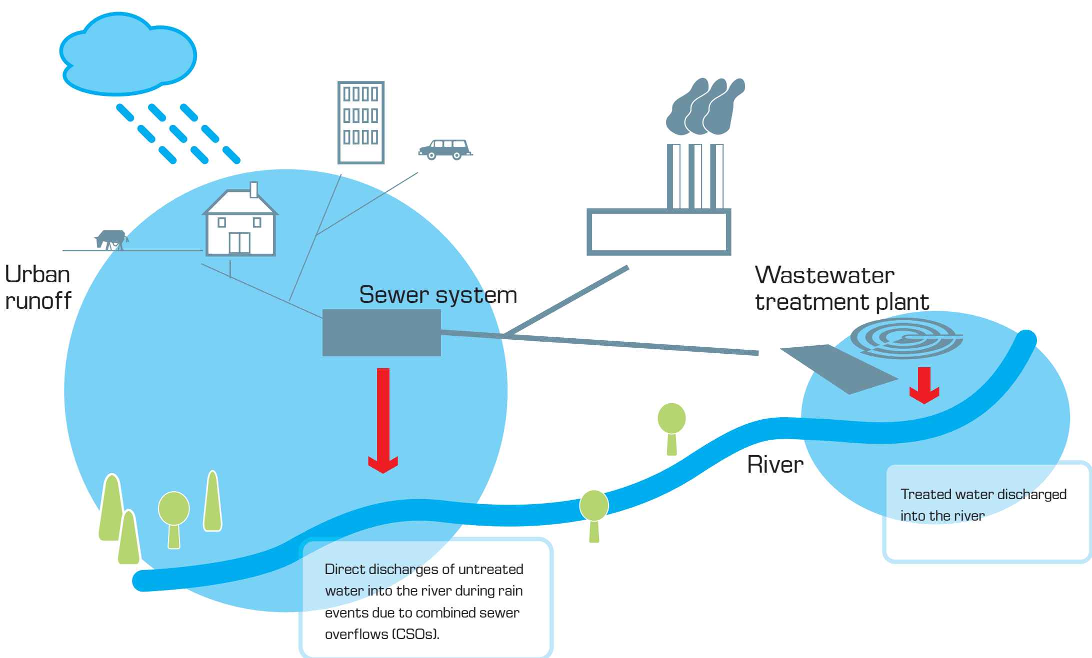


Figure 2.1: The integrated urban wastewater system (IUWS).

2.2.1 IUWS management

2.2.1.1 Historical development

Once communities started to agglomerate into cities, they automatically interacted with the natural water cycle; on the one hand they had to retrieve water to conduct it to citizens, and on the other hand stormwater had to be transported out of the city to avoid flooding, as through impermeabilisation of the ground, infiltration of stormwater was hindered. The effects of such urbanisation are to produce higher and sudden peaks in river flow and to introduce pollutant requiring the artificial treatment of wastewaters (Butler & Davies (2000)).

Hence, how to handle water in the city becomes an issue and many of the ideas promoted today have already been in practice within one or the other civilisation (Burian & Edwards (2002)). The Indus civilisation for example, flourishing during the 3rd millennium BC, used sumps at housing level for coarse material sedimentation before discharging wastewaters into open channels in the streets. The Minoan civilization seemed to use wastewater for field irrigation suggesting that they were aware of its fertilising capacities. The Persians, considering stormwater as sacred, collected it in cisterns for potable use and there existed laws for not mixing wastewater with stormwater. Such *decentralised* systems as they were, delegated a certain *responsibility* to the individual citizen as opposed to *centralised* systems emerging later with growth of cities.

All these infrastructures in ancient times were not planned but rather were they *optimised* using *trial-and-error*. *Engineering* as we know of today probably started with the Romans as they were the first to carefully *plan* road and drainage systems together and the cloaca maxima is probably one of the best known main collectors.

During medieval times, systems were being neglected as many people moved out of city centres and considered water infrastructures an unneeded service (Abeysuriya *et al.* (2006)). Hygienic practice went down, infrastructures were no more maintained and people just threw their solid waste as well as wastewater into gutters in the streets. What might have been of no concern at farms turned out to provoke epidemics in the cities once population densities grew again. As illustration on the city of London in the UK, in 1665, every parish along the river Fleet, one of the tributaries of the river Thames, was hit by the plague, as the Fleet was used as an open sewer (Petts *et al.* (2002)). It took a long time for politicians to accept the link between public health and wastewater until 1858 - the year of the Great Stink - where in hot summer, decisions were taken to invest into a new sewer system that would bring wastewater a long way out of the city using gravity sewers. Obviously the problem was somehow displaced towards degrading ecology downstream in the river.

At the end of the 19th century, we see the development of scientific basics of biological processes, irrigation fields and intermittent soil filtration or trickling filters. Most of the time though, untreated wastewater kept being discharged into flowing waters and the idea of self-purification of a river was upheld (Wiesmann *et al.* (2007)). In Europe, after the 1950s however, many urban areas were connected to a wastewater treatment plant and engineering solutions became more and more sophisticated in order to further reduce pollution. Attention shifted from purely protecting people from storm- and wastewater to finding solutions to reduce the impact that such waters have on receiving waters (Butler & Davies (2000)).

In general, through industrialisation, life near big cities began to flourish in the 19th century, and more rigorous planning of infrastructures was necessary. However, *maintenance* remained a problem due to fuzziness of responsibilities and various management schemes have emerged in different countries, from public administration both regulating and managing the water sector to liberalisation of the water market or even privatisation of infrastructures (WorldBank (2006)). Treatment facilities tended to be end-of-pipe and centralised solutions, especially as maintenance management was considered easier, however, investment costs are large mainly due to long connecting pipe lines.

In the 1980s, with the arrival of more and more performant computers, *modelling and simulation* emerged. This was a new tool to plan and evaluate complex systems to find solutions in urban drainage that went beyond the classical trial and error methods (see section 2.3). Technology, like actuators or online sensors for measurements allowed for remote *control* of infrastructures to further optimise their functioning.

2.2.1.2 Today

On a European level, due to the Urban Wastewater Treatment Directive (CEC (1991)), the greatest formerly bad polluted areas, where point source pollution predominated, have been improved in terms of river water quality (EEA (2005)). Remains, mainly in rural areas, diffuse pollution as the major problem to tackle. However, with the adoption of the Water Framework Directive, described above, focus on pollutant emission has been shifted to the receiving water itself. The latter becomes the indicator for the appropriateness of pollution management within the urban catchment, so that point source pollution cannot simply rely on emission standards. As discussed above, it requires an integrated consideration of influencing elements and the first INTERURBA conference (Lijklema *et al.* (1993)) set the stage for integrated planning and management of the integrated urban drainage system.

The consideration of the IUWS as a unity and the *emission-immission* approach of the WFD increases the degrees of freedom for wastewater management, as there are no precise directives on how to achieve a 'good' receiving water quality (a.o. Krebs (2003)). Knowing interrelated effects of one subsystem onto the other, favourable measures can be taken within the catchment, the sewer network, the WWTP or the receiving water. A good overview of interactions between the subsystems and the measures that can be taken within the IUWS to improve performance or water quality was prepared within the *CD4WC* project and a wide list of degrees of freedom for measures within the IUWS are explained in the deliverable CD4WC (2004). The project goes within the *CityNet* cluster (see Figure 2.2), a research project supported by the European Commission under the FP5. It is contributing to the implementation of the Key Action "Sus-

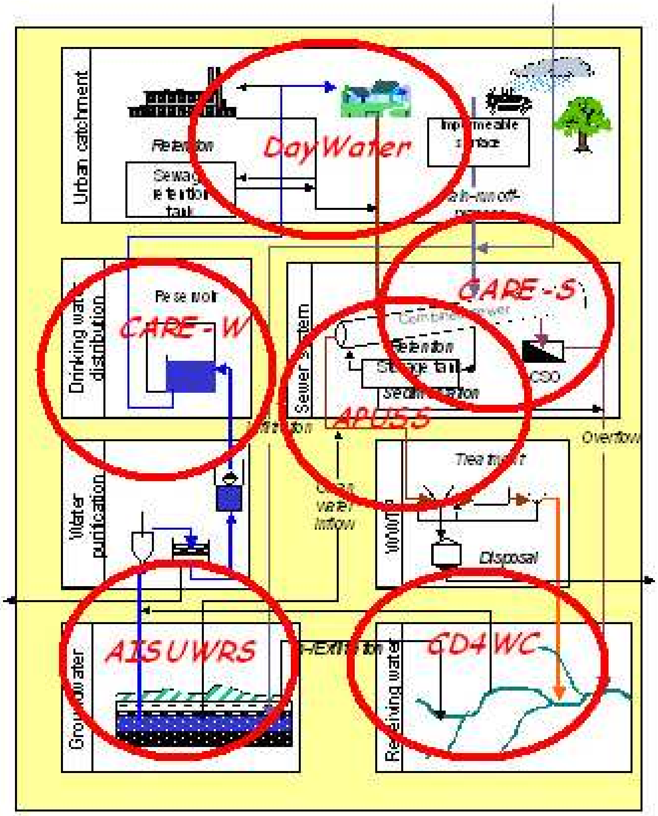


Figure 2.2: The CityNet project cluster under FP5.

tainable Management and Quality of Water" within the Energy, Environment and Sustainable Development Contract No: EVK1-2002-00570 (www.tu-dresden.de/CD4WC). Next to CD4WC for cost-effective optimisation of the urban wastewater system five other projects have been financed (http://citynet.unife.it): AISUWRS for development of an integrated contaminant flow and transport model for urban water systems; APUSS for development of methods and techniques to assess and quantify in- and exfiltration in sewer systems; CARE-W for the development of methods and a software to support an effective management of water supply networks; CARE-S for the development of a decision support system for cost-effective maintenance, repair and rehabilitation; DayWater for the development of a decision support system in urban stormwater management.

Certainly a major topic of discussion are the pro's and con's of centralisation / decentralisation of urban wastewater systems (Wilderer & Schreff (2000)). Historically, centralised systems proved very efficient in highly populated cities as they quickly improved public health and take away any responsibility from the user. However, besides the high investment costs for sewer pipes, centralised systems have large concentrated impact in case of failure. Other concerns are related to conventional, often centralised, wastewater treatment (Abeysuriya *et al.* (2006)), as they are not designed for pathogen destruction and mix nutrient and water cycles. The latter practice usually pollutes large amounts of water, making the treatment costly and complex, instead of nutrient recovery from urine for plant growth, for example.

In contrast to the end-of-pipe solutions, comparisons show that, efforts to reduce pollution at source, before it enters the sewage system, are often cheaper than investing into the construction of new treatment plants (EEA (2005)). A source control measure for more sustainable water management is dry weather flow reduction by assuming reduced water consumption through mentality change of inhabitants to save energy and water, high water prices, social peer pressure to display a certain behaviour and others. Several studies within a population have been carried out to find out about attitudes and driving factors of people towards water saving (e.g. Gilg & Barr (2006), Schosseler *et al.* (2007)). Alternative sanitation devices include water saving appliances, greywater reuse, dry toilets and others, like separation toilets, allowing for ammonium peak shaving and utilisation of urine as a fertilizer thereby not sending it to the WWTP at all (Otterpohl (2002)). Besides the fact of reducing the amount of clean drinking water to be polluted, it reduces the overall wastewater that needs to be treated and might diminish some of the overflow pollution loads discharged during rainwater events. Indeed, ecological sanitation (EcoSan) can be interesting for implementation especially in third world countries, where mainly decentralised systems allow closing local water and nutrient cycles to contribute to sustainable development (Langergraber & Muellegger (2005)). Instead of wastewater control, reduction of incoming rainwater is another option to reduce water inside sewer and treatment plant. This can be done through keeping impervious surfaces to a minimum for improved soil infiltration or rainwater reuse at housing level. Infiltration reduction in the sewer is another option to reduce the hydraulic load to the WWTP and to improve its performance.

Besides source control, there exists a large number of measures that can be considered to improver the performance of the IUWS, ranging from **construction** of additional infrastructures and/or **control** of actuators (pumps, valves, ...). Increased storage volume to reduce unwanted combined sewer overflows (CSOs), construction and control of storage tanks are widely applied. Instead of building new infrastructures, real-time-control (RTC) of sewer and/or WWTP have often proved to be an effective measure (a.o. Risholt *et al.* (2002)) to optimise usage of facility capacities but are often difficult to implement in practice due to the required expertise and maintenance. Older sewer systems are mostly combined systems, i.e. where wastewater is mixed to stormwater. They are equipped with combined sewer overflow (CSO) structures, which will discharge any water exceeding the sewer's capacity. It would indeed be economically unfeasible to have enough capacity on the full length of transport during rain events (Butler & Davies (2000)). Very often, newer drainage systems are built as separate systems. A clear advantage is that CSOs are avoided and that the WWTP only receives wastewaters and no stormwaters. Disadvantages are the existence of wrong connections and costs and special treatments for stormwater pollution are required.

Measures can also be taken on a receiving water level for mitigation of the morphologic impacts on a river from CSOs, like the increase of CSO storage volumes, reduction of slope, increase of grain size and widening of the river bed (Engelhard *et al.* (2005)). To react upon low DO concentrations, a valuable option can be aeration techniques (Vandenberghe & Vanrolleghem (2005)), i.e. the artificial input of oxygen through mechanical aerators, fountains or cascades enhancing water mixing. Increased shading from the riparian area (e.g. Mosisch *et al.* (2001), Ghermandi (2004)) is another option, which is expected to reduce solar radiation and therefore algae blooms in summer.

More and more sophisticated options become attractive for implementation by decision-makers, first of all due to the increasing number of available technologies, new environmental thinking and subsequent openness to **innovative** techniques and ideas and often due to cost-effectiveness on the long-term. However a main barrier for cost-effective planning are co-operation between stakeholders and therefore difficulties for integrated considerations. Indeed, good planning in advance often avoids expenses later. Also, regional characteristics of climate and socio-economic contexts determine the indicators and the subsequent appropriateness of a measure so that solutions are tailor-made for each case study, and this frame is given by the WFD.

Some specific possibilities to improve the IUWS, tested for the Luxembourg case study discussed in this project, are more explicitly described in Chapter 6 section 6.1.

2.2.2 IUWS management in Luxembourg

In Luxembourg, both drinking and wastewater are under *public management*. The law of 27 June 1906, concerning public health protection, delegates responsibilities for water provision to and evacuation of wastewaters out of agglomerations to municipalities. But as water was not an available resource in every municipality, drinking water syndicates were created. As such syndicates appeared to be economically and technically more efficient, most of the drinking and wastewater today is handled by syndicates disserving several municipalities each.

The biochemical condition of *rivers* in Luxembourg goes from good (e.g. Attert) to very bad (e.g. Alzette) (MI (2007)). Results of measurement campaigns regularly undertaken to evaluate the efficiency of all *WWTP*'s in Luxembourg that have capacities of more than 2000 inhabitants show that due to hydraulic restrictions, some of them cannot even treat all the discharged water during dry weather or lack second or tertiary treatment, bringing along the systematic discharge of insufficiently treated water into the receiving water. Indeed, half of the large treatment plants do not fulfil the requirements asked for by the Urban Wastewater Treatment Directive (CEC (1991)), and are currently being upgraded. Also *sewer networks*, often because of their high age, tend to be damaged allowing for infiltration of clean water, increasing the volume of the wastewater by diluting it. Although the investment into the water sector has considerably increased during the last ten years, a lot more funding is necessary for the next ten years.

In the frame of the *WFD* implementation, river basin characterisation was performed for the Moselle-Sarre basin (IKSMS (2004)), receiving about 98% of Luxembourg's surface waters. The water agency also has started the set up of a model for river basin management in a software called Pégase (Université de Liège, Belgium). The update of legislation and transposition of the

WFD contents into national law has recently started with a law proposal (MIAT (2007)). One of the discussed topics is harmonisation of water prices. Till today these prices are municipality dependent and, especially for wastewater, the consumer does not pay for the 'real' cost. A few years ago, a benchmarking exercise conducted regarding the water sector in Luxembourg (PwC (2003)) concluded that financing and tarifing schemes need to become more transparent and uniform, with better management and more rigorous quality assurance. It is foreseen that public subsidies are reduced and that polluter-pays principles are respected. For public involvement, as suggested by the WFD, the Luxembourg Water Agency has set up an internet site (www.waasser.lu) where citizens can retrieve information on the implementation of the WFD or view maps for the whole country relating to water issues like spring locations, rivers, flooding zones, habitats and others (http://gis.eau.etat.lu).

The *research* programme 'EAU: Gestion durable des ressources hydriques' (Sustainable management of aquatic Resources), funded by the FNR (National Research Fund), started in 2000 and ending in 2007, has considerably pushed forward water related research issues in a national and WFD implementation context. Conducted projects ranged from monitoring of surface waters to model applications for WWTP processes or sewer network control. Also, within this programme, an 'EcoSan' project for innovative sanitary concepts was conducted in order to introduce such new sanitation concepts within the building sector and to raise public awareness on saving water. Such public involvement is required by the WFD and results from public survey showed a lack of knowledge on the matter in general and predicted that the consumer will allow for higher fees if he is aware of risks and problems related to water (Schosseler *et al.* (2007)). Hence, information campaigns for the public are important components for efficient WFD implementation.

2.3 Modelling within the WFD

2.3.1 Modelling and simulation

Models have a wide spectrum of fields of application and a wide range of questions they can address. They can be used to gain better understanding of a certain phenomenon, to guide further investigations of the analysed system, to predict the spatial and temporal evolution of a system or simply as an educational tool for improved visualisation of underlying processes. To name just a few, application examples go from population models in biological sciences, to reaction kinetics in chemical sciences, to oscillators in physical sciences or even attempts to provide stock market predictions (e.g. Murray (2002), Nicolis (1995)). Often, and especially in environmental sciences, which is a highly interdisciplinary and collaborative discipline, models help to gather knowledge and to produce results that can directly be used in the context they have been elaborated for.

Indeed, the complexity of an at the same time physical, chemical and biological system as is the IUWS, due to the many parameters playing, the infinite amount of data to collect and our limitations in the experimental precision, make it difficult for us to analyse it properly and hence predict its evolution in time. Moreover, the time scales for processes to take place can be such that it becomes very time-consuming to collect enough data to assess the system and take the right actions to improve its operation. It can therefore be useful to represent relevant dynamical processes in a mathematical model allowing for computer simulations, in other words virtual experiments of these processes. Such mathematical modelling can serve as a tool to predict situations, give answers to precise questions and therefore help to improve the ecological and financial efficiency of an IUWS in this case.

Various kinds of models exist, more or less complex in structure. They can be conceptual models describing underlying concepts of processes, or exact mathematical equations of physical or chemical systems; they can be empirical models, like black-box models built on existing data and not relying on knowledge of the systems functioning itself. Stochastic models include the description of intrinsic randomness of processes within the system.

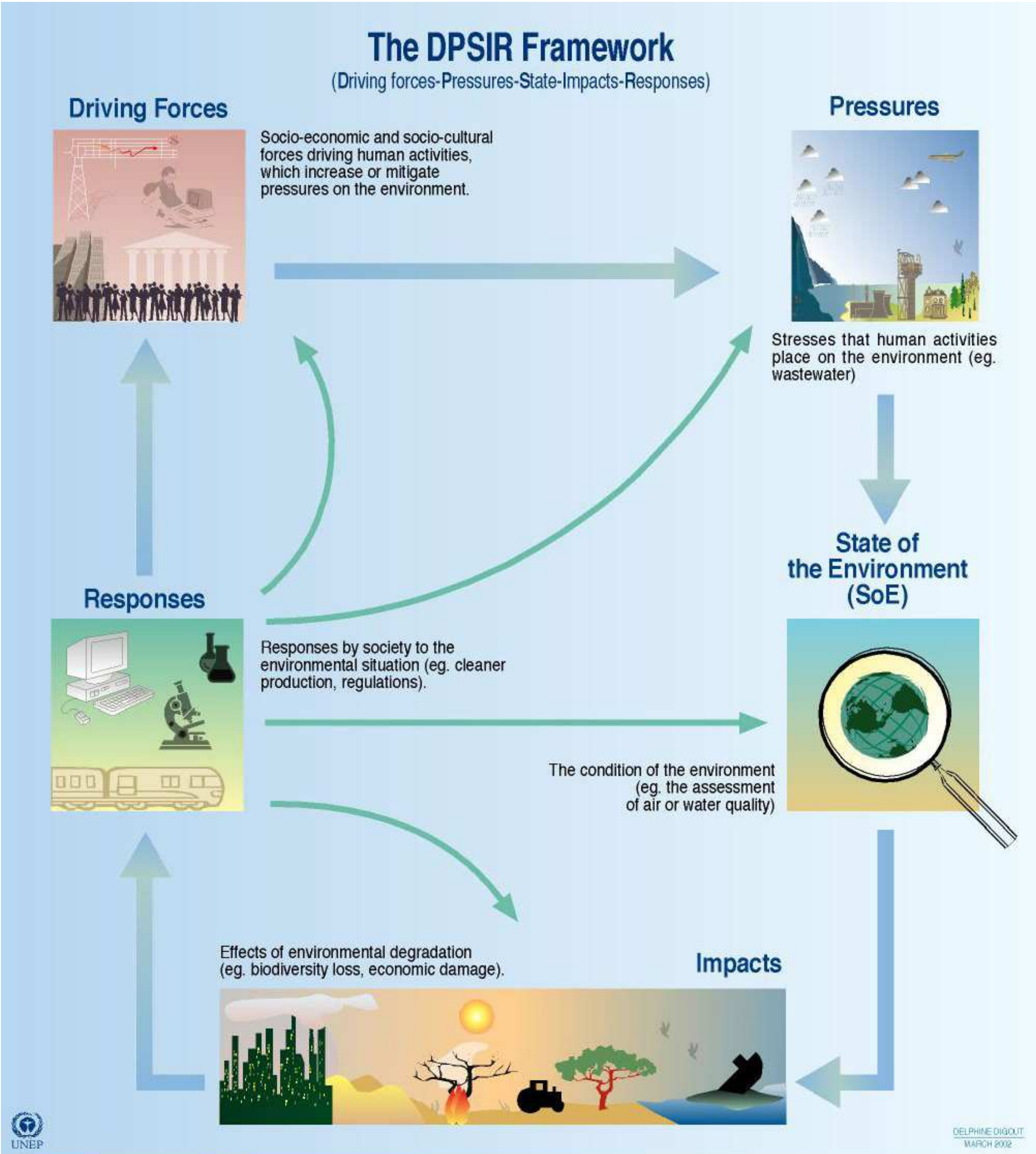
Models for hydrodynamic, chemical and biological processes exist for each of the three individual parts of the sewer-WWTP-river system and the challenge today is to pursue a more holistic approach, meaning the creation of an integrated model which connects these three submodels.

2.3.2 The WFD modelling context

It is widely accepted that modelling will play a major role within the WFD context (e.g. Wasson *et al.* (2003)). An underlying framework to the implementation of the WFD is the *Drivers-Pressures-State-Impacts-Response* (DPSIR) framework (EEA (1999), see Figure 2.3) for organising information about the state of the environment.

Rekolainen *et al.* (2003) redefine this framework and make a conceptual change for 'state' and 'impact' adapting them to 'chemical state' and 'ecological state', by arguing that ecological quality indicators define surface water status in the first place. Nevertheless, whatever is considered as the 'impact' factor to the specific river catchment, a large panoply of different kind of models go within all the different steps of the DPSIR and the different processes that influence the case study. Model types range from hydrological models to chemical and ecological models, as well as from management to socio-economic models. For the WFD implementation, modelling will be needed (Dorge & Windolf (2003), Rekolainen *et al.* (2003)):

- to improve *description* of river basins, i.e. to fill *information* gaps and to understand interactions,
- to understand the extent of *pressures* exerted within a catchment and to establish *reference conditions* as mentioned in REFCOND (2003),
- to design *monitoring programs* and interpolate monitored data, in order to improve the description and qualification of river basins,
- to perform *operational planning*,
- as instruments for *cost-effective* implementation of measures,
- to assess impacts to produce *management plans* and to tackle *multidisciplinary problems*,



Source : Global International Water Assessment (GIWA), 2001; European Environment Agency (EEA), Copenhagen.

Figure 2.3: DPSIR (Driver-pressure-state-impact-response) framework EEA.

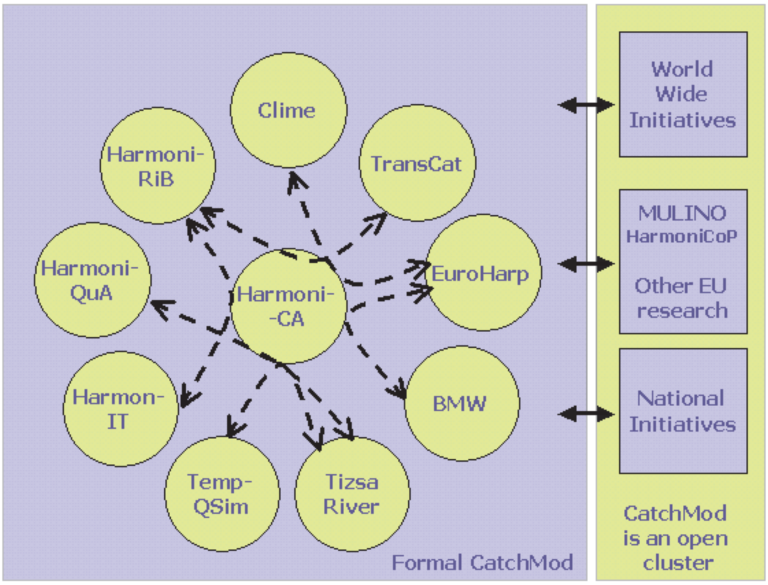


Figure 2.4: CatchMod project cluster within the FP5.

- to include *socio-economic* contexts and investigate the effect of water *pricing* on consumption.

Under the Fifth Development Programme (FP5), a project cluster on Integrated Catchment Water Modelling (*CatchMod*) was financed in order to support WFD implementation (see Figure 2.4). To name just a few, the *Harmoni-CA* project on Harmonised Modelling tools for Integrated Basin Management provides a forum for communication between projects, harmonised use and development of ICT tools within river basin management. The project *HarmonIT* focussed on the development of the tool *Open Modelling Interface and Environment (OpenMI)*. The OpenMI *Interface* enables data exchange between individual softwares when they run. In case the softwares in question are not Open-MI compliant yet, the OpenMI *Environment* provides software tools to made existing model codes compliant (Gijsbers *et al.* (2005)). *HarmoniQuA* is another research project and has developed the computer based Modelling Support Tool *MoST* to provide a user-friendly guidance and support for multi-disciplinary modelling and to serve as a quality assurance framework that will contribute towards enhancing the credibility of catchment and river basin modelling (Old *et al.* (2005)). *BMW* (Benchmark Models for the Water Framework Directive) has the objective to develop criteria to select appropriate models and integrated modelling systems to be use in the WFD process. Also under FP5 was the MULINO project, providing a decision support system (DSS), called MULINO DSS, aiming at integrating hydrologic, socio-economic and environmental models in a multi-criteria analysis tool (Mysiak *et al.* (2002)).

2.3.3 IUWS modelling

Since more than 15 years, it is well recognised that urban wastewater management cannot rely upon uniform, simple emission standards from sewer and WWTP (Lijklema *et al.* (1993), Schilling *et al.* (1997)). Indeed, every receiving water has its own physical, chemical and biological properties and must therefore be evaluated individually with respect to the effluents of the urban catchment. This asks for a coupled operation of the sewer and the WWTP, guided by what is best for the river water quality (e.g. Bauwens *et al.* (1995), Rauch *et al.* (1998a)). The emissions from the sewer network, the actions taken in an urban catchment and the effects on treatment plant performance or concentrations in the river are difficult to assess and correlate due to the complex interaction of processes in the system (Rauch & Harremoës (1998)). Langeveld *et al.* (2002) point to the importance of the dynamic interactions between sewer and WWTP to assess performance of the urban wastewater system. The same remains true at the CSO-river and WWTP-river interfaces and to represent these dynamics, models of the system can be built to perform virtual experiments.

Modelling of the IUWS allows detailed system analysis and can eventually give answers to questions regarding for example the origin and quantity of pollution into the receiving river, stemming either from the WWTP or, during wet weather conditions, from the combined sewer overflows (CSOs). Its purpose can be to estimate impacts on river morphology or water quality. However, it is not a trivial task to build such a model, first of all due to the complexity of the integrated system and therefore the model itself, and secondly due to the difficulty for the user to choose the appropriate sub-models for the integrated model out of a multitude of possible options (Rauch *et al.* (2002)). Certainly a model that is meant to comprise all the pollution inputs on a river basin scale needs a different degree of process detail than a model simulating behaviour of bacteria populations in the immediate vicinity of a wastewater treatment plant effluent. Hence, the choice depends on the objectives of the study in question but also on the availability or not of data (e.g. Willems (2003)). Indeed the model can only describe processes for which information on parameters is available. The construction and analysis of such models is not straightforward and a lot of efforts are spent in investigating these issues further.

Projects include, for example, comparison of different model approaches, further development of models for better linkage of submodels, harmonisation of submodel variables, analysis of sources of uncertainty, etc. Model applications are performed in view of various goals. An integrated model can be used to test scenarios in order to evaluate future impacts e.g. future housing construction or increase of drained impervious surfaces, or to assess certain measures intended to improve performance of the system, e.g. treatment volume increase at the WWTP or in-stream aeration of the river (e.g. Frehmann *et al.* (2002b), Vandenberghe & Vanrolleghem (2005)). Other applications include evaluation of operating strategies (e.g. Vanrolleghem *et al.* (1996), Erbe *et al.* (2002a)) like influent load increase to the WWTP or implementation of immission-based real-time-control (RTC) (e.g. Meirlaen *et al.* (2002); Vanrolleghem *et al.* (2005a)). Benedetti & Vanrolleghem (2007) use integrated models for planning of the WWTP. Overall, problems encountered with integrated model studies are the heaviness of the model and the data availability. Good overview on performed integrated studies can be found in Meirlaen

(2002). Below are some selected, non-exhaustive, examples of model implementations with different objectives and using different tools.

Rauch & Harremoës (1997) have analysed the IUWS via *extreme statistics*. Each detrimental effect has a certain return period, and the authors characterised the system's response through this return period for acute water pollution. It could be shown that CSOs and WWTP effluents cannot be considered separately and for a hypothetical case study it is found that prolonged hydraulic overloading of the treatment plant in order to reduce overflows can in turn hamper the plant efficiency and cause serious impact on water quality, hence showing that CSO events are coupled to the operation and emissions of the treatment plant. Later, with the same approach, the probabilistic software tool REBEKA has been developed for small, alpine rivers taking into account toxic as well as erosion impacts (Rauch *et al.* (2002)).

Dempsey *et al.* (1997) developed a tool called SIMPOL, representing key urban processes in a simple way to allow for quick assessment of a system. Quality parameters have been calibrated using information from a more detailed model and the authors speculate that the accuracy lost for a single event can be compensated by a larger number of simulations, and that the model is well suited to take into account *stochastic* processes like rainfall or dry weather composition. The output of CSO spills and WWTP effluents will serve as input files for river quality simulations.

Schütze *et al.* (2002) created the simulation and optimisation software SYNOPSIS in order to test *control strategies* in the sewer for the best interest of the river water quality (also in Butler & Schütze (2005)). Using the above named tool, Lau *et al.* (2002) showed that, above a certain threshold, further increasing *retention volume* does not have any significant added value for receiving water quality. It is agued that a reduction in direct untreated discharges, together with maintenance of WWTP effluent concentrations, more pollution load must be released from the WWTP over a longer time period.

Ignoring *design load criteria* at the WWTP, Seggelke *et al.* (2005) used online measurements of the actual WWTP state and the effluent characteristics, together with a sewer-WWTP model in KOSIM and SIMBA to predict the maximum possible inflow load that can be treated by the plant during wet weather without compromising the receiving water quality. Wiese *et al.* (2002) also showed that an increased inflow can be economically and environmentally favourable depending on the WWTP treatment capacity, thus promoting *integrated design and operation*. Other similar integrated studies in Germany are described by Erbe *et al.* (2002a) and Frehmann *et al.* (2002a), where the operation of retention basins in the sewer network was controlled. Here, it could be shown that the obtained *CSO reduction*, especially during small rain events, does largely compensate the increased WWTP effluent pollution due to prolonged high flow through the plant. This suggests that, comparing to results from above paragraphs, each integrated solution has to be tailor-made for the case study and depends on the volume or treatment capacities, the receiving water, the rain events,...

As already mentioned before, pollution should already be reduced at the source, and simpler model approaches can be used to characterise substance flows. Such development of a *substance* *flow model* in order to create a decision support tool where different source control options can be tested led to creation of the model SEWSYS® within MATLAB/Simulink to simulate 20 different substances based on pollution loads (Ahlman (2006)). The model includes generation of air pollution, traffic, wet and dry deposition of materials and stormwater generation and transport as well as sanitary flows. Tested scenarios range from reduction of construction material pollutant sources (e.g. tiled instead of copper roofs), reduction of traffic-induced pollution, stormwater infiltration to sedimentation basin solutions. One of the conclusions has been to opt for the addition of treatment facilities once pollutant sources are reduced by a maximum.

Fronteau (1999) also demonstrated the usefulness of an integrated approach to water quality management. To overcome the problem of **compatibility between the models**, Vanrolleghem *et al.* (1996) introduced conversion factors and in Meirlaen *et al.* (2001), a connector model is proposed. In the PhD thesis of Meirlaen (2002), whose work is the basis for this project, the calculation times were also reduced by *model reduction* so that only the processes and variables influencing the control strategy under study were retained. In one of the considered case studies, immission-based *real-time-control* is applied where an ammonia sensor in the river dictating how long overflows can happen before the river concentration reaches a certain set point and WWTP is overloaded. *Calibration of conceptual models* on the basis of simulations of complex models is performed by Meirlaen *et al.* (2002).

Such calibration from complex calibrated models is also proposed by Willems & Berlamont (2002) in order to reduce calculation times, and through probabilistic modelling they try to identify *sources of uncertainty* in the models. They find that hydraulics seem to have little contribution compared to the uncertainties related to water quality calculations. This is explained by the fact that the geometry of the sewer network is well known and existing flow data are more or less reliable. Mannina *et al.* (2005) have built an IUWS model, based on simplified processes, to assess uncertainty sources and claim that sewer system parameters have a large influence on quality characterisation in the river.

A multi-objective evolutionary algorithm is used by Muschalla (2006) to optimise the IUWS with an integrated model presenting the required detail to allow for multiple long-term simulations of the whole system. Based on economical and ecological objective functions, various measures can be optimised regarding dimensioning and locations.

Reda & Beck (1997) and Duchesne *et al.* (2001) have also tested different WWTP loading scenarios under stormwater using receiving water quality based criteria. In addition they have tested the *robustness of the scenario ranking* results under different model parameterisations, and both found that ranking for the best scenarios.

The inclusion of all subsystems into *one software* was realised by Meirlaen *et al.* (2000) and Erbe *et al.* (2002b). Recent development is the software CITY DRAIN© (Achleitner *et al.* (2007)), although implementation of processes for the conversion of matter in the river is still in progress. It was, like the here presented case study, developed within the CD4WC project and applied on the case study of Vils (Austria) to test several solutions for technical operability as well as for ecological and economical feasibility (Ebenbichler *et al.* (2006)).

2.4 New Developments in IUWS Management and Modelling

Harremoës (2002) summarises and comments the outcomes of the INTERURBA-II and identifies lacks and future directions for integrated urban wastewater system analysis. Best modelling and simulation results are certainly obtained for hydraulics in sewer networks and for treatment processes at WWTPs. However large amounts of uncertainty remain and these should be quantified together with risk management during decision-making. Very often the river system is omitted from application studies, not only because of data scarcity, but certainly also due to lack of knowledge regarding basic mechanisms in receiving waters. In this context, the 6th Framework Programme (FP6) has funded the REBECCA project on the relationships between physico-chemical and ecological status of the river, through collection of existing knowledge, identification of gaps and development of statistical tools to link the physico-chemical and ecological water quality. This is especially important for the WFD implementation and the research needs remain in the understanding of underlying phenomena and in elaboration of ecological dose-response models (Rekolainen *et al.* (2003)).

The Thematic Workgroup 2 (TWG2) on Water Supply and Sanitation in Urban, Peri-Urban and Rural Areas of the Water Supply and Sanitation Technology Platform (WSSTP) (www.wsstp.org) (FP6) has identified in its Vision Document a wide range of tools, techniques, technologies and process solutions for future research and development topics, and stresses the importance to recognise the need for integrated approaches to water management. The latter does not only include the combination of measures and interactions between different subunits in the wastewater system, but also emphasizes that socio-economic and cultural backgrounds need to be included into the considered study and therefore decision-making process. Acceptance of innovative techniques could enhance mass production of components of decentralised systems and therefore reduce their costs. Such techniques are especially interesting for regions where water resources are scarce and education in the domain will be important to enhance effective water supply and sanitation for a step towards improvement of hygienic safety and health (Wilderer (2003)).

Especially interesting for the existing centralised systems is, a software like OPEN-MI (see section 2.3.2), gives stakeholders the possibility to connect models they already have in different softwares, thereby enhancing collaborations towards integrated consideration of the water cycle. It can allow for cost-effective optimisation and goes in line with the suggested river basin management of the WFD. Within the Environment topic of the 7th Framework Programme (FP7), the new call (EC (2006)) specifically asks for projects in water bodies and resources management, also within a climate change context. It also points to research needed in relation to megacities, something that is directly linked to urban wastewater management, both regarding the extraction and treatment of large amounts of wastewaters and the subsequent ability of a receiving water to cope with such quantities of water and pollution. Facing scarcer and less reliable water resource and population growth, such research is investigated within the SWITCH project (FP6), whose goal is better management of urban water in nine case study cities world wide.

A step forward in model application is certainly the collection of data. Often phenomena related to water sciences are in need for large and long sets of data. It is therefore important to invest into technologies and use them within the WFD implementation (Allan *et al.* (2006)). Data are also needed in order to improve calibration of models (Silberstein (2006)) and indeed, data and models have to complement each other to both bring forward fundamental knowledge in the field of water and help to find engineering solutions for specific case studies. The FP7 call also mentions 'river basin twinning as a tool to implement EU initiatives', i.e. required harmonisation among river basins is required to transfer methodologies or tools from one basin to another within, for example, the WFD. Altogether, integrated resource management needs to be done in international co-operation partner countries, and experiences collected over the years should accelerate competences and infrastructures in developing countries.

2.5 Aim and Challenges of the Thesis

The *overall aim of the PhD thesis* is the development and application of a procedure to improve the integrated urban wastewater system (IUWS) by model-based evaluation of scenarios for system amelioration. This procedure consists of the construction of an integrated model of the urban wastewater system including urban drainage, WWTP and receiving river in order to perform, via model simulations, an impact analysis of various system (re)configurations.

The adopted procedure (see section 2.6) is meant to fit within the WFD implementation process, both in general and in relation to a real case study in Luxembourg. It should push forward the idea of a holistic approach within IUWS management, i.e. that the management problem is put into a global context before specific details are investigated.

The *challenge for modelling* of the IUWS lies above all within the structural complexity of the system itself. Besides the system's large spatial extent, the complexity is also the result of its non-linear dynamics that are the result of a complex interplay of a wide diversity of processes. Hence, setting the level of detail of the representation of such a system in order to attain the set goals is a global challenge of such modelling exercise and, in particular, for the here presented work on the case study.

> "The key point is that an engineering model has to model the essential features that are important to the resulting design and operation. All other details just obscure the picture and hamper engineering application" (Harremoës & Madsen (1999))

Within the WFD context, which is setting the overall objectives, this work is an attempt for simultaneous and coherent analysis of 3 systems that are often considered individually. The latter is due to the different problems applicable to the subsystems and therefore the different variables of interest, so that modelling approaches vary and individual software is typically used for each of them. Hence, to perform the abovementioned coherent analysis, the idea is pursued to build the integrated model in one software, here the modelling and simulation software WEST® (MOSTforWATER, N.V., Kortrijk, Belgium), to ensure easy connection of submodels. Therefore, one specific objective within this dissertation was the *expansion of the WEST® modelbase* with models for urban runoff and sewer transport, so that they can be applied together with the WWTP and river models already available in this modelling software.

The presented integrated modelbase becomes the basis for a *harmonised modelling and calibration* approach. The level of complexity, i.e. the level of detail of the different IUWS-models, is sought to be homogeneous and variables taken into account in one subsystem are easily transformed to be used in the downstream system. The model is a compromise: it is not excessively complex to avoid too long calculation times, however it contains all processes and variables considered necessary for the impact analysis. Indeed, the model structure should not be such that a too large number of parameters needs to be calibrated, as this would require an excessive amount of data. This element points to another side of complexity of IUWS-modelling, i.e. the limited data availability generally found for large model constructions with application and illustration on a real case study. To deal with this, the *planning and execution of targeted measurement campaigns* to fill information gaps and to produce data for use in model calibration, is required. The application of the methodology on a *real case study* with its merits and its flaws is therefore illustrating this challenge in integrated modelling.

The pursued goal of analysing the impact of various system (re)configurations asks for the development of a general method that allows for *straightforward interpretation* of the enormous amount of data that is generated by the long-term dynamic simulations of the different scenarios. The method should be easily applicable to different case studies and the chosen evaluation criteria should be suitable in the context of the WFD implementation.

With these problems in mind, the *ultimate concern* of this dissertation would be that the results of the work allow creating a user-friendly tool that can be operated by decision-makers or engineers working in the field for a better and faster evaluation of the needs and possibilities for a particular case study. The evaluation method of the simulation outcomes is certainly critical in this sense as it should give a good overview on advantages and disadvantages of each tested system (re)configuration. Moreover, the pursued merging of different science branches, i.e. hydrology and urban drainage, wastewater treatment and engineering, river water chemistry and ecology will bring about easier interdisciplinary communication between often different authorities mostly concerned either with sewer systems, WWTP's or rivers respectively.

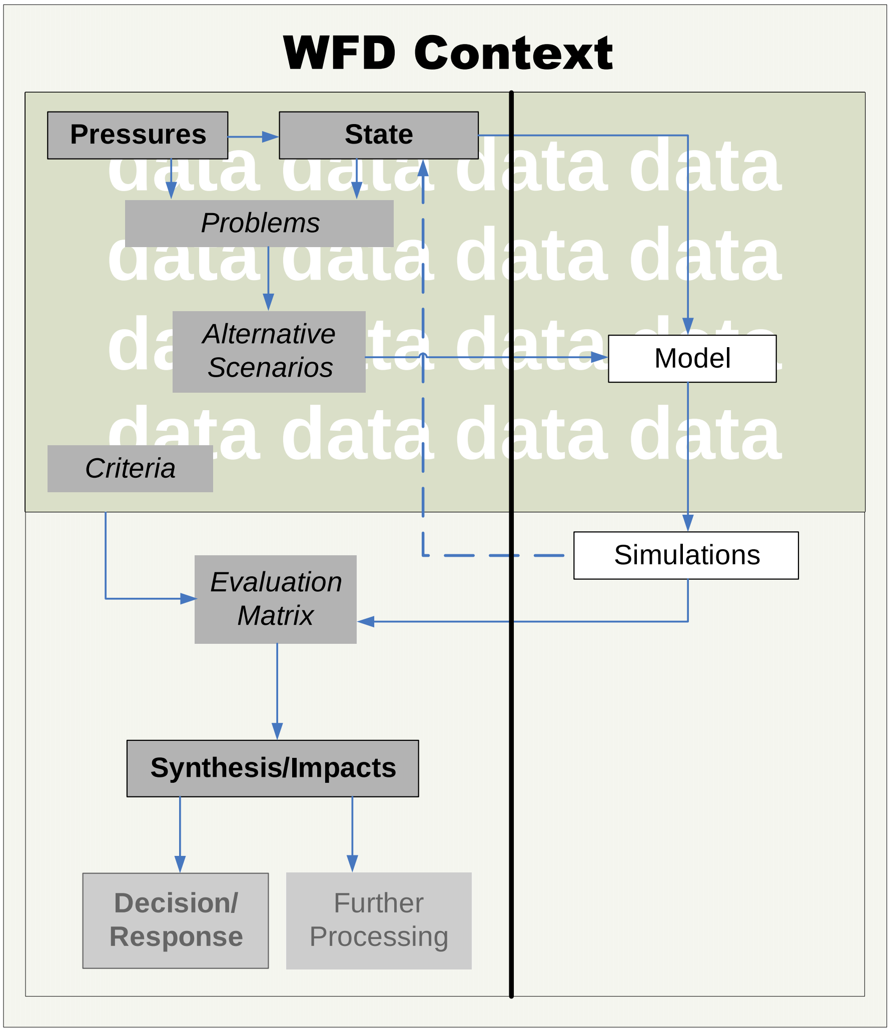


Figure 2.5: Overall scheme of the impact analysis approach adopted in this thesis.

2.6 Adopted Approach

The *Driver-Pressure-State-Impact-Response* (DPSIR) framework (see Figure 2.3), introduced in section 2.3.2, fits well into what is required by the WFD and has been chosen to serve as underlying structure for the approach adopted in this thesis (see Figure 2.5). To start the analysis, existing *data* and knowledge on the integrated case study will serve to characterise the *pressures* exerted on the receiving water in terms of water quality and to describe the *state* of the whole system. Characterisation will serve to point to *problems* within the subsystems, i.e. catchment, sewer system, WWTP and receiving river, and to define *scenarios* expected to improve system performance. At the same time, based on the objectives and the data, an integrated *model* is built to represent the system. Additional, more dynamic, *data* is collected in an integrated measurement campaign in order to fill information gaps that would limit the quality of model calibration. First *simulations* of the integrated model will help to further understand the state of the system and inter-relations among the subsystems.

Once models exist to evaluate all scenarios, scenario simulations are performed and, according to defined water quality *criteria* in agreement with the WFD, results are compared to the reference state, i.e. to the simulation results of the system as it exists now. To get a good visual overview of the large number of simulation results, these are compiled into the so-called *evaluation matrix* developed in this dissertation. Implementation costs of scenarios are also estimated. Although this is not contained within this work, the results are meant to then serve in a *decision* process of the operator, or, within the WFD implementation, can be *further processed* and used to feed other tools to establish management plans to reach 'good' ecological status of the river.

On a time axis, the PhD study started in September 2003, ended in September 2007 and, although there is some overlap between them, the Chapter numbers roughly follow the chronological order of the main steps of the study. The Chapters of the thesis are linked to the steps of the here adopted approach and Figure 2.6 gives an overview.

Chapter 2 sets the context by providing some background information and the state of the art literature on the WFD, urban water management practices and integrated modelling.

Chapters 3 and 5 concentrate on the modelling tool and the integrated model of this case study. Chapter 3 focuses on the more theoretical background of urban runoff and sewer transport models, implemented into the WEST® modelbase within this thesis. The modelbase built further on the embryonic version developed by Meirlaen (2002). Also presented in this Chapter are the WWTP, river and connector models, already available in WEST®. Chapter 5 explains how the 'Bleesbruck' sewer - treatment plant - river system was represented using the available models and illustrates the calibration exercise for each of the submodels.

Chapter 4 presents the case study and the two integrated measurement campaigns for WWTP and receiving rivers, that were conducted within the CD4WC project [1], together with collaborators from Ghent University.

Chapter 6 explains the selected system optimisation scenarios for the 'Bleesbruck' case study that are evaluated using the integrated model. It presents the evaluation criteria for WFD compliance and the here developed evaluation method dealing with the large amount of simulated data.

In *Chapter 7* the conclusions of the work are drawn and perspectives for further work are presented.

[1] The project goes within the *CityNet* cluster (see Figure 2.2), a research project supported by the European Commission under the FP5. It is contributing to the implementation of the Key Action "Sustainable Management and Quality of Water" within the Energy, Environment and Sustainable Development Contract No: EVK1-2002-00570 (www.tu-dresden.de/CD4WC).

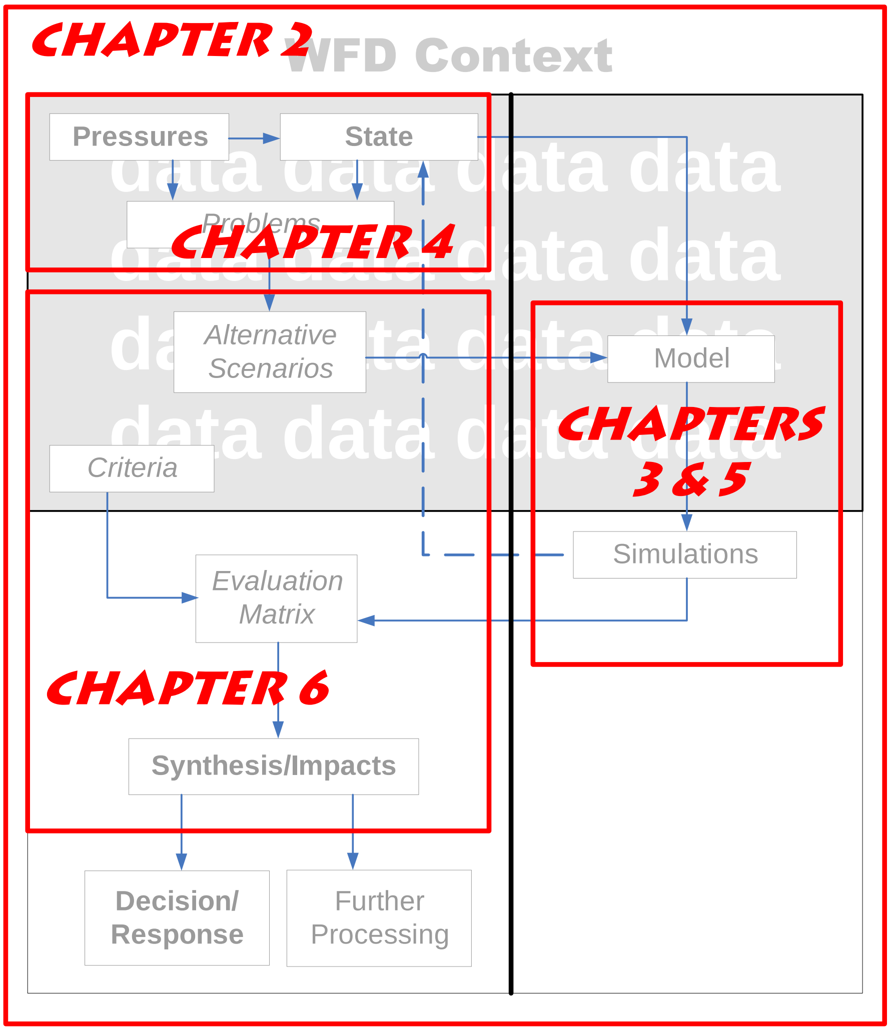


Figure 2.6: Overall scheme of the impact analysis approach related to the Chapters in the thesis

Chapter 3

Modelling & Simulation Tools

This Chapter first describes the WEST® software platform used in this study, together with a new software kernel for better calculation performance, called Tornado. Thereafter, hydrologic modelling as opposed to hydrodynamic modelling of sewer transport is briefly presented. The hydrological model KOSIM-WEST for catchment runoff and sewer transport, as implemented into WEST®, is presented. Simulation results from a hypothetical case study are compared to the original KOSIM model to verify that the underlying models are the same. The model is also applied on a real urban catchment with storage tank. In the last part, model principles for WWTP and river systems as well as connector model principles used to link the submodels are explained.

The following chapter is partly developed from and contained in the following article:

Solvi, A.-M., L. Benedetti, S. Gillé, P. M. Schosseler, A. Weidenhaupt and P. A. Vanrolleghem (2005). Integrated urban catchment modelling for a sewer-treatment-river system. *10th International Conference on Urban Drainage*, 21-26 August 2005, Copenhagen, Denmark.

3.1 Presentation of the Software WEST®

In this project and case study, WEST® (**W**astewater treatment plant **E**ngine for **S**imulation and **T**raining) (MOSTforWATER N.V., Kortrijk, Belgium), version 3.7.2, is used as software platform. It is presented in more detail in Vanhooren *et al.* (2003), Meirlaen (2002), Nopens (2005), from which parts of this section were derived. The main application of the software is the modelling and simulation of wastewater treatment systems, but basically any kind of processes that can be described by differential and algebraic equations (DAEs) can be represented in and simulated by WEST®. Next to the fact that, among the BIOMATH group (Ghent University), experience with WEST® existed, it seemed to be an appropriate software to use in this context: first of all due to his 'open' model base (see below), then its user-friendliness, so that new developments can be brought to the end-user easily and because it has reasonable calculation times.

All available models can be viewed and modified by the user in the **Model Editor** environment. Models are described in MSL-User (MSL stands for **M**odel **S**pecification **L**anguage), a high level object-oriented language specifically developed to incorporate models. The model base is aimed at maximum reuse of existing knowledge and is therefore structured hierarchically. All reusable knowledge - such as mass balances, physical units, default parameter values and applicable ranges - is thus defined centrally. WEST® has hence an open structure in that the user is allowed to change existing models. It also gives the possibility of adding models to those already present, like the IWA standard activated sludge models (ASM1, ASM2, ASM2d, ASM3, Henze *et al.* (2000)) and the river water quality model (RWQM1, Reichert *et al.* (2001)) for the WWTP and the river respectively. As will be described in detail in section 3.3, parts of the original KOSIM model (ITWH (2000)) were implemented into the WEST® model base.

In the **Configuration Environment** (see Figure 3.1), the user can graphically build the considered system under study (e.g. a WWTP). For each of the subcomponents of the system (e.g. activated sludge reactor, clarifier), the user can choose from a series of models that are coded inside the model base. Once the system configuration is set up, the complete model will be written automatically in MSL by extracting the relevant equations from the model base, the MSL-file is then parsed into low-level C-code, which in its turn is compiled to an executable WEST®model library (WML-file), which can be loaded into the **Experimentation Environment** (see Figure 3.2). Here, different 'virtual' experiments can be run with this model, parameter values for the model can be manually changed by the user, but also automatic parameter estimations, scenario analysis, senstivity analysis and optimal experimental design can be performed. The environment also allows for graphical presentation of the results. In this study, models were numerically solved using the Runge-Kutta 4th order algorithm with adaptive step size (Gerald and Wheatley, 1994) and the CVODE stiff solver (Cohen & Hindmarsh (1994)).

Due to the fact that models tend to become more and more detailed and also larger in size, the need for powerful software infrastuctures capable to handle such models remains. This is certainly true in water quality and integrated modelling where the complexity does not lie within the nature of its equations but in the sheer number of them. The **Tornado** engine is a new software kernel for WEST® which introduces a higher flexibility and results in a better

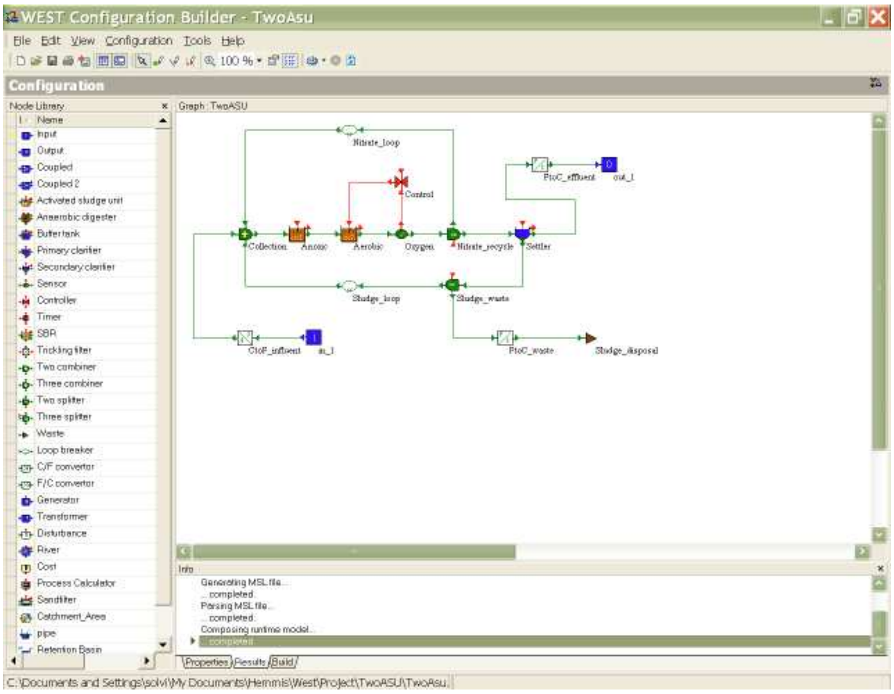


Figure 3.1: Graphical interface of the configuration environment in WEST®.

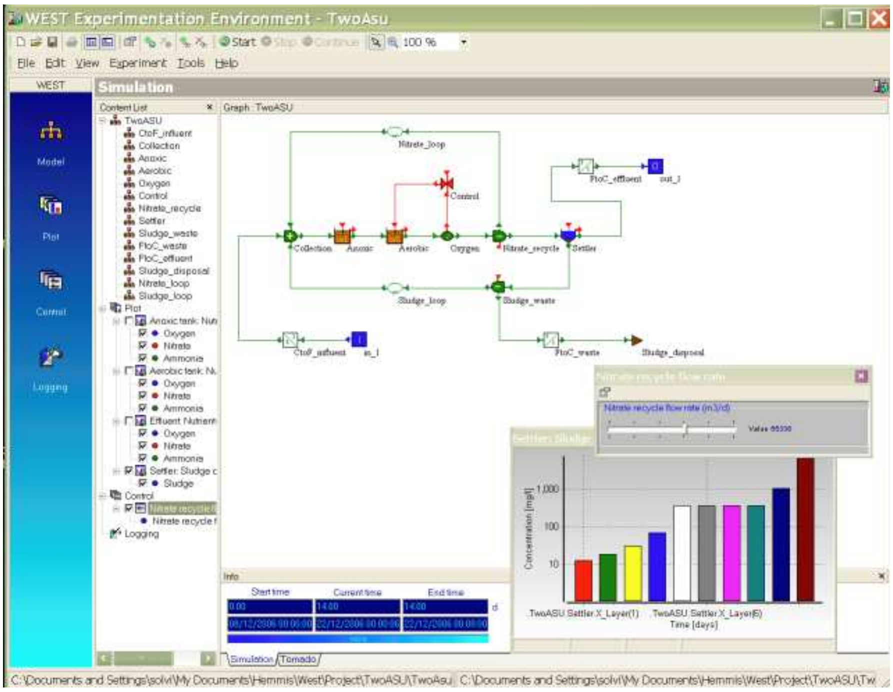


Figure 3.2: Graphical interface of the experimentation environment in WEST®.

performance (Claeys *et al.* (2006)). Although Tornado will only be available in the next version of WEST® (version 4), Tornado could already be used within the context of this integrated modelling case study which allowed to reduce simulation times by roughly a third.

3.2 Modelling of Urban Drainage

3.2.1 Subsystems and processes

From rain to collector effluent, water volumes undergo certain gains and losses. The same is true for pollutant loads (e.g. COD, metals, polyaromatic carbons, ...) and the corresponding pollutant concentrations will depend on the amount of water present.

Subsystems within urban drainage are the atmosphere, the drained surface and the sewer network. *Processes* within the subsystems depend on many factors, among them the *weather* conditions. Table 3.1 summarises processes occurring in a combined sewer system (inspired by Euler *et al.* (1986), Paulsen (1987)), also indicating which ones are modelled in KOSIM-WEST.

Table 3.1: Urban drainage processes overview for a combined sewer network.

<table>
<tr><th>Subsystem</th><th>Water</th><th>Pollutants</th></tr>
<tr><td colspan="3">DW (Dry Weather)</td></tr>
<tr><td>Atmosphere</td><td>Evaporation</td><td>Accumulation*
Deposition*</td></tr>
<tr><td>Surface</td><td></td><td>Accumulation</td></tr>
<tr><td>Sewer network</td><td>DW flow</td><td>DW pollution
pollutant transport
sedimentation
resuspension
biochemical processes*</td></tr>
<tr><td colspan="3">WW (Wet Weather)</td></tr>
<tr><td>Atmosphere</td><td>Rain
Evaporation*</td><td>Washout*</td></tr>
<tr><td>Surface</td><td>Runoff generation</td><td>Washoff</td></tr>
<tr><td rowspan="5">Sewer network</td><td>DW flow</td><td>DW pollution generation</td></tr>
<tr><td colspan="2">mixing DW and WW flow and pollution</td></tr>
<tr><td colspan="2">storage</td></tr>
<tr><td colspan="2">combination and splitting</td></tr>
<tr><td></td><td>sedimentation
resuspension
biochemical processes*</td></tr>
</table>

*: not represented within the KOSIM-WEST model

Although pollutant processes within the atmosphere will not be considered in the model, the availability of certain pollutants on the surface depends on activities and emissions into the air within the considered catchment. During runoff generation on surface during rain events, part of the rain water is lost due to wetting, depressions, evaporation and/or infiltration to generate what is called *effective rain*. This effective rain can washoff the particulate matter that has been accumulating during the dry weather period. The routing of the water over the surface, where peaks are time translated and suffer from retention, can be considered as a first period and in a second phase, taking place inside the sewer network, dry weather flow adds to the stormwater. These effluent curves are again submitted to retention and change through combination and splitting of flows or storage. Pollutants settle and get resuspended in the network and biochemical transformations take place. More details on the processes and the way they are modelled within WEST® are given in section 3.3.

3.2.2 Hydrological versus hydrodynamic modelling of water

Due to often very heterogeneous conditions on the surfaces, surface runoff in urban drainage simulations is generally modelled using simplified, i.e. hydrological principles. In the sewer system however, geometric data from pipes and structures clearly define boundaries for water transport and make hydrodynamic modelling and simulation possible. The physical-mathematical representation of the water transformations in the sewer system can be described by first order partial differential equations, the so-called Saint-Venant equations (e.g. Hager (1999)), composed of a continuity equation for mass conservation and a momentum equation for energy conservation.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{3.1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\frac{\partial y}{\partial x} - gA\left(S_o - S_f\right) = 0 \tag{3.2}$$

y	water depth
t	time
Q	flow rate
x	distance
A	area of flow cross-section
g	gravitationnal constant
S_o	bed slope
S_f	friction slope

Solving equations (3.1) and (3.2) numerically will give information on variables $Q(x,t)$ and $y(x,t)$. In a hydrological consideration of the sewer system, pipes will be modelled as 'black-box' models using a transfer function for transport without exaclty representing physical processes in the structure. The main idea behind the very often used Kalinin-Miljukov method (Euler (1973)), also applied in this study, is to consider the unsteady flow in the pipe as being steady in stretches of certain length. This assumption then allows to model each of these sections as a linear reservoir, thereby replacing the continuity equation (3.1) by a retention equation and the momentum equation (3.2) by a 'Flow-Volume' relationship of the form:

$$\frac{dV}{dt} = Q_{in}(t) - Q_{out}(t) \tag{3.3}$$

$$Q_{out}(t) = \frac{1}{k}V(t) \tag{3.4}$$

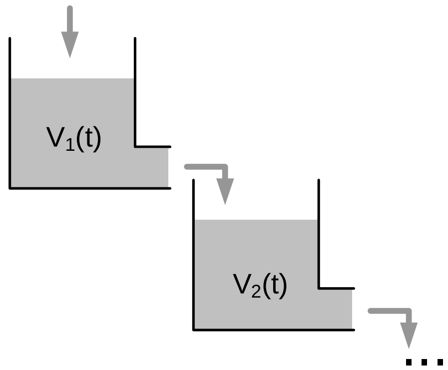


Figure 3.3: Linear reservoir cascade.

where Q_{in} and Q_{out} are in- and outflows, and V represents the water volume in a considered tank. We obtain a linear tank cascade (see Figure 3.3) and the parameters, such as the required number of tanks n and the residence time k of the transfer function transforming inflow into outflow are determined from the physical properties of the pipe (see section 3.3.2.1). Hence, there is no coupling between flux and height of water at different locations and disadvantages of this modelling approach are clearly the accuracy of results in case the system is not under ideal flow conditions, e.g. especially in structures where downstream structures/pipes have an effect on the behaviour of the water upstream.

Within the integrated context of this study, hydrologic modelling of the urban drainage has nevertheless considerable advantage compared to hydrodynamic modelling. Besides the much lower calculation times, hydrologic models have a low need for calibration data due to the reduced number of parameters, provide a better overview on the model structure so that it becomes easier to handle and run them. Indeed, due to the mathematically simpler representation, these models have obviously higher calculation stability. In this attempt to build an integrated model, it is considered to be an appropriate tool to test integrated case study scenarios on a long term basis. Indeed, if detailed geometric data is unavailable, modelling should be very parsimonious, and only the most dominant processes should be described by using only few parameters. Many goals of simulation studies will not require hydrodynamics and do not make worth the effort and resources to collect the data and construct the model, something that can always be done during a second, more detailed stage of a study. On the other hand, if a more detailed model of a system is available, it might be used to calibrate a simpler model with low calculation times (e.g. Meirlaen *et al.* (2001), Willems & Berlamont (1999)). Note that the hydrological modelling approach will also be used for river flow simulation, which makes the whole integrated modelling approach consistent.

In this case the model is based on KOSIM (ITWH (2000)), which has already been used in an integrated context by Fronteau (1999), Schütze *et al.* (2002), Seggelke (2002) and Meirlaen (2002). Like the conceptually similar SMUSI (Muschalla & Ostrowski (2002)) software, KOSIM was designed to calculate pollutant loads to the WWTP and the receiving waters in the context of planning and dimensioning of sewer system and storage tanks (according to e.g. ATV (1992)) and is widely used and accepted in Germany. Some additional model approaches to the KOSIM 6.2 version have been implemented into WEST® to account for certain features that seemed of importance for integrated consideration of the integrated urban wastewater system, in which emission loads and immission concentrations are looked at: first flushes of particulate matter and backwater effects (see sections 3.3.1.3 and 3.3.2.1).

3.3 The KOSIM-WEST Modelbase

The KOSIM modelling tool (Paulsen (1987), ITWH (2000)) is designed for long-term simulations of dry weather generation, rainfall-surface runoff and transport in the sewer system. The model is able to give pollutant loads (for up to 6 components) in response to individual rain events. The submodels behind are conceptual models for flows and are based on average values for pollution. The mathematical expressions behind the KOSIM models are discrete timestep equations. These have been transformed into the original underlying ordinary differential equations (ODEs) so that they can be combined with the differential and algebraic equations (DAEs) of other subsystems of the IUWS and numerically solved by the solvers contained in WEST®.

To illustrate this conversion on an example, we use the process of surface wetting. The total wetting loss W_{max} is the height of rain needed to wet the surface at the beginning of a rain event and this water volume cannot be included in the runoff. In KOSIM, the calculation is based on consecutive timesteps and the wetting W_t at time step t is

$$W_t = W_{t-1} + i_t \cdot \Delta t, \text{ when } t \leq t_W \tag{3.5}$$

where i_t is the rain intensity at time step t and $W(t_W) = W_{max}$. In WEST® , the expression takes a continuous form, and by approximation in the limit of $\Delta t \to 0$, (3.5) can be written as the differential equation

$$\frac{dW}{dt} = i(t), \text{ when } W \leq W_{max}. \tag{3.6}$$

Figure 3.4 illustrates how such a model is represented in the MSL-User language within the WEST® model base. The code expresses the change in wetting state, depending on whether it rains or not. In the former situation, the change in wetting state is zero if the surface is wet already, or equal to the rain intensity otherwise (see equation 3.6). In case it does not rain, the wetting state is reduced through evaporation $e(t)$, a function given by the evaporation submodel discussed below. Hence, surface drying is given by:

$$\frac{dW}{dt} = -e(t). \tag{3.7}$$

As was explained in the previous section, KOSIM was evaluated to be an appropriate tool for this integrated modelling context, where we do not want to model all geometric details

```
// wetting losses
DERIV(state.WettingLosses,[independent.t]) = state.WettLossChange;
state.WettLossChange =
    IF (interface.In_1[Rain] > 0)
    THEN
        IF (state.WettingLosses >= parameters.MaxWettingLosses)
        THEN 0
        ELSE interface.In_1[Rain]
    ELSE
        IF (state.WettingLosses > 0)
        THEN - interface.In_1[Evaporation]
        ELSE 0;
```

Figure 3.4: Some lines of MSL code

due to model overloading and exagerated data requirements. An embryonic version of KOSIM-WEST was created within the integrated modelling project of Meirlaen (2002) and the tool was completed within this thesis. Not all models and features contained in KOSIM have been translated but only those that were estimated to be necessary here, so as to have the KOSIM-WEST toolbox as detailed as needed and as simple as possible. Examples of omissions are that there is no evaporation taking place during wet weather conditions as these losses were estimated small compared to other losses, pollutants are all modelled to stem from impervious surfaces only in order to keep the number of parameter values to a minimum. Surface flow times are the same from pervious and impervious surfaces, and the storage tanks implemented so far were chosen in relation to those needed for this case study, but due to the open model base, the tank characteristics and models are easily extendable.

Figure 3.5 gives a general overview of the processes and structures that are contained in WEST®. The described system can be divided into two environments where the water is passing through: the catchment (surface runoff and DWF in local sewer networks) and the sewer system (main collector). All elements and processes will be described below.

The extendability of the model base is also true for modelled pollutants; here the variables were chosen for easy connectability with variables from ASM1, ASM2d and ASM3 models (Henze *et al.* (2000)) and RWQM1 (Reichert *et al.* (2001)). Components in the KOSIM-WEST model are water, soluble and particulate chemical oxygen demand (COD), total nitrogen (TN), total phosphorus (TP), ammonia and orthophosphates.

3.3.1 Urban catchment

In the context of this work, the urban catchment is defined as a collection of wastewater producing units with their local sewer system, including the area surrounding these units and connected to the sewer system. The water originating from a catchment and entering the main collector system is hence composed of rainwater from surface runoff and wastewater from households or commercial and industrial sites. A coupled model for the urban catchment was created to bring all processes together (see Figure 3.6). With rain as input data, it contains a model generating

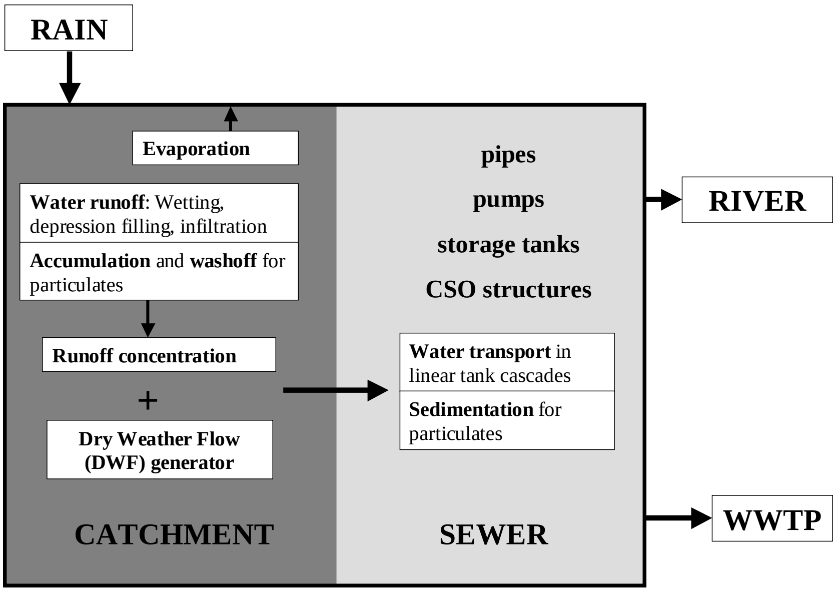


Figure 3.5: Elements and processes within the KOSIM-WEST model.

potential evaporation, a model transforming rain into runoff, a runoff concentration model to account for time of travel and retention of peaks, and a model to generate DWF. The latter is only added after the flow time is taken into account so that, if measurements of DWF after the catchment are taken, the latter can be used as such.

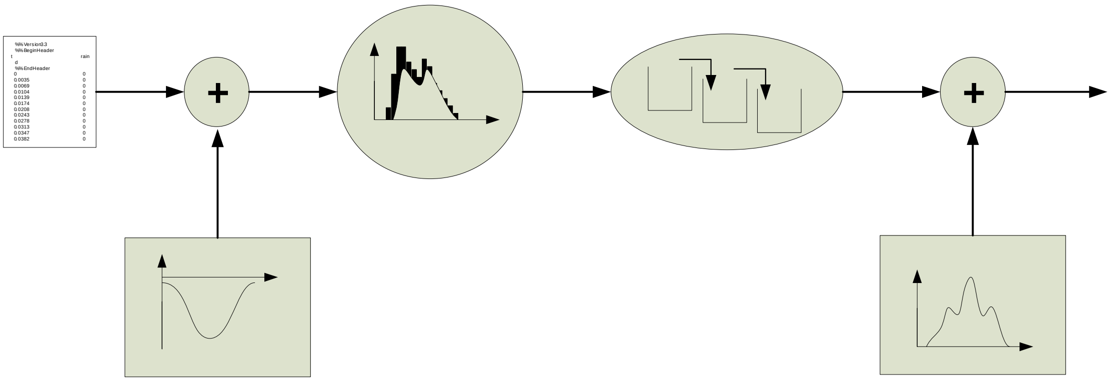


Figure 3.6: Submodels within the 'catchment' model: A potential evaporation model; a second submodel for transformation of the incoming rain data into effective runoff (taking into account wetting, depression and infiltration losses on the surface); a submodel for time translation and retention through a 3 tank cascade and a DWF generator.

3.3.1.1 Rain and Evaporation

Rain input data can be fed into the WEST® model in simple time-rain vector format using the time interval that seems appropriate for the model use. The latter should be as small as

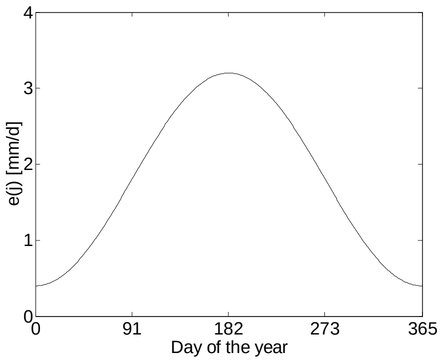


Figure 3.7: Variation of the daily potential evaporation $e(j)$ during the year with $\overline{E}_y =$ 657mm.

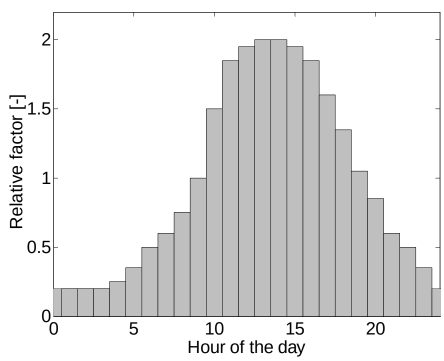


Figure 3.8: Hourly distribution of the mean daily potential evaporation $e(j)$.

possible (e.g. 5 minutes) to make sure to simulate peaks that activate combined sewer overflows. The spatial distribution of rainfall is uniform all over one subcatchment, but can vary for each individual one.

The amount of evaporation $e(t)$ is much higher in summer than it is in winter, and the same is true for day and night. It therefore depends on the day of the year, the hour of that day and a mean annual evaporation $\overline{E}_y$. The potential evaporation $e(j)$ for a specific day j of the year is given by the expression (taken from ITWH (2000) and valid for central Europe):

$$e(j) = \Big[\frac{7}{9} sin\Big(\frac{2\pi}{365} \cdot (j - 91)\Big) + 1\Big] \cdot \frac{\overline{E}_y}{365}. \tag{3.8}$$

To take into account the daily variation of evaporation, $e(j)$ is multiplied by an hourly factor $f(h)$ to finally give the potential evaporation $e(j, h)$ within a certain hour h:

$$e(j, h) = f(h) \cdot e(j) \tag{3.9}$$

Figure 3.7 and Figure 3.8 depict daily potential evaporation and hourly distribution factors. It was estimated that as evaporation losses are small compared to other losses, evaporation only takes place during dry weather and then recovers storage capacities for wetting, depressions and infiltration.

3.3.1.2 Runoff

In the runoff model unit, the incoming total rain is transformed into the effective rain entering the main collector system. The amount of rainwater runoff depends on the area A connected to the network and the proportion of impervious and pervious area expressed by the factor $f_{i/p}$. In fact, whereas evaporation, wetting and depression filling are taking place for both surfaces, infiltration into the soil only happens on pervious areas. During intermittent dry periods, such wetting, depression or infiltration capacities regenerate due to evaporation, as presented in the previous section. To take into account first flush concentrations, KOSIM-WEST uses conceptual models to simulate accumulation and wash-off of particulate matter on impervious surfaces (also contained in the research version of KOSIM, Paulsen (1987)).

Impervious Surfaces As already introduced at the beginning of section 3.3, the wetting losses $W(t)$ are described by the differential equation

$$\frac{dW}{dt} = i(t) \text{ when } W(t) \leq W_{max}. \tag{3.10}$$

where i is the rain intensity and $W(t = t_W) = W_{max}$. W_{max} is the maximum storage volume available for wetting. Surface depressions will start to store the rain water as soon as $t > t_W$ and $W(t) = W_{max}$. Their filling state D is modelled to have an exponential behaviour

$$D(t) = D_{max} \cdot (1 - e^{-c \cdot I(t)}) \qquad \text{with } I(t) = \int_{t_W}^{t} i(t)\, dt, \tag{3.11}$$

where D_{max} is the maximum depression height, c is the rate of storage loss and I the height of rain fallen down after wetting. The equation expresses that, the more rain has already fallen, the less water can be stored in depressions. Hence, the effective rainfall R from impervious surfaces at time instant t is the leftover of the rain after the surface wetting minus the depression losses, times a factor Ψ_{max}:

$$R(t) = \Psi_{max} \left(I(t) - D(t) \right). \tag{3.12}$$

Ψ_{max} is the maximum runoff coefficient and lies between 0 and 1. It allows to distinguish between impervious and connected surfaces and takes care that continuous losses are not included in the runoff (e.g. rainwater reuse). The runoff coefficient function is defined as the change in runoff with respect to the rain, i.e.

$$\Psi(t) = \frac{dR}{dI} = \Psi_{max} \Big(1 - \frac{dD(t)}{dI} \Big), \tag{3.13}$$

To include an initial runoff coefficient Ψ_o that takes into account that, at the beginning of the depression filling process, i.e. at $t = t_W$, some of the connected surface is immediately contributing to the runoff, we can calculate:

$$\Psi_o = \left. \frac{dR}{dI} \right|_{I=0} = \Psi_{max} \left(1 - D_{max} \cdot c \right), \tag{3.14}$$

so that c can be given by

$$c = \frac{1 - \frac{\Psi_o}{\Psi_{max}}}{D_{max}}. \tag{3.15}$$

By introducing the filling degree $\epsilon(t) = D(t)/D_{max}$, and taking its derivative with respect to time, we find:

$$\frac{d\epsilon}{dt} = c \Big(1 - \epsilon(t) \Big)\, i(t). \tag{3.16}$$

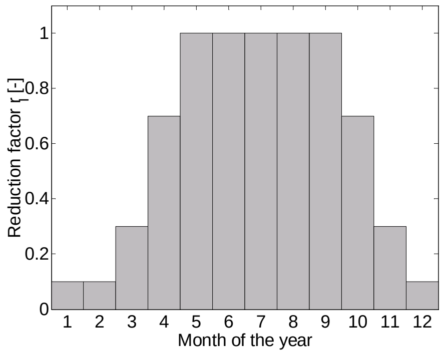


Figure 3.9: Monthly reduction factor for interception and wetting losses on pervious areas.

The runoff intensity a_{imp} at time $t > t_W$ is then given by deriving equation (3.12) with respect to time and by using expression (3.16) of the filling degree, we obtain

$$a_{imp}(t) = \frac{dR}{dt} = \Psi_{max}\left[i(t) - D_{max}\,\frac{d\epsilon}{dt}\right]. \tag{3.17}$$

The flow from impervious area is

$$Q_{imp}(t) = a_{imp}(t)\,\varphi A, \tag{3.18}$$

where A is the total area of the catchment and φ the fraction of impervious to pervious surface.

Similarly to the modelling approach for wetting losses, depressions empty according to:

$$\frac{d\epsilon}{dt} = -c \cdot \epsilon(t) \cdot e(t), \tag{3.19}$$

Pervious Surfaces In contrast to impervious surface wetting, the rain not only wets the pervious surfaces, but also gets intercepted by vegetation. These combined wetting-interception losses are dealt with in the same mathematical manner, but the parameters differ. Also, to take into account seasonal variation of vegetation, a reduction factor (see Figure 3.9) is being introduced to make sure that in winter interception of rainwater is smaller than it is in summer, for example. Parameter values depend on whether the surface is covered by lawn, conifers or deciduous trees (all contained in ITWH (2000)).

Before depressions start to fill, the infiltration process reduces the quantity of water that participates to the runoff. The amount of infiltration that can happen on a surface depends on the nature of the soil, and in KOSIM four parameter categories are given: gravel/grit, fine sand, loess and clay. The model is based on the time-dependent Horton equations for the soil infiltration capacity f, where f_0 is the maximum or initial value and f_∞ the minimum for the infiltration capacity. Using k_- as the regression constant, the infiltration capacity and its derivative are given by:

$$f(t) \;=\; f_\infty + (f_0 - f_\infty)\,e^{-k_- t} \quad \text{for } i(t) > f(t) \tag{3.20}$$

$$\frac{df}{dt} = -k_-\Big(f(t) - f_\infty\Big) \quad \text{for } i(t) > f(t) \tag{3.21}$$

Before discussing infiltration for the case with an amount of rain smaller than the amount of water the soil can take up, the regeneration process of the infiltration capacity in the case of absence of rain is presented. The equation for the infiltration capacity and its derivative with respect to time are:

$$f(t) = f_0 - (f_0 - f_\infty)\, e^{-c_E k_+ t} \quad \text{for } i(t) = 0 \tag{3.22}$$

$$\frac{df}{dt} = c_E k_+\Big(f_0 - f(t)\Big) \quad \text{for } i(t) = 0, \tag{3.23}$$

with k_+ as the regeneration constant and a correction factor $c_E = e(t)/\overline{E_d}$, where $\overline{E_d}$ is the mean daily evaporation, i.e. $\overline{E}_y/365$.

In case $0 < f(t) < i(t)$, (3.20) is not valid as the soil uptake capacity is larger than the water present. In this case, KOSIM divides each timestep into a total infiltration part and a regeneration part as during dry weather. In KOSIM-WEST, a linear superposition of (3.21) and (3.23) with a weighting function depending on the amount of rain at time t is used:

$$\begin{aligned}\frac{df}{dt} = {} & \frac{i(t)}{f(t)}\Big[-k_-\Big(f(t) - f_\infty\Big)\Big] + \\ & \Big(1 - \frac{i(t)}{f(t)}\Big)\Big[c_h k_+\Big(f_0 - f(t)\Big)\Big] \quad \text{for } 0 < i(t) < f(t).\end{aligned} \tag{3.24}$$

All this has been implemented under the form:

$$\begin{aligned}\frac{df}{dt} = {} & \min\left(1, \frac{i(t)}{f(t)}\right)\Big[-k_-\Big(f(t) - f_\infty\Big)\Big] + \\ & \left(1 - \min\left(1, \frac{i(t)}{f(t)}\right)\right)\Big[c_h k_+\Big(f_0 - f(t)\Big)\Big] \quad \text{for all } i(t).\end{aligned} \tag{3.25}$$

Once the infiltration capacity is exploited, depressions begin to fill as for the impervious areas and mathematical formulaton of these processes is the same as for impervious surfaces. Regeneration of depression storage and wetting capacities is modelled in the same manner as for impervious surfaces, except that parameter values are different due to different soil characteristics. In KOSIM, the emptying of the depressions during dry periods can happen at the same time through evaporation and through infiltration, whereas, for simplicity, in WEST® this only happens through evaporation.

Figure 3.10 illustrates the different losses from a total bloc rain falling onto a pervious surface to the final water runoff entering the sewer system.

3.3.1.3 Non-water components

Modelled pollutants in KOSIM-WEST are soluble and particulate COD, TN, TP, ammonia and orthophosphates; other components can however easily be added to the model base. The variables were felt useful in an integrated modelling context. For simplicity and minimisation of

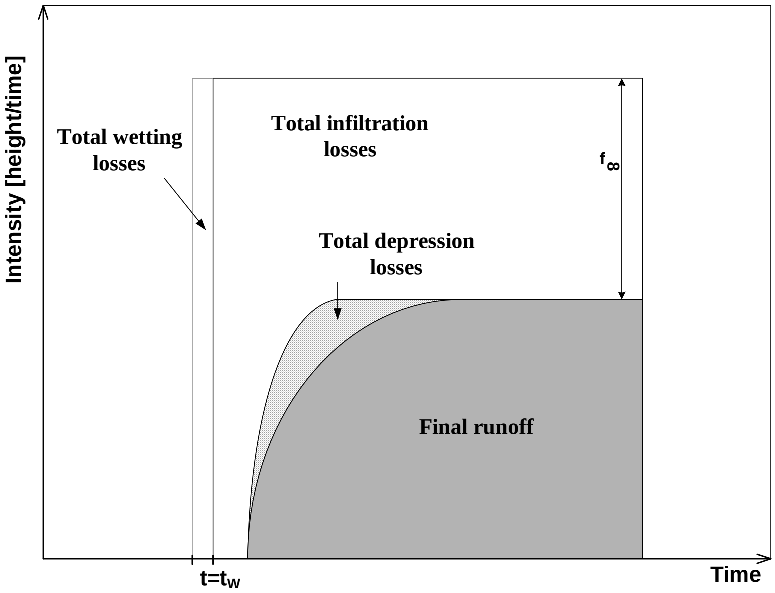


Figure 3.10: Runoff from pervious surface resulting from a bloc rain event. In this case $\Psi_{max} = 1$.

the number of parameters, components are supposed to run off from impervious surfaces only and the simplest modelling approach to calculate the flux $F_n(t)$ of a pollutant n at time t is represented by the equation:

$$F_n(t) = \overline{C_n} \cdot Q(t), \tag{3.26}$$

where $\overline{C_n}$ is the mean pollutant concentration given by the user and $Q(t)$ the calculated water runoff.

Included in the research version of KOSIM (Paulsen (1987)), although omitted in the KOSIM-XL version 6.2, it was also felt useful to give the option to the user to account for accumulation and wash-off of particulate matter. The amount of accumulation depends on many factors like urbanisation, traffic, street nature, particle size etc. and on the duration of the antecedent dry weather period. In this case we used linear accumulation, so that during dry weather periods, the change in accumulated mass Ma is

$$\frac{dMa}{dt} = \phi \cdot A_{imp} \quad \text{when } i(t) = 0, \tag{3.27}$$

where ϕ is the accumulation rate of solids and A_{imp} is the impervious area.

Wash-off during rain events involves a series of parameters: rainfall intensity, height and duration, particle characteristics, type and condition of the street's surface and others. It is here modelled using an exponential relationship expressed by the differential equation:

$$\frac{dMa}{dt} = -k_e \cdot Ma(t) \cdot i(t) \quad \text{for } i(t) > 0, \tag{3.28}$$

with k_e the wash-off coefficient and i the rainfall intensity (Alley & Smith (1981)). A comprehensive review on modelling of accumulation and wash-off of particulates can be found in Bertrand-Krajewski *et al.* (1993) or Ashley *et al.* (2004).

3.3.1.4 Runoff concentrations

After the water losses described above, the remaining water has to flow over the surface and through the local sewer network until it enters the main collectors. Hence, the model will have to simulate time translation and retention of peaks and this routing is lumped into a tank cascade model where the outflow of tank $n-1$ is the inflow of tank n (see Figure 3.3 and for example Engel (1994)). Each tank is modelled as a linear reservoir described by the system of equations (3.3) and (3.4) presented above, composed of the continuity equation and a storage equation, expressing mass conservation and supposing proportionnality between what is contained inside and the outflow. The *concentration time* t_c is the time the rainfall needs in order to travel from the remotest place in the catchment to the end of the local sewer system. It will be determined by the relative catchment characteristics and is represented by the parameter k, which can be understood as the residence time of the water inside one tank. From the KOSIM manual (ITWH (2000)), it has been retained in KOSIM-WEST that $n = 3$ has often proved to be a good choice and k will be in the order of magnitude of $t_c = n \cdot k$.

3.3.1.5 Dry Weather Flow (DWF)

Besides a mean daily quantity $\overline{Q}_{PE}$ of wastewater produced per population equivalent (PE), the amount and composition of wastewater to be transported depends on the number of inhabitants living in the catchment, the time of the day, and on the kind of catchment it comes from (domestic, industrial, commercial...). Based on hourly contribution factors taken from KOSIM ITWH (2000), a WEST® generator uses interpolation between the hourly values to calculate wastewater flows and pollutant charges. Figure 3.11 shows such readiliy available pattern for different population numbers, but hourly factors can be modified easily for a given case study. Flow and pollution are independent of each other and can have different patterns.

The model also takes into account lower week-end flows and polution by multiplying them by some factor between 0 and 1, to be fixed by the user. Additionally, the user can define a similar relative factor for tourism, i.e. higher activity periods, together with start and end days of the year.

3.3.1.6 Imported Water

The quantity of infiltration into the sewer is individual for each sewer system. In the model it is assumed that the intruding water is unpolluted and its amount is entered as a mean flow $\overline{i_s}$ per total connected area. Factorisation by a yearly pattern, that can be calibrated according to the case study, reflects that infiltration is often higher in winter than it is in summer. Such infiltration pattern can also be defined by the user (e.g. in section 4.1.1.3 of Chapter 4).

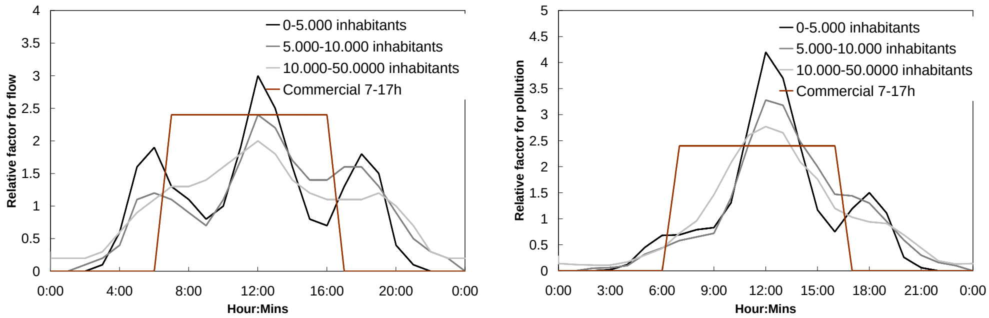


Figure 3.11: Flow (left) and pollution (right) pattern from the KOSIM library and available in KOSIM-WEST. A new user-defined pattern can be entered into the model base.

3.3.2 Sewer transport

3.3.2.1 Pipes

As explained in section 3.2, pipe flow is modelled as a linear tank cascade where each tank is supposed to be under steady flow. Hence, each tank is represented by the equations (3.3) and (3.4). To determine the residence time k in a tank and the number of tanks n needed for a pipe of length L, the Kalinin-Miljukov method is applied. The latter was originally developped for open channel flow (Euler (1973)) and further developed and approximated for application to partially filled circular pipes (Euler (1983)). The pipe of length L is divided into a number of tanks $n = Integer(\frac{L}{L_c})$, with characteristic length

$$L_c = 0.4 \cdot \frac{d}{s}. \tag{3.29}$$

d is the diameter of the pipe and s is the slope. The corrected specific length of the individual pipe stretches is then given by $L^* = \frac{L}{n}$. To find the linear reservoir constant k of one tank, we use

$$k = 0.64 \cdot L^* \cdot \frac{d^2}{Q_{max}}, \tag{3.30}$$

where Q_{max} is the maximum discharge of a pipe and is evaluated using the Colebrook-White equation (e.g. Hager (1999))

$$Q_{max} = a \left[-2 \cdot \log \left(\frac{2.51 \cdot \nu}{d\sqrt{2gds}} + \frac{k_s}{3.71d} \right) \cdot \sqrt{2gds} \right]. \tag{3.31}$$

Here, a is the cross-sectional area, g is gravity, ν the kinematic viscosity and k_s the pipe roughness.

One should note that this model cannot represent backwater effects and in the following section, a model is presented allowing to fix maximum flows according to available measurements or hydrodynamic simulation results to improve the overall behaviour. In the WEST® configuration environment, the user is given a choice between an individual tank model up to a 10 tanks-in-series model. An Excel file serves to calculate n and k, and in WEST® , the ODE's (3.3) and (3.4) will be solved for each tank.

Backflow model *Backwater effects* can occur because of high levels of water at the sewer outlet, for instance in the river or due to tide, because of a decrease of discharge capacity from one trunk to the next trunk downstream (a slope decrease or a roughness increase) or because of the presence of an obstructing structure downstream. Underdimensioned pipes or the presence of sediments can be another reason, so that the maximum outflow capacity of a pipe is not sufficient to conduct all the flow downstream (see a.o. Motiee *et al.* (1997), Engel (1994)) and water gets stored upstream, i.e. in the adverse direction of flow, thereby increasing head which in itself leads to increasing flow. In the situation that no more room is available for increased flow, we witness flooding. Especially in flat sewer systems such backwater effects occur frequently and can cause a CSO device upstream to overflow. As the volume of water and pollution loads into the river are vital factors for the impact assessment of the urban wastewater system on the receiving water, situations of backwater effects were estimated to be necessary to be modelled.

Hydrological models can however not simulate these phenomena and it seems impossible to find a general function that could describe them (Sartor (1999)). Indeed, the actual CSO volume, frequency and intensity can vary strongly from hydrologic simulation results, especially as they tend to overestimate flow maxima. More or less complicated attempts were found that try to account for backwater effects (Engel (1994), Mehler & Ostrowski (1996)). One possibility is to fix the maximum pipe flow to Q_{max} calculated from Colebrook-White equation (3.31) and send any excess water to a ficticious storage tank, which gets emptied once the flow goes below Q_{max}. Another idea is the elaboration of $Q - h$ relationships in the affected locations through measurements or hydrodynamic simulations, although this does not completely solve the problem as they always remain dependent on the event and the neighbouring devices. Another proposed possibility is an extension of the Kalinin-Miljukov method by introducing a backwater constant $k_{bw} > k$ that is used once $Q > Q_{max}$. An improvement of the ficticious storage tank method mentionned above is similar and consists of allowing for higher Q due to the head increase upstream.

What is important in modelling an IUWS is not to loose the advantages of fast calculations compared to hydrodynamic simulation times. The here described conceptual backflow model was developed in WEST® to achieve more realistic behaviour in case backwater effects are relevant and related CSOs become more frequent. In the here proposed model, fictitious backflows have been added to the collector units of KOSIM-WEST and is a combiner-splitter combination (see Figure 3.12): the combiner sums the water coming from the upstream pipe and from the downstream backflow whereas the splitter, according to a given maximum flow Q_{back} sends any excess water back to the upstream combiner and so forth. They allow for overall behaviour of the sewer model to be closer to reality, even though they will not respect the increase of Q_{back} with increasing head upstream, and provides the modeller with an additional calibration parameter for flows in the sewer.

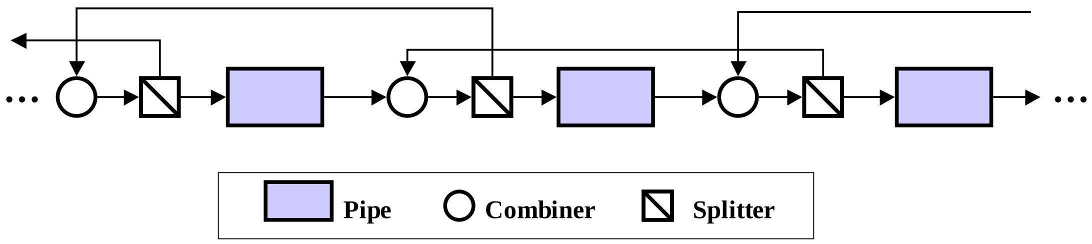


Figure 3.12: Backflow model implemented into KOSIM-WEST.

To calibrate Q_{back}, flow data collected within the sewer network or data from hydrodynamic simulations are needed. For the integrated case study model (see Chapter 5), the method using simulations in InfoWorks^{TM} CS (Wallingford Software, UK) is applied and illustrated. As we are dealing with a tank cascade in which pollutant concentrations are considered completely mixed, the pollutants are sent back with the same concentration as occurring inside the considered tank.

Sediment transport in sewers As for surface accumulation and wash-off, sediment deposition and resuspension can be important elements to be modelled (e.g. low slopes, low flows, etc.). Models range from complex to simple, but often data are scarce to calibrate the models. Hence parsimonious models are to be preferred.

The model implemented into WEST® was initially proposed by Bechmann *et al.* (1999) and was reformulated in Willems (2004). It simulates exponential behaviour of deposited particles and differentiates between 2 different flow conditions: the regime where flow is below the maximum DWF, $Q_{DWF_{max}}$, and the regime where it is above this flow.

In DWF conditions, when $Q_{in} < Q_{DWF_{max}}$, the model simulates the deposition of particulate matter according to the rate κ and constant b_1. The evolution in time of the deposited matter s is then given by:

$$\frac{ds}{dt} = \frac{1}{\kappa}(\overline{s} - s) + b_1(Q_{DWF_{max}} - Q_{in}), \tag{3.32}$$

where $\overline{s}$ being the mean mass of the considered particulate matter in the sewer. Equation (3.32) has two contributions that determine the amount of sedimentation; one is related to the deposit already present and the other depends on the inflow Q_{in} to the pipe. In wet weather conditions, the model equation remains the same with new parameter k^* and b_2:

$$\frac{ds}{st} = \frac{1}{k^*}(\overline{s} - s) + b_2(s)(Q_{DWF_{max}} - Q_{in}) \tag{3.33}$$

with

$$b_2(s) = b_{max}(1 - e^{-\frac{s}{k^*}}) \tag{3.34}$$

Hence, the more material there is available the more violent the first flush effect will be and b_{max} is the maximum rate at maximum deposit s and k^* is a parameter reflecting how fast b_2 increases with s.

3.3.2.2 Combiners and splitters

Combiners and splitters are models without volume. The combiner adds flows and pollutant fluxes coming from separate structures.

A choice of two splitter models is given to the user: the first one separates the flow into a set flow to be entered by the user with the remaining flow leaving the splitter through the other side (absolute splitter), whereas the second splitter divides the flow into two fractions according to a given flow fraction parameter (relative splitter).

A special kind of splitter is the combined sewer overflow (CSO) model. The here implemented model has been taken from KOSIM. It splits the flow into two parts Q_{out} and Q_{over} when Q_{in} reaches a certain critical value Q_{crit}, i.e. a part of the water stays in the sewer system while the leftover is transferred over the overflow weir into the receiving water. It accounts for an increase of Q_{out} with increasing Q_{in} using a linear correction factor $\delta = \frac{Q_{out}(Q_{in}=5\cdot Q_{crit})}{Q_{crit}}$, so that

$$Q_{out} = \frac{\delta - 1}{4} \cdot Q_{in} + \frac{5-\delta}{4} \cdot Q_{crit} \quad \text{if } Q_{in} \geq Q_{crit}. \tag{3.35}$$

The idea behind the model is illustrated in Figure 3.13; if the splitting correction factor $\delta = 1$, then the Q_{in} term is no longer taken into account.

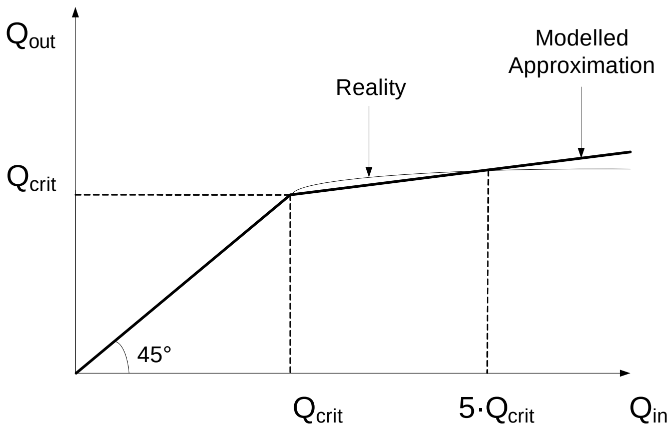
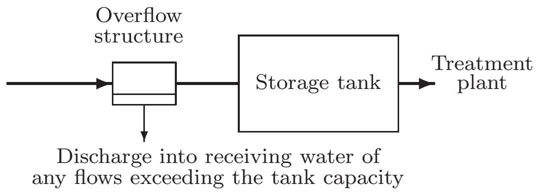


Figure 3.13: Illustration of modelled versus real flow from a CSO.

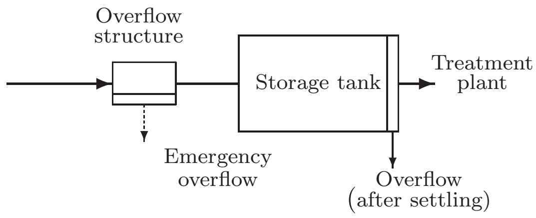
3.3.2.3 Storage tanks

Combined sewer overflow tanks (CSOTs) are installed to reduce CSO volumes, intensities and frequencies, and/or to serve as primary treatment for removal of particulates. Two different kinds of tanks (Figures 3.14(a) and 3.14(b)) can be chosen inside KOSIM (ITWH (2000)) and each of them can be placed in-line or off-line inside the sewer network (also see ATV (1992), Schütze *et al.* (2002)).

In case of in-line placement, all wastewater flow is automatically passing through the tank,



(a) By-pass tank: Stormwater tank retaining the first flush of stormwater with immediate overflow when full



(b) Pass-through tank: Stormwater tank with overflow for settled combined sewage

Figure 3.14: Two kinds of stormwater tanks (ATV (1992)).

whereas for the off-line placement, only excess water enters the tank (i.e. during rain events) and it is emptied when the flow capacities to the WWTP allow for it. The *by-pass* tank (BPT) in Figure 3.14(a) is found primarily in small sewer networks where the flow time of the wastewater is below 15-20 minutes. It is meant to retain first flushes containing the more heavily polluted water. The overflow device, which lies upstream of the tank so that excess water does NOT pass through the tank, only activates once the tank is full. The *pass-through* tank (PTT) depicted in Figure 3.14(b) are generally built in larger catchments where pollutant concentrations are more distributed over time and first flushes are less important. These tanks are supposed to mechanically treat the sewage through settling of suspended solids and excess water will pass through the tank before being discharged.

In the models, the tanks are supposed to be ideally mixed, i.e. any inside flows (hydraulics) are not considered. However, outflow from the tank can be simulated in 3 different ways:

- Q_{out} depends on the water level in the tank and a sluice position at the outlet,
- Q_{out} is fixed by a constant outflow (e.g. in case of a pump),
- Q_{out} is determined from a known $Q-h$ relationship.

For a tank of length l_T, width w_T, depth d_T and volume V_T, with a lateral weir for overflow when the tank is full, the outflow Q_{out} is calculated by using the water level $h(t)$ and the cross-sectional area of the downstream pipe with diameter d:

$$Q_{out}(t) = \begin{cases} \sqrt{2gd_T}\, c_o\, c_P\, d^2 & \text{for } h > d_T \\ \sqrt{2gh(t)}\, c_o\, c_P\, d^2 & \text{for } d < h < d_T \\ \sqrt{2gh(t)}\, c_o\, c_P\, d\; h(t) & \text{for } h \leq d \end{cases}$$

c_P accounts for the shape of the pipe's cross-section ($c_P = \pi/4 = 0.785$ for circular pipe) and c_o is a parameter between 0 and 1, allowing to reduce the cross-sectional area thereby taking into account a sluice position. For overflow calculation from the tank, we use the overflow equation of a rectangular weir (Butler & Davies (2000)), reflecting energy conservation:

$$Q_{over}(t) = \alpha[H(t)]^{\frac{3}{2}} \quad \text{with} \quad \alpha = \frac{2}{3}\, c_d\, w_w \sqrt{2g}. \tag{3.36}$$

$H(t)$ is the water head above the weir crest, w_w is the weir width and c_d is the discharge coefficient of the weir depending on the weir's geometry. Its value lies beween 0.6 and 0.7. In case $Q - h$ relationships for a tank are known, they can also be implemented into the KOSIM-WEST® model base.

As the exact KOSIM models for BPT and PTT were not known to the author, two models have been implemented into WEST® as they will be needed for the subseuent case study: A first flush tank discharging incoming water when the tank is full and a stormwater tank with sedimentation. Sedimentation of particulates is taken into account by a sedimentation factor f_s situated between 0 and 1. The sedimented particles will be flushed from the tank at the end of the event when the volume of water goes below a certain threshold, thereby representing flushing gates. Figure 3.15 shows pollution fluxes out of WEST® and KOSIM tanks to illustrate their behaviour and it can be seen that flushing gates are also used in the KOSIM sedimentation model. As basin structures are rather site-specific, the WEST® open model base allows for easy implementation of new models and an off-line PPT model is tested for a real case study in section 3.3.4.

3.3.2.4 Pump systems

This model is a special version of the PTT without sedimentation described above, i.e. a pump system with a volume. It asks the user to define a pumping flow rate Q_{pump} with given set volume points V_{start} and V_{stop} that indicate when to start and when to stop the pumps.

3.3.3 Comparison of KOSIM-WEST with KOSIM

3.3.3.1 An example

The main submodels (catchment, pipe, basin) created in WEST® were evaluated with respect to their reliability in comparision with KOSIM. For a simple example (see Figure 3.16), simulation outcomes for flow and pollution from both softwares were compared. It should be noted that accumulation and washoff on surfaces, sedimentation and resuspension in the collector as well

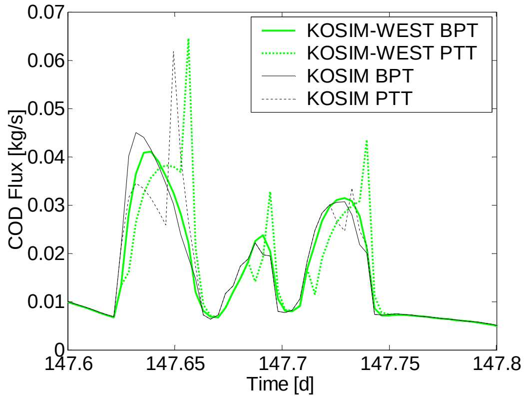


Figure 3.15: Pollution flux to WWTP out of stormwater tanks to illustrate pollutants behaviour in WEST® and KOSIM stormwater tanks.

as backwater effects are not included in the test case as these features were not available in the KOSIM-XL 6.2 version.

The hypothetical system consists of a 40 ha catchment with 4000 PE, a 300 m^3 stormwater pass-through tank with overflow and 25% sedimentation, and, a 700 m long collector. Infiltration was taken to be 0.05 l/s/ha and default parameters for the catchment model can be found in Appendix A. The rain input data is taken from KOSIM examples and input parameters needed are the same in both softwares thanks to the fact that the model concepts are identical. Evaporation was left as shown in Figures 3.7 and 3.8, no seasonal variation of infiltration and no week-end low flows were taken into account for this example.

Outflows and pollutant fluxes from a one year simulation were compared for all submodels. Figures 3.17 and 3.18 show simulation results after the catchment model. Curves for the catchment outflow overlap well and a plot of the absolute differences shows that the latter increase with increasing variations, which is supposed to be due to the different solving methods of the softwares: fixed time steps for KOSIM against varying time steps for numerical solving of ODEs in WEST (smaller time steps are used when important dynamics occur). The mass balances were verified (see Table 3.2), and are in a range of 0.1% over one year, and momentary mass unbalances are due to differences in peak amplitudes.

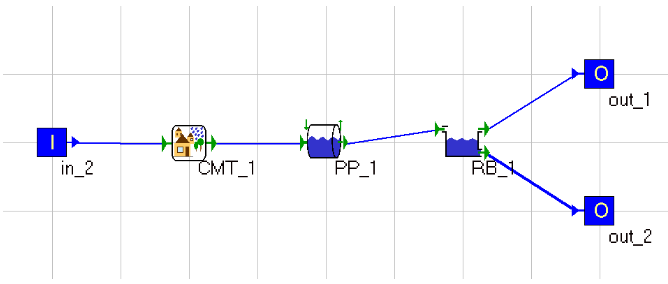


Figure 3.16: Pipe and basin in WEST®

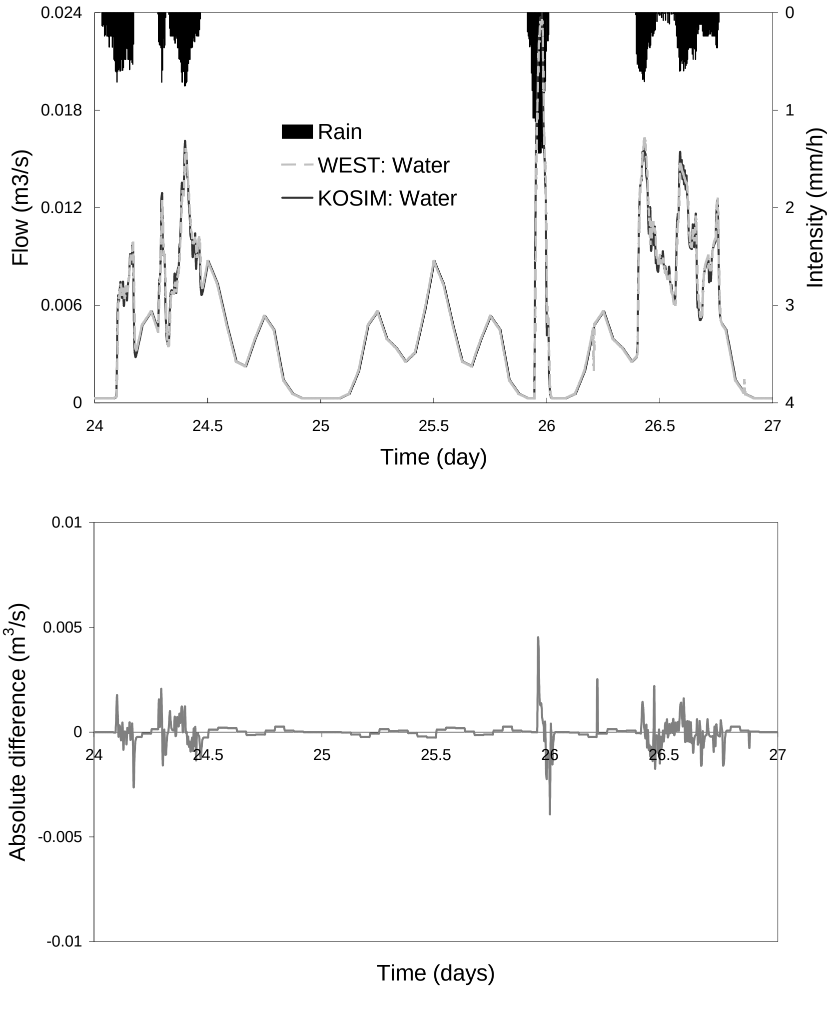


Figure 3.17: Comparison of KOSIM-WEST simulation results with KOSIM results for flow from the catchment submodel.

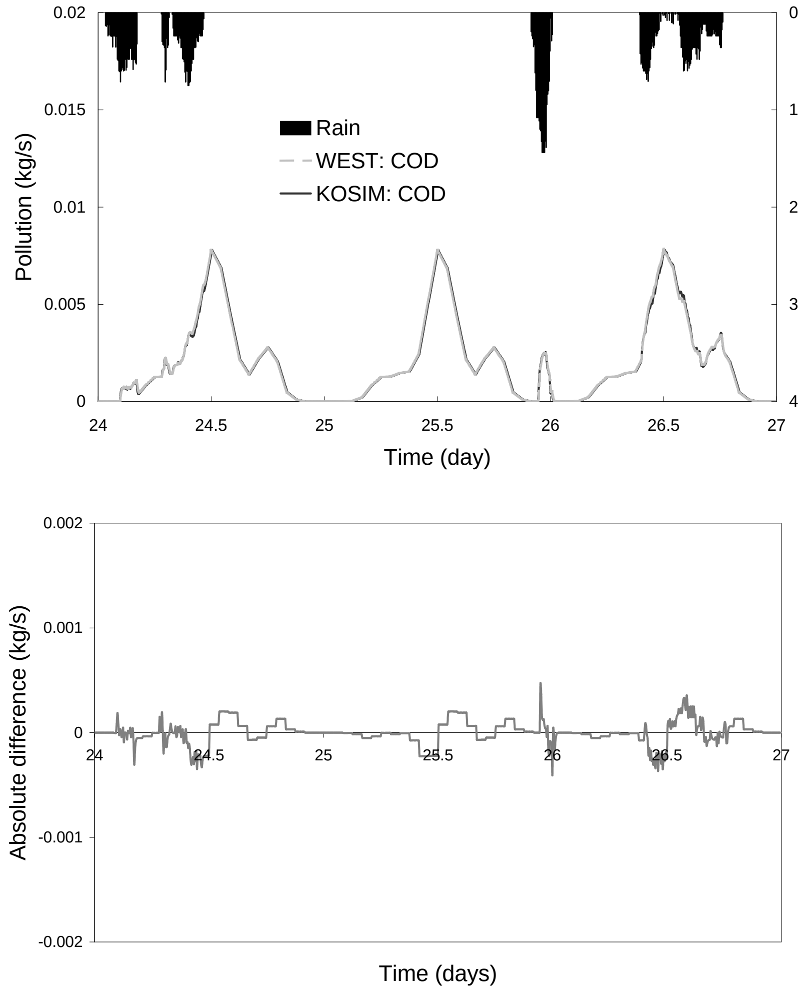


Figure 3.18: Comparison of KOSIM-WEST simulation results with KOSIM results for COD from the catchment submodel.

Table 3.2: Mass ratios for KOSIM-WEST and KOSIM after one year of simulation.

1-year simulation (KOSIM-WEST/KOSIM) —	
Flow	1.0017
COD	0.9998

3.3.3.2 Discussion: Similarities and differences

Although the concepts behind both KOSIM and KOSIM-WEST models are identical, their specific features and their general applicability differ. The implementation of KOSIM-WEST has quite different objectives than the development of the original KOSIM software. As already mentionned before, KOSIM is used for dimensioning of rainwater management infrastuctures according to the German regulation guidelines such as ATV A-128 (ATV (1992)) or the new ATV A-198 (ATV-DVWK (2003)).

Some of the features implemented into KOSIM-WEST (e.g. accumulation & wash-off of particulates, sedimentation in pipes, approximation for backwater effects) are not available in KOSIM version 6.2. However, in the new KOSIM version 7.1, the developers added the possibility to model first flushes and implemented static storage in pipes to account for backwater effects. Indeed KOSIM remains a widely used tool for engineers. Table 3.3 tries to highlight the main differences and hence possible application domains of KOSIM and KOSIM-WEST.

Table 3.3: Applicability domains for KOSIM and KOSIM-WEST.

	KOSIM-WEST		KOSIM
+	model accessibility in the model-base	+	immediate calculations done according to ATV guidelines
+	models can be extended or added	−	modifications to the model base not possible
+	simulation information possible on all variables	−	information on flow and pollutant fluxes only
	⇒**analysis** of existing structures **research**		⇒**dimensioning** (German guidelines) **planning**
−	longer calculations due to adaptive time steps	+	very short calculation times
+	contained in the same software as models for WWTP and river	+	can be coupled to GESIM software (ITWH, Germany) to evaluate total emissions from sewer and WWTP
+	control options available in WEST®		
	⇒integrated simulations **⇒control strategies**		**⇒ long-term simulations** **⇒ high number of simulations**

First of all, interception of information on for example depression losses during runoff is possible in KOSIM-WEST, whereas in KOSIM only flows and pollutant loads from CSOs or tanks can be accessed by the user. Also, the possibility for addition of new models for special processes or structures is a very useful characteristic of WEST®. It allows to modify models according to user needs and allows future implementation of a water quality model that takes into account biochemical transformations in the sewer. However, the computational speed of KOSIM is considerably higher than the one of KOSIM-WEST of an order of 10, and is therefore suited for the long-term assessment of a sewer system alone. Moreover, KOSIM allows for quick generation of reports and yearly evaluation of the system using the German ATV guidelines and is ideal for all kind of planning of storage volumes or other rainwater infrastructures. However, for an integrated study, the outweighing advantage of KOSIM-WEST is that it is contained in the same software as models for WWTP and river systems. It can easily be connected to these models so that data transfer is easy from one submodel to another. This can especially be useful when developing integrated real-time control strategies for sewer and WWTP.

3.3.4 KOSIM-WEST: The 'Bonnevoie' catchment

3.3.4.1 Aim of the modelling exercise

This work was performed within a project on 'Material Flows in the Catchment Area of the River Alzette: Impacts of Contaminants on the Water Resources Quality (Micro/Macropollutants and Nutrients)', funded by the Fonds National de la Recherche (FNR), Luxembourg. The aim of the project is to evaluate loads for nutrients, suspended solids (SS), total organic carbon (TOC), metals and polycyclic aromatic compounds (PAC) into the river Alzette, to estimate event mean concentrations and to look at first flushes both in the sewer and in the river. During several months in 2005, a number of rain events have been monitored in the Bonnevoie catchment, one of the multiple subcatchments of the river basin. Combined sewage flows into a storage tank and measurements were taken in terms of flows, pipe and storage levels, as well as a certain number of pollution variables (a.o. SS, TOC, ammonium, orthophosphate, metals, ...).

As no data was going to be available at CSOs within the integrated 'Bleesbruck' case study, the idea behind the following modelling and simulation exercise was to test the KOSIM-WEST tool and see how the here available measured data can be used. The aim was to gain some experience with the tool and, through the possible discrepancies between measurement and simulation data, to find out what the data requirements for the calibration of such an urban catchment model are. Default parameters as presented in Appendix A are used and results are presented without calibration.

3.3.4.2 The Bonnevoie model

The 'Bonnevoie' sewer catchment, situated within Luxembourg city, has a combined sewer network and a drainage area of 245 ha. The population is estimated to be around 13000 and without any overflows, the combined sewage flows via a storage pipe of diameter 2000 mm and length 130 m into a storage tank of volume 3500 m^3, before it is treated at the WWTP 'Bonnevoie'.

The model construction required several steps, including subdivision of the area according to the ramification geometry of the network, and determination of the drained areas and their

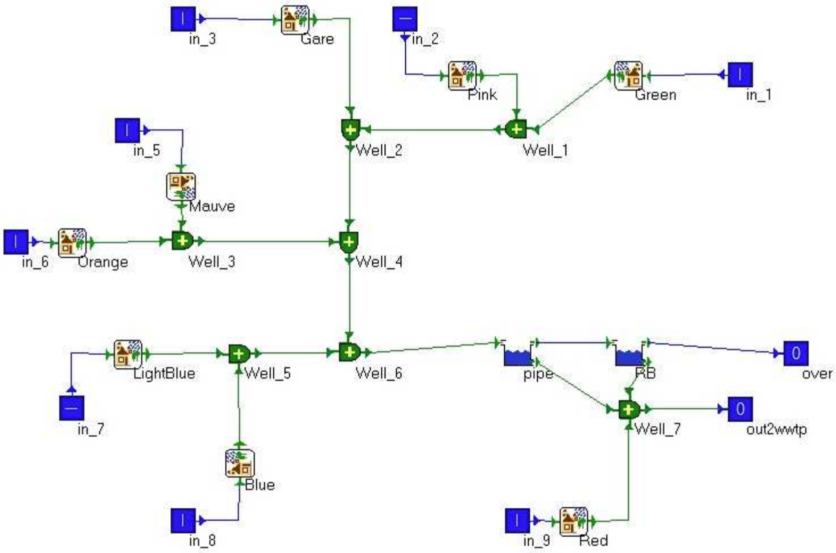


Figure 3.19: Model configuration of the Bonnevoie catchment in WEST®.

respective degree of imperviousness. AutoCAD sewer data were available and inspected, and Bonnevoie was subdivided into 8 subcatchments as depicted in the model configuration in Figure 3.19.

To determine the degree of imperviousness of contributing surfaces for rainwater flow, a land use map was used to identify roofs, parking lots and streets which were supposed to be completely impervious, giving 28 % of impervious surfaces (Figure 3.20). Previous visual inspection and estimations from AutoCAD surface maps gave approximately the same results.

The storage pipe is modelled as a stormwater tank, overflowing into the storage tank once the water level reaches 2m. Inputs to the model are rain data from a gauging station about 3km away, measuring rain depth in 10 minute intervals.

3.3.4.3 Simulation results

Simulations were performed for an interval of 175 days, from April to November 2005, for which data was available.

First of all, simulation results where assessed with respect to water quantity. Figure 3.21 shows measured and simulated flow inside the storage pipe, and measured and simulated levels in the stormwater tank. DWF comparison turned out to be difficult due to the irregularity in the measured flow. The linear increase in flow over weeks and subsequent sudden drops are due to sediment and material accumulation from construction sites and cleaning in the sewer system. This accumulation provoked erroneous flow measurements due to the inability of the flowmeter to measure at such large water heights. It was also difficult to discern a specific daily pattern from the data, so that the simulated pattern was left as default (see Figure 3.11). The very low measured flows in case of low water heights led to the conclusion that infiltration is minimal so

Figure 3.20: Bonnevoie contributing catchment surface with impervious (red) and pervious (grey) surfaces.

that it was left zero in the model. This plausible as the catchment is situated in the Luxembourg Sandstone and no groundwater is present at these heights of the sewer system.

The stormwater tank level was estimated to be the best variable to evaluate wet weather water quantities, as it is directly linked to the water volume in the tank and therefore also to eventual overflows to the river Alzette. Closed mass balances were considered to be important, and can be quantified by the *bias* measure

$$B = \frac{\overline{S}}{\overline{M}}, \tag{3.37}$$

where $\overline{S}$ and $\overline{M}$ are the mean values of the simulated and the measured data, respectively. The value obtained was 0.983. Wet weather peaks in the storage tank as well as the storage pipe level have been inspected to evaluate whether both the start of an event and the peak amplitude coincide with the simulated ones. Although water mass balances in the tank proved to be good, comparisons show that some of the measured and simulated stormwater peaks have time shifts or do not even appear at all either in measurements or in the simulation data (see Figure 3.21). This is probably due to spatial variablity of rain and shows that especially for representation of a catchment in a model, rain data needs to be recorded within the catchment or to be interpolated with other measurement stations. In this case, the measurement station is considered to be too far away from the simulated Bonnevoie catchment.

To evaluate the quality of pollutant simulations, COD and ammonium were chosen as variables, i.e. a particulate and a dissolved component. As COD was not directly measured, total organic carbon (TOC) was used as a correlated variable. Although the specific correlation was

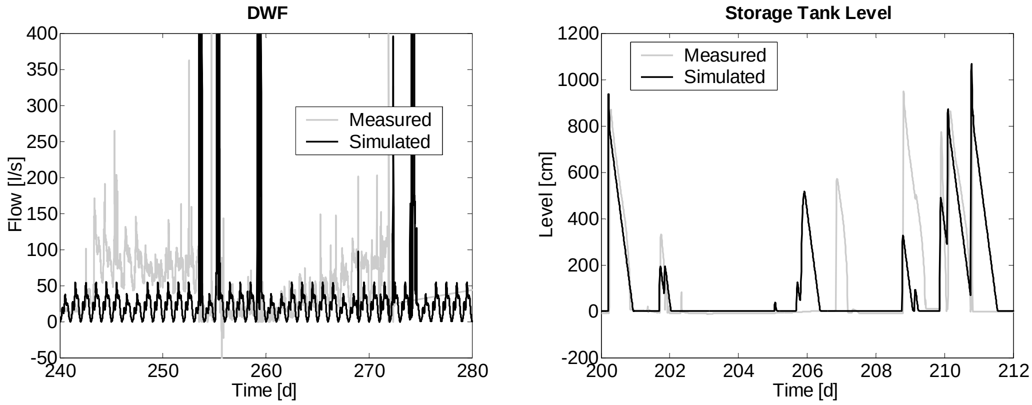


Figure 3.21: Measured and simulated dry weather flow in the storage pipe (left), measured and simulated storage tank levels (right).

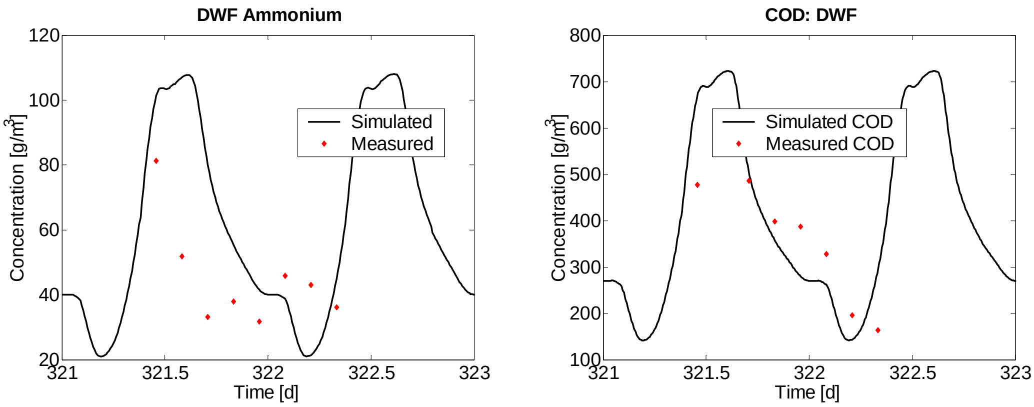


Figure 3.22: Measured and simulated DWF pollution variables: ammonium and COD (measured TOC is transformed using COD=3xTOC).

not measured at the time, the generally accepted factor of 3 gCOD/gTOC is used in all the Figures. Only one day of DWF pollution data was available to calibrate the DWF conditions and a comparison is shown in Figure 3.22. From these data, i.e. one day of 8 composite samples, no pattern can be determined and a longer data set and more frequent data is necessary. Also, the order of magnitude of the concentrations cannot be further calibrated, as one day might not be representative. Therefore the default input loads were left as such.

For the comparison of simulated and measured wet weather pollution, 4 rain events have been selected (see Figures 3.23 and 3.24). For Events 2 and 8, the simulated start of flow increase corresponds with the measured starting time, but the quantities in the pipe do not correspond. Event 4 contains several consecutive events suggesting that a first flush phenomenon should not be observed during the event for which pollution was analysed. Event 6 simulates too much water compared to measured values.

Looking at ammonium, it is observed that the simulated concentrations drop when flows rise and that they correspond to the order of magnitude of the measured concentrations. Note however that dry weather pollution could not be calibrated, and especially ammonium has a quite high contribution within combined sewage, making DWF calibration necessary. As no measurements hasve been taken prior to the event, no conclusions can be drawn regarding the amount of dilution taking place, i.e. about the respective contributions of ammonium from dry and wet weather flow. The same is true for COD (although compared to TOC here, which is related to the COD variable).

3.3.4.4 Conclusions

With a good estimation of pervious and impervious contributing surfaces, simulations seemed to reasonably represent quantities of wet weather water masses using default values for the catchment runoff model. This is true supposing that the same amount of rain is provided by the data as was actually taking place in the catchment. However, although some of the measured and simulated amounts and peaks inside the storage tank coincided well, times of rain events often did not correspond. This is due to spatial variability of rain and suggests that for modelling and simulation, rain data needs to be registered locally or interpolated in case single events are of importance to the modelling exercise. Especially when pollution is to be represented within the model, we need more accurate results on start of the event and the flows.

The results for pollution suggest that before evaluation of wet weather pollution concentrations, dry weather mean concentrations and its daily pattern are important to be calibrated. Online probes to continuously monitor before and during the event are necessary to calibrate a model on the pollutant concentrations in dry and wet weather, and to assess whether a first flush effect exists in the catchment.

It should be noted that direct comparison between the Bonnevoie and the Bleesbruck catchment is not considerated appropriate, for several reasons. Bonnevoie is a fairly steep catchment compared to the 'Bleesbruck' catchment, so that wet weather pollutant behaviour in the sewer system is certainly different in the two networks. Although only one rain gauge in the centre of the catchment will be used, the Bleesbruck case study is much larger and spatially wider spread so that, averaging out of the spatial variability of rainfall is expected. Overall, this example helped to gain experience on the application of KOSIM-WEST and showed that with reasonable information on contributing surfaces, the model can fairly well represent urban drainage on the long-term as is necessary for the Bleesbruck catchment.

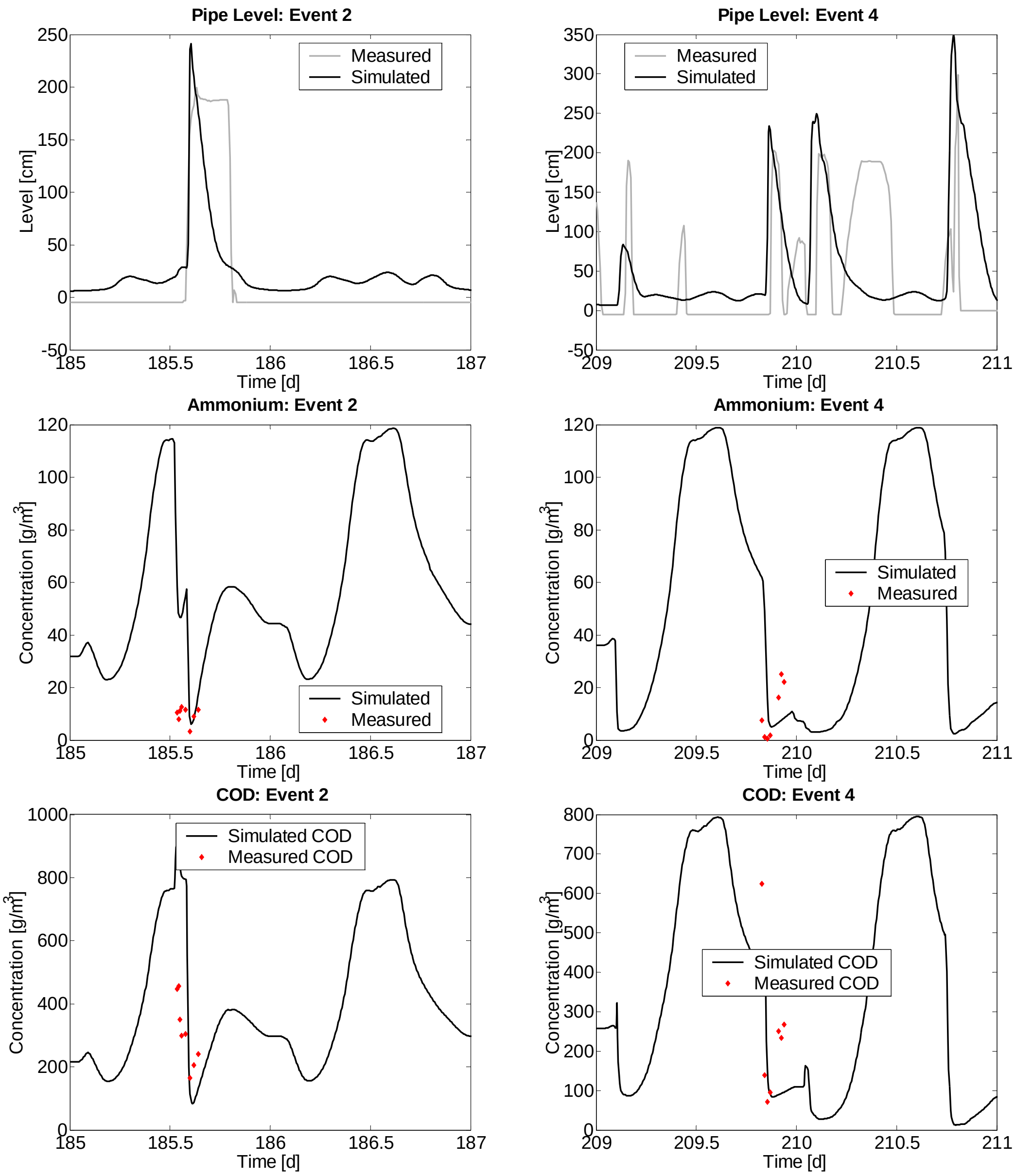


Figure 3.23: Plots of measured and simulated flow (top), ammonium concentrations (middle), TOC/COD concentrations (bottom) for events 2 and 4 in the storage pipe.

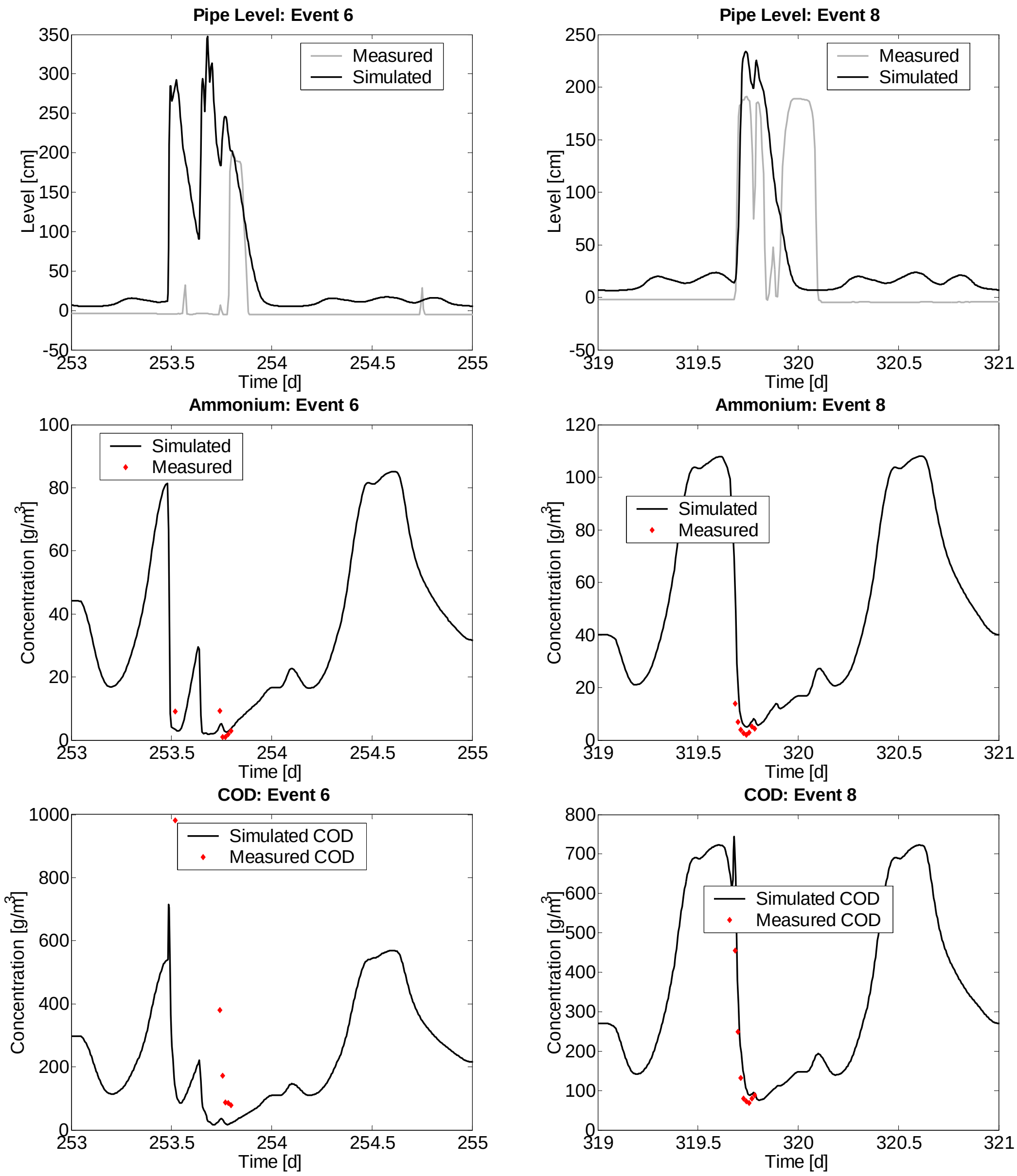


Figure 3.24: Plots of measured and simulated flow (top), ammonium concentrations (middle), TOC/COD concentrations (bottom) for events 6 and 8 in the storage pipe.

3.4 Integrated Modelling with WEST®

3.4.1 Models for WWTP and river

WEST® is mainly used for modelling and simulation of wastewater treatment plants. Besides containing various models for *sensors* and *controllers*, it incorporates different models for *buffer tanks*, *clarifiers* and *activated sludge units*. The overall equation for such a system respecting mass balances is

$$Accumulation = Input - Output + Reaction$$

so that for the component concentration vector $\mathbf{c}(t)$,

$$\frac{d(V(t)\mathbf{c}(t))}{dt} = Q_{in}(t)\mathbf{c}_{in}(t) - Q_{out}(t)\mathbf{c}(t) - V(t)\mathbf{r}(\mathbf{c}(t), \mathbf{p}) \tag{3.38}$$

where $\mathbf{c}_{in}$ is the component concentration vector of the inflow to the system and $\mathbf{r}$ is the conversion rate vector, which is a function of the actual concentrations $\mathbf{c}$ and the model parameters $\mathbf{p}$. Conversions in the activated sludge are modelled with the IWA state of the art models ASM1, ASM2, ASM2d and ASM3 (Henze *et al.* (2000)). As processes are numeruous, visualisation and evaluation of the model is very much simplified by representing them in Petersen matrix format. The biological processes considered (bacterial growth & decay, nitrification, ...) are placed in matrix rows and state variables (bacteria, ammonium, dissolved oxygen, ...) in matrix columns. To illustate this on an example, Table 3.4 depicts the matrix for conversion processes in the Streeter-Phelps model (a.o. Chapra (1997)), which is the simplest but pioneering model in river water quality modelling, describing the increase and following decrease of the oxygen deficit downstream of a source of organic material. The reaction term r_i of the ith component in equation (3.38) can be obtained:

$$r_i = \sum_{j=1}^{m} \nu_{ij}\rho_j \tag{3.39}$$

where ν_{ij} is the stoichiometric coefficient and ρ_j is the kinetic process rate for process j.

The RWQM1 (Reichert *et al.* (2001)) for biochemical transformations in the river was developed for easy integration with the ASM family and is also contained in the WEST® model library. The latter and the temperature dependent ASM2d model will be used within the case study and is therefore presented in more detail in Chapter 5.

Table 3.4: Process kinetics and stochiometry matrix for the Streeter-Phelps model.

Component i	$\rightarrow$	1	2	Process rate
$\downarrow$	Process j	DO	BOD	ρ_j $[ML^{-3}T^{-1}]$
1	Reaeration	1		$K_2(DO_{sat} - DO)$
2	Biodegradation	-1	-1	$K_1 BOD$

K_1 =degradation rate
K_2 =reaeration coefficient
DO_{sat} =Oxygen saturation coefficient

3.4.2 Connector models: The continuity-based interfacing method (CBIM)

To create an integrated sewer-WWTP-river model, submodels need to be interfaced, in this case KOSIM-WEST variables need to be linked to ASM and RWQM variables, as well as ASM to RWQM variables. Problems arise from the following three arguments (Vanrolleghem *et al.* (2005b)):

- some state variables used in one model do not exist in the connected model
- the 'meaning' of a state variable in one system may not hold for the other system (e.g. components can be considered as inert in one system but may be biodegradable in another)
- the elemental composition of a component variable in one model is not identical with the component variable in the connected model.

A connector model, respecting closed mass and elemental balances, was proposed in a case study on the river Lambro (Italy) that links the states of the ASM1 and RQWM1 (Meirlaen *et al.* (2001), Benedetti *et al.* (2004)). The continuity-based interfacing method (CBIM) (Vanrolleghem *et al.* (2005b)), creates a formalised frame on the basis of this work. The main idea of the interfaces is that one constructs a set of algebraic transformation equations on the basis of a Petersen matrix description of the two models to be interfaced (i.e. from origin model P to destination model Q). Through this approach it is possible to maintain the continuity of elements C, H, N, P, O, charge and COD, while the two models remain unaltered. The methodology consists of the following steps (Benedetti (2006)):

1. Formulation of elemental mass fractions and charge density.
2. Set-up of the composition matrix.
3. Definition of the transformation matrix.
4. Implementation of the transformation equations.

3.4.2.1 Formulation of elemental mass fractions and charge density

The main hypothesis in this phase is that the mass of each component k is made up of constant fractions of the elements C, N, O, H and P. The elemental mass fractions α_k^C, α_k^N, α_k^O, α_k^H and α_k^P are given in grams of element per gram of component. For the components for which the elemental composition is known, the calculation of the mass fractions is straightforward. For the components for which the elemental composition is not known, it is necessary to make some assumptions and to use data provided by literature (e.g. Reichert *et al.* (2001)). As a result, for component k,

$$\sum_{All\ E} \alpha_k^E = 1 \quad \text{for } E = \{C, N, O, H, P\}. \tag{3.40}$$

Then, also α_k^{COD} and α_k^{Ch} (Ch stands for charge) can be calculated. The COD equivalent of a component is defined as the grams of oxygen that are consumed during oxidation of a mass unit of the component to NH_4^+, CO_2, H_2O, H^+ and PO_4^{3-}. The COD equivalent of a component k is related to the mass fractions of elements and charge through the relationship (Reichert *et al.* (2001)):

$$\alpha_k^{COD} = 32\frac{\alpha_k^C}{12} + 8\alpha_k^H - 16\frac{\alpha_k^O}{16} - 24\frac{\alpha_k^N}{14} + 40\frac{\alpha_k^P}{31} - 8\alpha_k^{Ch} \tag{3.41}$$

Using the charge Ch_k of a component k and the molecular weight m_k, the charge density is

$$\alpha_k^{Ch} = \frac{Ch_k}{m_k}. \tag{3.42}$$

Such formulation of mass fractions and charge density is done both for the components of the origin and the destination model.

3.4.2.2 Set-up of the composition matrix

All fractions of all components from both origin and destination model can be placed into a matrix $\boldsymbol{\alpha} = \alpha_k^E$, where $k = 1, ..., p, ..., p+q$ where p and q are the number of components in the origin and destination matrices P and Q respectively. To set-up the final *composition matrix* $\boldsymbol{i}$ used to connect the models, $\boldsymbol{\alpha}$ needs further conversion. The ASM, as well as the RWQM model components are expressed in various *stochiometric units* like $M(COD)/L^{-3}$, $M(N)/L^{-3}$, $M(P)/L^{-3}$ or $M(H)/L^{-3}$ depending on the kind of transformations they are submitted to. An element of the composition matrix $\boldsymbol{i}$ is therefore given by:

$$i_k^E = \alpha_k^E \cdot M_k \tag{3.43}$$

where M_k is expressed in grams of component k per gram of stochiometric unit (COD, N,...) and can be calculated using the molecular weights of the stochiometric units. Also,

$$i_k^{COD} = \alpha_k^{COD} \cdot \sum_{All\ E} i_k^E$$

and

$$i_k^{Ch} = \alpha_k^{Ch}$$

.

3.4.2.3 Definition of the transformation matrix

The main concept behind the *transformation matrix* $\boldsymbol{\theta}$ is that the components of the origin model P are transformed *completely* into the variables of the destination model Q. To ensure this, a number of transformations have to be specified. The definition of these equations depends on the knowledge available on the processes. Usually the number of transformations t to be defined is equal to the number of state variables p of the origin model. Each transformation converts a number of components of the origin model to a number of components of the destination model. Every transformation j for component k is characterized by its stoichiometry $\theta_{j,k}$. While stoichiometry coefficients of the origin components are set to an arbitrary value (with negative sign in order to maintain the right direction of the transformation), the coeffcients of the destination state variables are set so that each transformation maintains the COD content. For each transformation j the elemental continuity must be guaranteed, which is easily checked by the equation (where k are the components and E the elements):

$$\sum_{All\ k} \theta_{j,k} \cdot i_{k,E} = 0 \tag{3.44}$$

Each transformation j is also characterized by its *transformation rate* ρ_j which, together with the stoichiometry coefficient, specifies the amount of the component k transformed per unit of time, equal to $\theta_{j,k} \cdot \rho_j$.

3.4.2.4 Implementation of the transformation equations

The set of interface unknowns consists of the stoichiometric coefficients $\nu_{j,k}$ and the transformation rates ρ_j. Together they enable the calculation of the outflux from the destination model. In order to solve the unknowns it is necessary to set up a system of two sets of equations taking into account the fluxes in and out from the interface.

$$\Phi_k^{in} = -\sum_{j=1}^{N} \nu_{j,k} \cdot \rho_j \quad \text{for } k = 1, ..., p \tag{3.45}$$

$$\Phi_k^{out} = \sum_{j=1}^{N} \nu_{j,k} \cdot \rho_j \quad \text{for } k = p+1, ..., p+q \tag{3.46}$$

where Φ_k^{in} is the known positive influx of a component k of the origin model, Φ_k^{out} is the unknown outflux of a component k of the destination model, p is the number of origin state variables and q is the number of destination state variables. It is important to check that all transformation rates ρ_j are positive, in order to assure that the transformations are in the right direction (origin model to destination model). In case this is not verified, the transformation equations should be modified.

Chapter 4

The Case Study

The Chapter first characterises the integrated case study, situated in Luxembourg, by individually describing its 3 subsystems which are the urban catchment, the wastewater treatment plant 'Bleesbruck' and the receiving river system. A further section is devoted to the measurement campaigns conducted within the project. The last section summarises the deficits and pressures of the case study components.

The following chapter is partly developed from and contained in the following article:

Solvi, A.-M., L. Benedetti, V. Vandenberghe, S. Gillé, P. M. Schosseler, A. Weidenhaupt and P. Vanrolleghem (2006). Implementation of an integrated model for optimised urban wastewater management in view of better river water quality. A case study. *IWA World Water Congress and Exhibition*, 11-15 September 2006, Beijing, China.

4.1 Description of the Case Study

4.1.1 The urban catchment

The case study is situated in the lower northern part of Luxembourg (see Figure 4.1), near the town of Diekirch. It consists of a semi-rural sewer catchment drained into one treatment plant and discharging into 3 receiving waters (Attert, Alzette and Sûre) with differing water quality. The main municipalities in that region have put themselves together into the 'Nordstad' association for sustainable development, planning and cooperation. Indeed, the Nordstad is considered to become one of the 3 development poles in Luxembourg, next to Luxembourg City and the city of Esch-sur-Alzette, so that urban drainage is important both for good spatial development and planning of the populated areas, as well as for ecological reason in view of a 'good' river water quality.

The catchment (see Figure 4.1) contains about 20 subcatchments, where the towns of Diekirch and Ettelbruck are the major ones with around 6000 and 7500 inhabitants. The main collector system as well as the treatment plant are maintained and operated by the S.I.D.E.N. (Syndicat Intercommunal Des Eaux résiduaires du Nord).

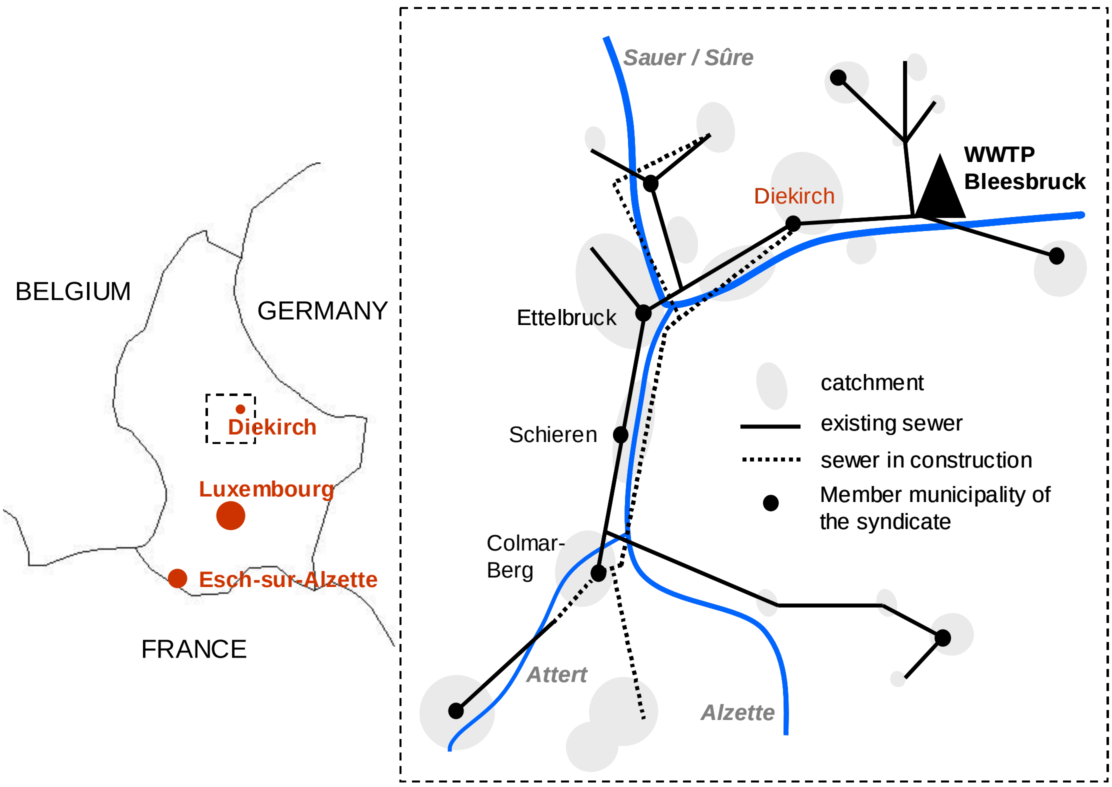


Figure 4.1: Schematic overview of the urban catchment with its WWTP 'Bleesbruck' and its receiving waters, situated around the town of Diekirch, Luxembourg.

4.1.1.1 Climate, soil properties and land use.

The average monthly temperature in Luxembourg, measured over 30 years, is around 9°C and varies between 0 and 18°C in winter and summer respectively. Mean annual precipitation is 862mm.

The soils in the investigation area have developed on a variety of substrates but are in general loamy cambisols with different skeleton content. Karstic Luxembourg and Bicarré sandstone formations play an additional role as important geological formations (Division des eaux souterraines et des eaux potables, Administration de la Gestion de l'Eau, Luxembourg).

The region contains dispersed agglomerations of different sizes, ranging from 100 to a few thousand inhabitants and farming is found throughout the area.

4.1.1.2 Wastewater

More than 90% of the urban catchment is drained as combined sewage, i.e. mixing stormwater and wastewater. In the 1950s, it was decided to have a common wastewater treatment for the towns of Diekirch and Ettelbruck (see Figure 4.1), so that a 6 km main collector was built. Ten years later, the municipalities of Schieren and Colmar-Berg were connected too. During the following decades, the population around the area grew and additional industries developed, all discharging into the above collector. Therefore, in 1999, a study was performed by the engineering office Dahlem, 'Schroeder & Associés' (Luxembourg) to check the hydraulic situation of the whole sewer network and propose renovation measures as well as new management strategies. In that sense it was concluded that capacities of both the local sewer networks as well as of the main collector were insufficient, resulting in frequent combined sewer overflows. To remedy against this situation, it was decided to replace several CSO structures by stormwater tanks and a parallel collector is planned to be built. Some of the retention tanks were completed in Diekirch during 2006 and are taken into account in one of the simulation scenarios in Chapter 6. The main characteristics of the population, the industries and the sewer network are contained in Table 4.1.

Table 4.1: Main catchment characteristics for the existing situation

Drained catchment area	≈ 900ha
Imperviousness	≈ 20%
Population	≈ 25000 inhabitants
Industry	brewery, dairy industry, slaughterhouse, dump site
Sewer network	≈ 60 km (combined, with few exceptions)

The catchment area and sewer network length was compiled from existing engineering maps. The current population was calculated from census data (STATEC (2003)). Main industries are a brewery (Diekirch), a dairy (Erpeldange[1]) and a slaughterhouse (Ettelbruck). Next to these, the S.I.D.E.C. (Syndicat Intercommunal pour la gestion des Déchets) dump site rejects polluted

[1]The current dairy site will be relocated to the south of the urban catchment (Bissen), with a much higher production of dairy products. Hence, the loading to the WWTP will increase and the latter is planned to be upgraded in the near future.

surface waters into the main collector. Near Colmar-Berg, the 'Good-Year' Mold Plant and the 'Arcelor' Wire-Drawing are sending their already treated wastewater. A recently performed inventory in the context of a WWTP extension study done by the engineering office 'Ingenieurbüro Peil GmbH' (Düren, Germany) provided estimated data on population equivalents (PE) from households and industry. Pollution concentrations in industry waters were estimated from literature (WorldBank (1997a,b,c)) and on-site sporadic measurements done by the operator. In Table 4.2, tentative values are given for wastewater production and wastewater pollution. Obtained data show that pollution from industry is very variable both in terms of composition as well as in terms of time, due to different processes going on at different hours of the day and different days of the week.

Table 4.2: Main catchment characteristics for the existing situation. PEs are calculated supposing that: 1PE= 150l/d.

	Brewery	**Dairy**	**Slaughterhouse**	**Good-Year**	**SIDEC**
PE	3000	1800	1600	600	3000
$\mathrm{COD_{tot}}[mg/l]$	2000	1800	2000	1500	12000
$\mathrm{TN}[mg/l]$	50	160			
$\mathrm{TP}[mg/l]$	40	16			
$\mathrm{COD_{sol}}$[mg/l]	500	400	500	200	2000
NH_4[mg/l]	20	100			
PO_4[mg/l]	16	12			

Due to its rural, nature-related character, the region is a popular place for tourism both in hotels and campings, especially during summer months. Nevertheless the Bleesbruck investigated site stays rather unaffected.

4.1.1.3 Infiltration into the sewer system

Infiltration into the sewer system is an often encountered problem as it entails both severe economic as well as ecological consequences. For this case study, infiltration is a well-known fact as reported by the operator and, by visual inspection of the flow data, a rather high baseflow is observed in summer and winter. Origins of infiltration, measures against and effects are discussed under the scenario description in Chapter 6.

Evaluation of the amount of infiltration water is necessary for the construction of the model. Several methods exist (Fuchs *et al.* (2003)), among which the subtraction of drinking water use in the urban catchment from the total dry wither flow (DWF) to the treatment plant or measurement of the nightly minimum flows during dry weather conditions, assuming the water use is close to zero at certain night hours. One has to be careful with these methods in catchments with long flow times, where wastewater arrives at the treatment plant at any time and connected industrial sites additionally lead to nightly wastewater flows. The here applied method is called the *21-days moving minimum method* (Brombach *et al.* (2002)) and uses daily average values of inflow to the WWTP. These are supposed to be composed of the mean sanitary flow plus the infiltration flow. A function will extract the lowest value out of the last 21 days, supposing that over this period of time, there is at least one day with dry weather conditions. The method has

Table 4.3: Monthly sewer infiltration factors for the Bleesbruck catchment (to be multiplied by the mean yearly infiltration).

J	F	M	A	M	J	J	A	S	O	N	D
1.31	1.43	1.25	1.06	0.96	0.77	0.90	0.74	0.64	0.83	0.98	1.13

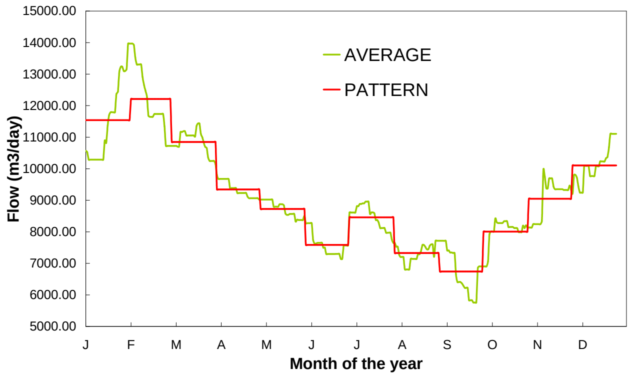


Figure 4.2: Sewer infiltration pattern for the studied catchment. The pattern was deduced from 4 years of hydraulic inflow data to the WWTP.

the advantage of not being limited to singular measurements during dry weather flow days. It should be noted that the here evaluated infiltration is the real infiltration minus the exfiltration from the sewer network.

To quantify the mean infiltration rate per area and its seasonal variability for the Bleesbruck catchment, average daily values were calculated for 4 years of inflow data from 2001 to 2004. By inspecting the varying pattern of the WWTP influent data, mean daily sanitary flow (i.e. domestic plus industrial) was estimated to be around 4000m^3/d. By subtracting this from the daily average inflow value, one obtains a mean daily infiltration flow. The calculated mean infiltration flow over the year for the Bleesbruck catchment becomes 9150m^3/d, i.e. 0.116 l/s/ha of total area with less infiltration in summer/autumn and more infiltration in winter/spring. For easy calibration in WEST® the monthly normalised flow pattern, shown in Figure 4.2, was implemented into the software. This way, the mean infiltration rate per area together with the monthly pattern can be calibrated for the simulation year 2005, which will be used for the subsequent scenario analysis (see Chapter 6).

4.1.2 The Bleesbruck WWTP

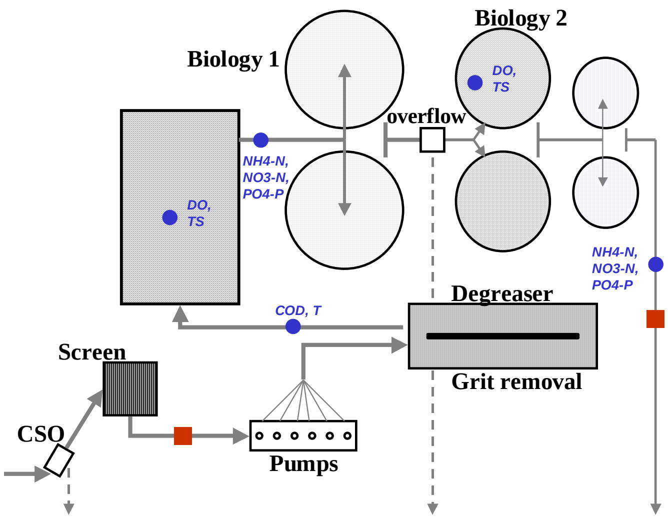
The Bleesbruck treatment plant is a so-called AB-system. It has several components, and in order of wastewater passage they are: screen (8mm), pumps, grit removal/degreaser unit, an activated sludge (AS) tank for high loaded COD removal, 2 clarifiers, overflow, 2 AS volumes for nitrification and another 2 clarifiers (see the aerial photograph in Figure 4.3 and the flow chart in Figure 4.4). Phosphorus removal is achieved by chemical precipitation in the first AS unit. On-site are also 2 centrifuges for sludge dewatering and 2 sludge digesters. Due to the

Figure 4.3: Aerial photograph of the Bleesbruck treatment plant with some of its major elements (starting from top left and going clockwise): sand removal/degreasing unit, first AS unit, one of the 2 primary clarifiers, 2nd AS units, 2 secondary clarifiers and 2 digesters.

presence of these sludge treatment infrastructures, sludge from neighbouring WWTPs is brought to Bleesbruck, and according to their sludge composition, they are added to the wastewater at the beginning of the treatment or the centrifuges or digesters respectively.

The here described project can be considered as a follow-up project of the European project LIFE98 ENV/L/000582 where the same activated sludge wastewater treatment plant Bleesbruck has been modernised with tools for real-time monitoring and control of the treatment processes (Schosseler *et al.* (2000), Schosseler *et al.* (2003)). In fact, a higher treatment efficiency has been achieved through model-based analysis and control of the biological treatment processes, based on the input of analytical on-line data. Next to temperature, pH, conductivity and total COD after the grease removal, total solids and dissolved oxygen are monitored on-line inside the 2 AS units and quality parameters like ammonium, nitrate and phosphate are measured at their outflows. Figure 4.5 shows the pre-filtration units and analytical equipment installed at the WWTP. A model was implemented into the SIMBA software (ifak System GmbH) and is currently used to regulate the aeration in the second biology.

Table 4.4 gives the emission limits set by the Urban Wastewater Directive (CEC (1991)). For phosphorus, especially during peak loads, the WWTP does not manage to respect emission limits. But, especially in terms of nitrogen removal, the treatment capacities are largely exceeded. Apart from the fact that the nitrification unit of the WWTP is smaller than the COD removal unit and is already overloaded during wet weather conditions, the sludge treatment centrifuges



1st AS tank	$1040m^3$
1st clarifiers	$2x690m^3$
2nd AS volumes	$2x275m^3$
2nd clarifiers	$2x360m^3$

Figure 4.4: Flow of the wastewater through the Bleesbruck WWTP. Marked in blue circles are the online measurement locations and marked in red squares are the CD4WC measurement campaign points for TSS, COD, NH_4, NO_3 and PO_4.

Figure 4.5: Pre-filtration units and analytical on-line equipment at the Bleesbruck WWTP (WTW Trescon analyser for NH_4, NO_3 and PO_4).

Table 4.4: Emission limits according to the EU Urban Wastewater Treatment Directive (CEC (1991)).

Chemical Oxygen Demand (COD)	< 125mg/l
Total nitrogen (TN)	< 15mg/l
Total phosphorus (TP)	< 2mg/l

send sludge water to the influent of the WWTP. These centrifuges work only during week-days and the return waters have ammonium concentrations between 300 and 400 mg/l, so that during these periods ammonium emissions can exceed 20 mg/l. Due to the fact that no denitrification takes place, the TN limit is always exceeded, even when the WWTP manages to the treat ammonium load.

4.1.3 The receiving water system

The wastewater infrastructures of the case study discharge water into 3 main receiving waters with differing water quality.

The river *Sûre*, with its hydrographic basin of about 4250 km^2, has its source in the Belgian Ardennes, crosses Luxembourg from West to East and, in Wasserbillig (German border), flows into the river Moselle, a sub-basin of the Rhine International River Basin District. The average discharge at the study site amounts to approximately 16.2m^3/s, with a summer base flow of around 3.8m^3/s. At approximately 25 km upstream of the WWTP Bleesbruck, the Sûre is retained by a dam, becoming a drinking water reservoir serving around 33% of the Luxembourgish population. Therefore its water quality can be considered as being very good until, near the town of Ettelbruck, it receives the water of the river *Alzette*. The latter, with a basin of around 1120 km^2, crosses the highly populated and industrialised South of Luxembourg, thereby carrying a high quantity of nutrients and other pollutants. For nutrients, this is mostly due to outdated WWTPs, not fulfilling emission criteria, whereas especially steel industry is responsible for metals in sediments. The Alzette's average flow is about 6m^3/s with summer base flow of 1.2m^3/s. The river *Attert*, the smallest of the 3 rivers, collects waters from around 700km^2 and is considered to be of good water quality, until it passes the WWTP of Bissen. It seems that this treatment plant is overloaded, and it is planned that these wastewaters are connected to the Bleesbruck sewer network in the near future.

More information on the rivers will follow in section 4.2 on the measurement campaigns conducted within the context of this project.

4.2 Measurement Campaigns

This section summarises the measurement campaigns conducted in spring and autumn of the year 2005, within the framework of the EU CD4WC project (FP5). They were performed in collaboration with colleagues from BIOMATH, Ghent University. The data was collected with

Figure 4.6: Receiving waters (from top to bottom, from left to right): the Attert, the Alzette downstream of the catchment in Luxembourg city, the Sûre upstream as a drinking water reservoir, the Sûre in the catchment just before the town of Diekirch.

the intention to use them to construct an integrated model of the sewer catchment, the WWTP and the receiving waters.

4.2.1 Planning

Before execution of a measurement campaign, good planning is necessary beforehand, so that data are fit for use afterwards. Therefore, the planning should be driven by the purpose of study and questions like: What do we need the data for? What variables are of importance? For how long do we need data for and at what frequency? Where do we need to measure? Recommendations for the set-up of an integrated measurement campaign can be found in Vanrolleghem *et al.* (1999b).

To calibrate a model of a dynamic system, a high number of data points are required. Typically, data collected during monitoring surveys are designated to check basin-wide water quality for regulatory compliance. These are certainly not appropriate to calibrate a model on an urban catchment level, as some necessary variables might not be measured at all, or the frequency and the locations of measurements are generally not sufficient (Radwan *et al.* (2003)).

As already mentioned several times, the modelling purpose is the investigation of impacts onto receiving waters under different management scenarios of the urban wastewater system. In general, impacts are multiple and require thorough characterisation of the rivers at different scales and for different variables. This is a complicated task as it requires biological, physico-chemical and structural analysis in all compartments of the river, i.e. water column, bottom and banks. In this study, a selection was made according to the purpose of the study, which is to assess the impact of the UWWS regarding biochemical criteria in the water bulk.

Inside the integrated model, chemicals will be modelled using KOSIM-WEST (described in Chapter 3), ASM2d (Henze *et al.* (1999)) and RWQM (Reichert *et al.* (2001)). The variables to be measured were decided to be total chemical oxygen demand (COD), soluble COD, biological oxygen demand (BOD), total suspended solids (TSS), Chlorophyll A (ChlA), ammonium (NH_4), nitrate (NO_3) and orthophosphates (PO_4). The number of measurements needed to characterise each variable can be considered infinite, both due to spatial and temporal factors. First of all, the modelled river stretches extend over about 20 km, through different geological regions and therefore different vegetation and soils, and the urban catchment contains about 20 subcatchments and their associated multiple CSOs. On a temporal scale, the river conditions are changing over a year's season, pollution may be more or less diluted and different vegetation occurs in the river. The combined sewage has very fast dynamics regarding both hydraulics and chemical composition and conditions in wet weather situations not only depend on each event itself, but also on the amount of infiltration etc. For this study, it was decided to do measurements at the WWTP and the river only. The reasons were numerous: First of all, limits in financial resources often set boundaries to the number of measurements that can be performed within a project. As no dynamic data was available at all to allow river model calibration, the river system was given priority and the high availability of data at the WWTP were estimated enough in number and quality to calibrate the sewer network model. Moreover, due to the high number of CSOs in the catchment (around 60) makes it impossible to sample all of them or to chose a representative

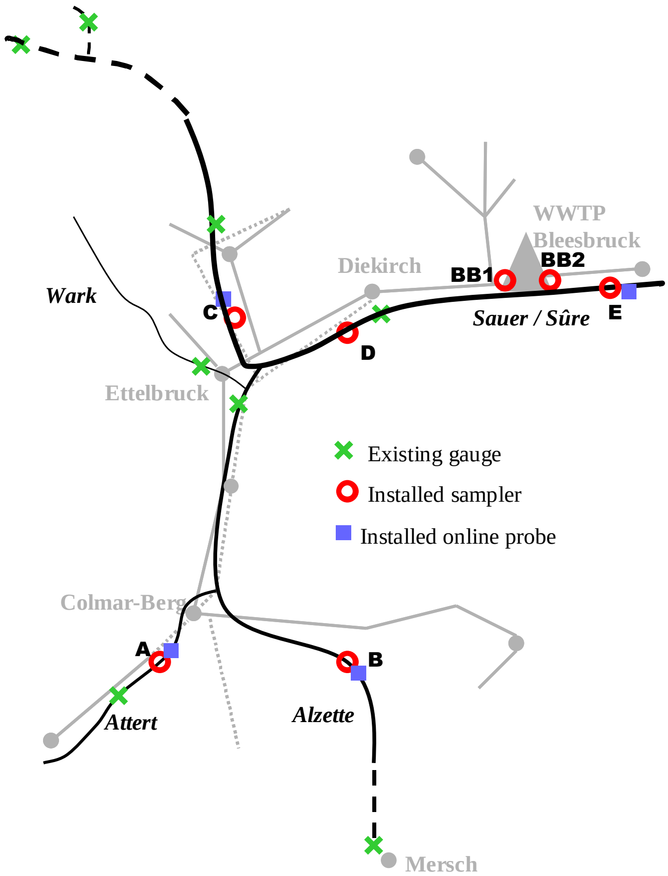


Figure 4.7: Locations of measurement points.

CSO to monitor. The conditions on the urban surfaces (regarding surface parameters or DWF pollutant compositions) and inside the network (regarding infiltration) would be supposed to be uniform over the entire catchment.

The sampler locations were chosen so that data were available at each input to the river model to be constructed, i.e. A, B, C in Figure 4.7. Locations D and E in the river are calibration points for the river model. Another 2 measurement points were located at the WWTP (BB1 for inflow and BB2 for outflow). The data from 7 existing gauges and their respective rating curves were obtained from the Luxembourg Water Agency. One gauge data set, situated just above sampling point C, was provided by the 'Environment and Agro-Biotechnology' (EVA) department of the CRP Gabriel Lippmann (Luxembourg). As the river's biochemical quality dynamics are different during spring and autumn, due to growth/presence of algae during spring and death/absence of algae during autumn, two measurement campaigns were planned: the spring measurement campaign took place between 16 June and 30 June 2005 and the autumn measurement campaign between 24 September and 4 October 2005. In order to both have measurements over longer periods for overall characterisation of river water quality and measurements for dynamic calibration of the model, the campaigns were composed of daily composite samples during 10 to 12 days and of 2 days of intense 2 hour sample campaigns. The latter period should be at

Figure 4.8: Pictures of the sampler at the WWTP outlet (left), the YSI Hydrodata multi-parameter probe (middle) and water samples in the lab (right).

least as long as the flow time of a volume element to traverse the system from its upstream to its downstream boundary. The daily mixed samples were composed of 4 samples taken every 6 hours in the river, and 60 samples taken every 24 minutes at the WWTP respectively.

4.2.2 Materials and methods

Three multi-parameter probes (YSI Hydrodata) were installed at A, B and E (see Figure 4.7) to measure temperature, conductivity and dissolved oxygen (DO), plus one handheld DO probe at C. The pH was known to be constant, so it was not monitored nor will it be modelled. Five refrigerated automatic samplers were placed on river sites, where electricity was provided by pumping stations or other facilities located near the sampling sites. River samples were collected once a day, whereas the samplers at the WWTP were emptied more often as no refrigeration happened onsite. To avoid that sediment material was entrained and to make sure that sampling tubes stayed located in a constant flow region in the river, they were attached to vinery screws or pouls fixed inside the river sediment. In the lab, TSS were retained by a 0.45μm glass fibre filter and dried at 105°C. Chlorophyll A was determined using fluorometric analysis. Samples for COD, NH_4, NO_3 and PO_4 were analysed using spectrophotometric test kits both on-site, at the SIDEN laboratory (Bleesbruck), or for the filtered samples at the BIOMATH laboratory (Ghent University).

Apart from a few short heavy local rains, the month of June 2005 was a very dry month (see Figure 4.9). By inspection of gauges in the river, it was deduced that these small rain events had no effect on river flows (i.e. evaporation and infiltration dominant). For the autumn campaign, one rain event (day 274) did increase flows in the river.

In parallel to the water quality measurements, 4 tracer tests along the river were performed to get information about river hydraulics, i.e. travelling times and dispersion. Rhodamine WT was identified to be an appropriate tracer dye as it has little adsorption on sediments and

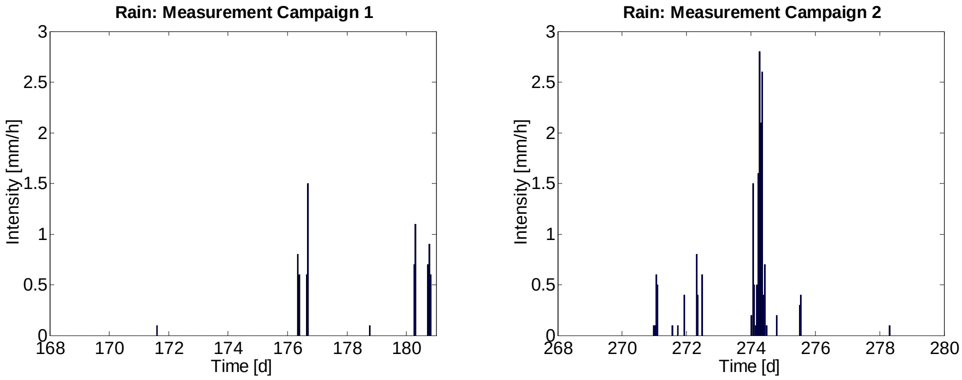


Figure 4.9: Rain data for the two measurement campaigns.

Figure 4.10: Pictures of tracer test in the Alzette and the Sûre.

is measurable at very low concentrations using a YSI in-situ fluorometer. In Figure 4.11, an example of results from such a tracer test is shown.

4.2.3 Preliminary results

Average results correspond to what was expected in advance regarding the biochemical statuses of the distinct river parts. In terms of ammonia, measurement point B (Alzette) shows concentrations around 3-4 mg/l. Such high values are stemming from badly treated wastewaters upstream, WWTPs not complying to effluent standards and currently in planning or building process to be upgraded. Oxygen concentrations in summer can attain supersaturation levels, due to high number of sessile algae. At other locations, oxygen levels can go below 5 mg/l at night. Measurement point C on the other has low levels of ammonium. More discussion on results of the measurement campaign is found in sections of construction and calibration of the river model (see Chapter 5 section 5.4). Treatment plant effluent measurements confirm that the WWTP is not complying with the Urban Wastewater Directive in terms of total nitrogen (TN) and total phosphorus (TP) (CEC (1991)).

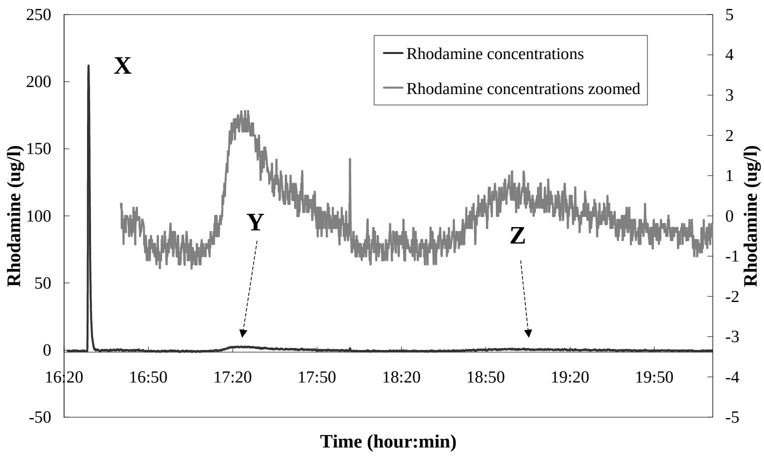


Figure 4.11: Example of a tracer test measurement at locations X, Y, Z in the Sûre. X is situated just after the Alzette has flown into the Sûre and distances are $\overline{XY}$ =1000 m, $\overline{YZ}$ =1500 m.

4.3 Pressures and Impacts

Identification of anthropogenic pressures upon a river basin is a task required by the WFD Common Implementation Strategy (CEC (2001), CEC (2003)). Within these pressures are point sources like WWTP and CSOs, and for assessment of impacts it requires collection of emission data from sewer and WWTP, as well as available data giving information on the river status (Borchardt & Richter (2003)).

Data from sporadic measurements over 5 years from the Luxembourg Water Agency and data from the measurement campaigns mentioned above were analysed to characterise river stretches. It could be confirmed that the good quality of the Sûre deteriorates after mixing with the Alzette. The latter travels through the industrialised and most populated areas of Luxembourg, collecting effluents from many WWTPs and therefore carrying concentrations of often more than 1.5mg/l of ammonium. In summer, eutrophication is well visible in some of the stretches so that supersaturation is giving oxygen levels above 10 mg/l. In other places, where oxygen concentrations do not reach such high levels, organic pollution can bring dissolved oxygen concentrations below 5 mg/l in early morning. Diffuse pollution is difficult to assess as agricultural activity varies and exact data do not exist.

At the WWTP, despite of model-based oxygen control, nitrification is not efficient through low autotroph development due to a too low sludge age. Another cause of bad nitrification is the presence of on/off actuators, not well suited for exact and efficient aeration. Also, phosphate peaks are not efficiently eliminated due to delayed control action so that the effluent contains total phosphorus (TP) and total nitrogen (TN) concentrations above the limits set by the Urban Wastewater Treatment Directive (CEC (1991)). Currently a sludge storage volume is being built in order to accept the sludge coming from smaller treatment plants, which are so far just added to the influent as they arrive.

The sewer system is composed of a main collector running along the rivers to gather wastewater from approximately 25000 inhabitants, a dairy, a brewery, a slaughterhouse and 2 commercial areas. Apart from a few smaller storage pipes and 2 local retention basins, no storage volume is available so far. However, construction works are ongoing in the sewer network with the transformation of some CSOs into retention basins and a parallel collector to reduce the hydraulic loads into the existing collector, especially as another catchment and industrial area will be connected in the future. Although no measurements can support this, some older overloaded CSOs appear to overflow regularly. This was shown by simulations with the sewer model and confirmed by experience from the operator. Using WWTP inflow data, infiltration was evaluated and can range from 100 to nearly 300% in summer and in winter respectively.

Using this knowledge on deficits within the system, the scenarios developed in Chapter 6 will propose alternatives to improve wastewater management within that catchment and simulations of these scenarios will serve for impact assessment of the catchment on the receiving waters.

Chapter 5

The Integrated Model

The Chapter describes the construction and calibration of the integrated model of the sewer-WWTP-river system 'Bleesbruck'. First, a more general introduction to the here adopted approach for model construction and calibration is given. In the following sections, each submodel is described individually and calibration results are discussed. In the last section, integrated simulations are briefly introduced.

The following chapter is partly developed from the following articles:

Solvi, A.-M., L. Benedetti, S. Gillé, P. M. Schosseler, A. Weidenhaupt and P. A. Vanrolleghem (2005). Integrated urban catchment modelling for a sewer-treatment-river system. *10th International Conference on Urban Drainage*, 21-26 August 2005, Copenhagen, Denmark.

Solvi, A.-M., L. Benedetti, S. Gillé, P. M. Schosseler, A. Weidenhaupt and P. A. Vanrolleghem (2006). Construction and calibration of an integrated model for catchment, sewer, treatment plant and river. *7th International Conference on Hydroinformatics*, 4-8 September 2006, Nice, France.

5.1 Model Construction and Calibration

Model Construction

As already mentioned in Chapter 2, it is not a trivial task to build an integrated model, first of all due to the complexity of the integrated urban wastewater system and therefore the related model itself, and secondly due to the difficulty for the user to choose the appropriate sub-models for the integrated model out of a multitude of possible options (Rauch *et al.* (2002)). The choice depends on the level of data availability and the objectives of the study in question (e.g. Willems (2003)). As a main guidance, the model should be as detailed as necessary and as simple as possible to achieve best possible results within case study objectives (Meirlaen *et al.* (2001)).

The aim of the presented project and model lies within the context of the implementation of the EU WFD (CEC (2000)), where long-term simulations and scenario analysis have been chosen for impact assessment of the urban wastewater system on the receiving river system. Considering the high variability of rain events, simulation results then include a wide range of different system behaviour in response to the different events (Rauch *et al.* (2002)). Evaluation is performed with respect to biochemical components like DO, COD, ammonia and orthophosphates, so that the model needs to comprise all relevant biochemical processes. However, the desired long-term evaluation needs the model not to be excessively complex, so that calculation times stay reasonably short, especially as sewer and combined sewer overflow dynamics need high temporal resolution in model computation. Hence, the use of conceptual models for hydraulics and transport have been favoured instead of hydrodynamic models, as they require less data and are nevertheless expected to be able to give good enough results when looking at long periods. To further reduce the model, model simplification was performed by diminishing the number of simulated CSOs and by leaving out irrelevant processes from the river water quality model. Moreover, the large size of the system makes it very difficult to gather enough data to calibrate a detailed model that would be able to predict responses of the system to each rain event at multiple locations.

Linkage difficulties at the interfaces between the subsystems and data transfer problems between sub-models during simulations are a problem, and it is therefore useful to work with a single simulation tool for all subsystems. Hence, for this project, the entire sewer-WWTP-river model is implemented in the WEST® software platform (MOSTforWATER N.V., Kortrijk, Belgium, Vanhooren *et al.* (2003)). The underlying state-of-the-art models were selected according to the above mentioned needs. For the sewer catchment and network, the implemented KOSIM-WEST model (described in detail in Chapter 3) was applied. For biochemical reactions in the system, a simplified version of the IWA river water quality model RWQM (Reichert *et al.* (2001)) and the IWA activated sludge model ASM2d (Henze *et al.* (2000)), where chemical phosphorus precipitation can be simulated, are used. The sub-models are connected by means of interface models (Benedetti *et al.* (2004), Vanrolleghem *et al.* (2005b)), which transform the state variables of one sub-model into the state variables of the following sub-model. Components of such an integrated model in the WEST® configuration environment are depicted in Figure 5.1.

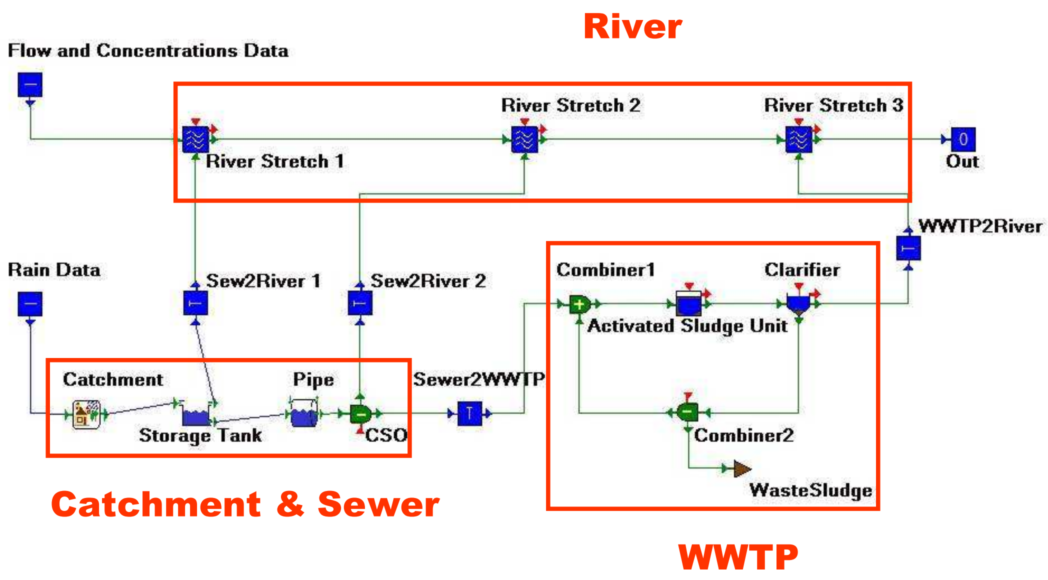


Figure 5.1: Integrated model inside the WEST® environment, displaying main subunits for the subsystems urban catchment, sewer, WWTP, river and connectors for transformations of the different submodel variables.

Overall, for the 3 submodels, all available data were gathered, analysed for quality and subsequently used for the model construction. Before calibration, in the case no data was present, parameter values were either fixed with default values from literature, or estimations were done where possible. The adopted methodology and approaches to build an integrated model in order to achieve our goals are presented in this Chapter.

Model Calibration

During the calibration process, the parameter set of the model is adjusted so as to reduce to a minimum the difference between model predictions and measured data of the real system. Hence, after the model is built and initial conditions are set, the dynamic model can be run and the simulation results can be compared to system information. These results should lie as much as possible within the errors of our measurement data. To match simulated results and measured data, one can proceed to varying model parameters manually via a trial and error process or one can use a more sophisticated automated calibration procedure. For manual calibration, the 'goodness of fit' is generally judged by visual inspection of results. Mathematical objective functions are needed for an automated calibration procedure. In this case, by sampling within a defined parameter space, multiple simulations are run and the best parameter set with respect to a chosen objective function is identified. There exist many statistical measures whose numerical value evaluates the quality of the model given a certain parameter set. The goal of the study will determine the required preciseness in predictions, hence the used objective function or the necessary indicators for visual calibration. Such objective function will for example be chosen according to whether peaks or low flows are important for a given study. Also, during visual calibration, focus might lie on mass balances over simulation time rather than process dynamics.

No automated calibration was used for this project. Visual calibration was considered of primary importance before applying any mathematical goodness of fit criteria. Hence, the overall dynamics, peak times and amplitudes for water quantity and then for pollutants are expected to best match the available data, at least within a reasonable range of magnitude of the errors associated with the observations. The range of errors will probably depend on the variable, as for example the uncertainty on water quantity in the sewer network will be lower both in measurement and in the model compared to the uncertainty on water quality variables.

To perform the calibration for the submodels, the following approach was adopted: Assuming that best available knowledge in the field is embedded within the default parameters of the model, a subset of parameters, which considerably influence the decision variables, were changed (Vanrolleghem (2007)). A rigorous sensitivity analysis to select best parameters was not performed. One should however be aware that if a model is overparameterised with respect to data availability, the subset of calibration parameters is not unique (Brun *et al.* (2001)). Chatfield (1995) discusses and gives extensive literature on aspects of model formulation, data mining and uncertainty.

A good overview on statistical tools for model adequacy testing is given in Berthouex & Brown (2002). Over long simulation periods, like in this case, relative magnitudes of different quantities should be predicted accurately (Beven (2001)). Considering that within this study, the impact of the urban wastewater system on the receiving river is to be estimated using long-term scenario simulations, mass balances were estimated important, and can be quantified by the *bias* measure

$$B = \frac{\overline{S}}{\overline{M}}, \tag{5.1}$$

where $\overline{S}$ and $\overline{M}$ are the mean values of the simulated and the measured data.

Another often applied mathematical measure is the root mean square error (RMSE), which quantifies how much the model over- or underestimates the measurements; it is the mean square difference between the predicted and the observed value,

$$RMSE = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(M_i - S_i)^2} \tag{5.2}$$

where n is the number of observations, and, M_i and S_i are the measured and simulated data points respectively. The coefficient of determination R^2 is a commonly used measure in model evaluation and is the square of the Pearson's Product Moment Correlation Coefficient, so that

$$R^2 = \left[\frac{\sum_{i=1}^{n}(M_i - \overline{M})(S_i - \overline{S})}{\sqrt{\sum_{i=1}^{n}(M_i - \overline{M})^2}\sqrt{\sum_{i=1}^{n}(S_i - \overline{S})^2}}\right]^2. \tag{5.3}$$

However, attention has to be paid when using R^2 as it is insensitive to consistent over- or underestimations of predicted values (e.g. Achleitner (2006)). Another performance measure, often used in hydrological runoff modelling, based on the error variance is the modelling efficiency of Nash & Sutcliffe (1970), defined as

$$NS = 1 - \frac{\sum_{i=1}^{n}(M_i - S_i)^2}{\sum_{i=1}^{n}(M_i - \overline{M})^2}. \tag{5.4}$$

NS values equal to 1 indicate a perfect fit between observed and predicted data, while NS values equal to 0 indicate that the model is predicting no better than using the average of the observed data. It should be noted that the above described modelling efficiency is not an ideal measure of goodness of fit in the case of water quantity modelling (Beven (2001)): often errors are higher at peak values so that, due to the errors being squared, it tends to give greater weight to high flows, which might be important in flood prediction, but not so much, as in this case, when one is interested in high concentration periods, i.e. low flows. To remedy against the overweighting of high flow errors, we can use the logarithmic form of the Nash-Sutcliffe coefficient:

$$logNS = 1 - \frac{\sum_{i=1}^{n}(ln(M_i^t) - ln(S_i))^2}{\sum_{i=1}^{n}(ln(M_i) - ln(\overline{M}))^2} \tag{5.5}$$

Some of these measures will be used in the described calibration process of the three individual models where they apply, however visual inspection and calibration were estimated of major importance when evaluating the quality of the model in terms of the objectives of the model.

Although the calibration was performed individually for each subsystem, all calibrations were performed using data of the year 2005. Two measurement campaigns were conducted in June and September 2005 at the WWTP and in the river (see Chapter 4). For sewer and WWTP, calibrations were done for the period from March to November using WWTP online data and the measurement campaigns data. Simulation results from the calibrated sewer model were fed into an already calibrated WWTP model for validation. For the river model, only data from measurement campaigns was available, and outcomes from the sewer and the WWTP were inputs to the river model. Approaches adopted for calibration differ for each subsystem.

For the **urban drainage model**, several steps were used. First, hydraulic calibration of the hydrologic model was performed using hydrodynamic simulation results of the main collector in InfoWorks^{TM} CS (Wallingford Software, UK). Second, water quantity and quality were calibrated with online measurements at the WWTP, but overflows at individual catchments could not be adjusted, as, apart from visual inspections and experience of the operator, no data was available regarding the overflow structures activity.

Using the existing SIMBA model as a basis, the **WWTP model** in WEST® has been calibrated three times:

- One-week model calibration
- One-week model validation
- 8 months model calibration

In each calibration step, the model configuration was changes slightly, however the model quality improved every time. The general approach adapted here to calibrate the ASM biochemical parameters can be summarised as follows: after characterisation of influent compositions, biochemical parameters were calibrated to fit sludge balances and oxygen before nutrients were fitted to data. The one-year calibration was a necessary step to be able to use the model for the purpose of this long-term assessment of the system, i.e. to account for seasonal differences. During the last 10 years, several attempts within different groups have been going on to systematise ASM calibration for activated sludge in order to promote the use of the models and to allow for easier comparison between case study results. The existing IWA Task Group joins several members of the international research community in the field to unite calibration protocols and a good overview of these protocols can be found in Sin *et al.* (2005).

The main objective of the **river model** calibration is water quality as it will be the relevant criterion during scenario analysis. The river model was calibrated using the data from the two measurement campaigns. The main components of importance are nutrients and dissolved oxygen.

5.2 Catchment and Sewer Network Model

This section follows and explains the sewer model construction and calibration. Data collection required for the KOSIM-WEST model parameters is summarised in Figure 5.2. To enlighten the model in order to keep calculation times to a minimum without loosing any quality in the simulation results, original CSOs within one subcatchment were lumped into one CSO in the model, and this was verified comparing overflow masses from original and lumped models. To improve hydraulic results, the conceptual model was calibrated using simulation results from a hydrodynamic model in InfoWorks^{TM}CS. As no measurements were available at CSOs, the model is calibrated using online flow and quality measurements from the WWTP influent.

5.2.1 KOSIM-WEST model formulation

The KOSIM-WEST modelbase are extensively described in Chapter 3. The modelbase contains models of hydrological nature to transform rainwater into runoff, to generate DWF from households or industries and to transport this combined sewage to the WWTP. The user can make distinction between impervious and pervious catchment surfaces. On the former the model accounts for wetting, depression and evaporation losses, whereas on the latter rainwater can also infiltrate. These models generally have linear or exponential behaviour and parameters (rates, maximum losses, ...) depend on soil and vegetation characteristics. Up to the main collector, i.e. on the surface and inside local sewer networks, water drainage is modelled by 3 tanks-in-series, accounting for time translation and retention of peaks. For pollution, concentrations from the surface are either supposed constant or, for particulate matter, can be chosen to vary due to accumulation (considering the antecedent dry weather period) and washoff (depending on the intensity of the rain). Dry weather patterns are generated using population densities and water use per inhabitant. Transport in main collector pipes, going to the WWTP, are also modelled by tank cascades, where the parameters are calculated using the Kalinin-Miljukov method. An additional parameter for maximum water quantity has been added permitting to limit water quantity passing though a pipe, thereby accounting for backwater effects. Besides CSO structures, two types of stormwater tanks have been implemented; a first flush tank and a stormwater tank with sedimentation. But, as explained in Chapter 3, all models can be modified, or new ones added according to the user's need.

5.2.2 Data collection

Values for industry wastewater quantity and pollution as well as infiltration are already given in Chapter 4 section 4.1.1 and the supplementary data on individual catchments and the main collector needed for the model are compiled in Figure 5.2. This data was collected from different stakeholders and comprises demographic data (STATEC (2003)), a WWTP extension study (Ingenieurbüro Peil, Germany), aerial photographs, sewer maps from the operator SIDEN and an engineering office (Schroeder & Associés, Luxembourg), where no precise data was available and parameters were taken from literature.

The needed input parameters for each individual catchment are the number of *population equivalents (PE)* from domestic, commercial or industrial wastewater sources, drained *surfaces*

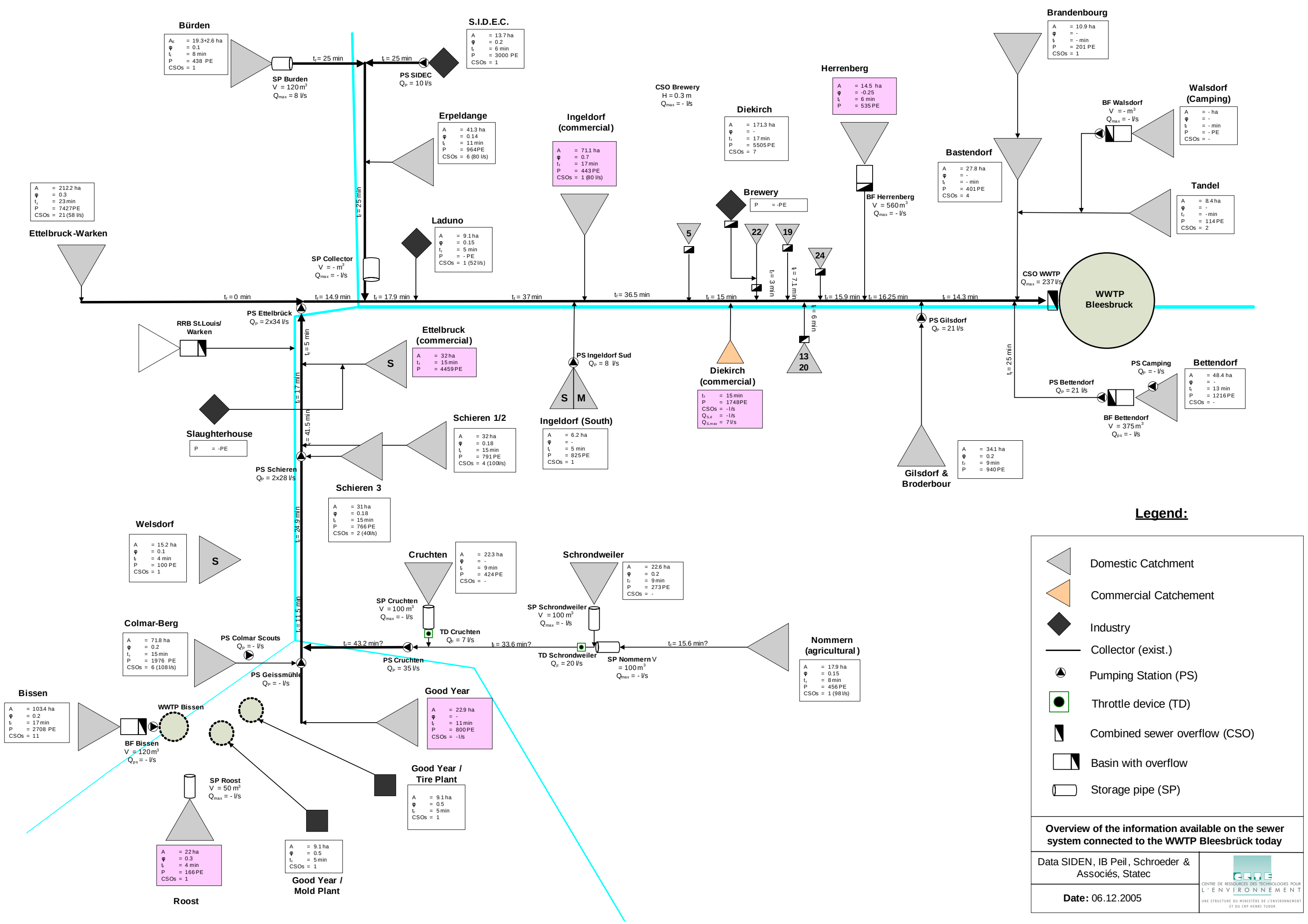


Figure 5.2: Schematic representation of the Bleesbruck urban catchment containing the data needed to construct the model in KOSIM-WEST. Some disconnected catchments will only be connected in future.

(A) and their degree of *imperviousness* (φ). *Flow times* (t_f) in a subcatchment were estimated using elevations and extensions of the catchment. Also, local sewer networks were inspected for *CSO structures* or other special structures like storage pipes and tanks. The main collector, which takes up all combined sewage from the subcatchments, will also be modelled by tanks-in-series. Using *diameters*, *slopes* and *lengths*, parameters n and k were evaluated using the Kalinin-Miljukov method described in Chapter 3. Flow times in the collector were also calculated.

Parameters relating to climate and surface runoff are given in Appendix A. They determine the amount of rainwater losses, and the here chosen parameter sets are taken from ATV (1992) and from the KOSIM handbook (ITWH (2000)) for impervious and pervious surfaces (clay/loess soils) respectively, while values for accumulation and washoff of COD were taken from literature (Ashley *et al.* (2004)).

5.2.3 CSO reduction in subcatchments: Preliminary analysis

Within the data collection phase, the location and draining area of CSOs were extracted from local sewer maps and the number of CSOs for subcatchments are indicated in Figure 5.2. This number of detected CSOs in villages ranges from 1 in small catchments to 20 CSOs for the town of Ettelbruck. While gathering this data, it was realised that modelling all of the CSOs would certainly blow up model size and simulation times. Moreover, keeping every detail in the model would require more input parameters probably increasing model uncertainties, which does therefore not necessarily improve the results (Willems & Berlamont (2002)). Hence, it was decided to keep the number of CSOs to a minimum, always leaving the possibility of refining the model according to future needs.

To check whether lumping the CSOs into a smaller number of CSOs would not change results significantly, the following analysis was performed, here illustrated on the example of the town of Ettelbruck (see Figure 5.3).

The critical flow rate Q_{crit} at which the overflow structures activate were calculated using Q_{max} from downstream pipes, supposing that the pipe downstream cannot take more water and causes any excess water to flow over. Q_{max} is hence evaluated according to the formula,

$$Q_{max} = a \left[-2 \cdot \log \left(\frac{2.51 \cdot \nu}{d\sqrt{2gds}} + \frac{k_s}{3.71d} \right) \cdot \sqrt{2gds} \right]. \tag{5.6}$$

where, a is the cross-sectional area, s the slope, d the diameter, g gravity, ν kinematic viscosity and k_s the roughness coefficient of the pipe. The CSO model is described in Chapter 3 section 3.3.2.

First, the CSOs located on the outskirts of a town, often new residential areas and which have quite a high overflow limit with regard to the drained area and are therefore suspected to overflow rarely, were tested using a one-year simulation (for Ettelbruck: CSO 1, CSO 2, CSO 4, CSO 5, CSO 12, CSO 21). In case of no overflows, the CSO was omitted from the model. Often the throttled base flow would enter an older sewer part, where the CSO had been designed for a much smaller area than it was confronted with now. Indeed, these CSOs do overflow regularly as confirmed by the operator. Hence, in a second step, CSOs connected in-series were lumped into

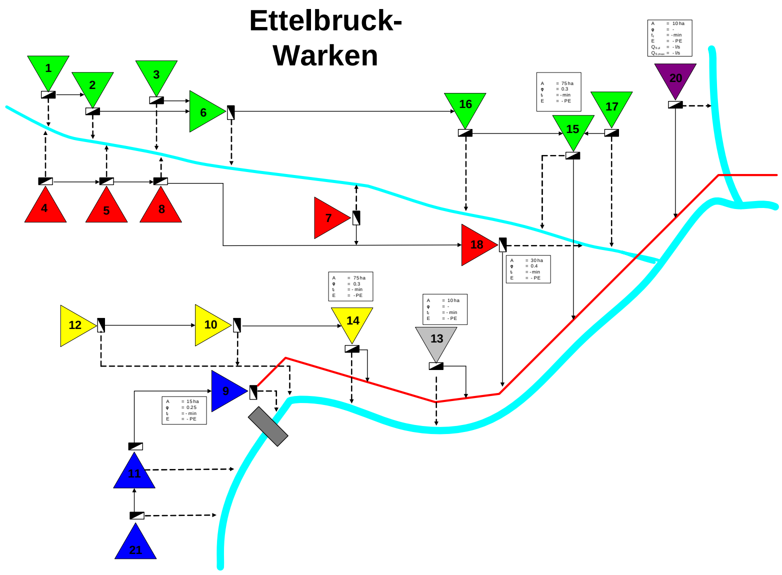


Figure 5.3: Schematic representation of CSOs with their respective drained surface in Ettelbruck, as synthesized from sewer maps (see Figure 5.2 for legend).

one CSO and this simplification was also tested with one-year simulations (e.g. for Ettelbruck: CSO 3, CSO 6, CSO 16, CSO 15, CSO 17). The new critical overflow limit for the lumped CSO was adjusted and both mass balances and overflow peaks were compared. An example of such comparison between the sum of all CSOs and the lumped CSO is depicted in Figure 5.4. It can be seen that the overlap in the shown example is very good considering the whole event. For the illustrated example, the mass balance ratio B of the calibrated CSO over the sum of the original CSOs is 0.998 for the whole year. Further simplification was tested for CSOs connected in parallel to the main connector if they were discharging into the same river model stretch.

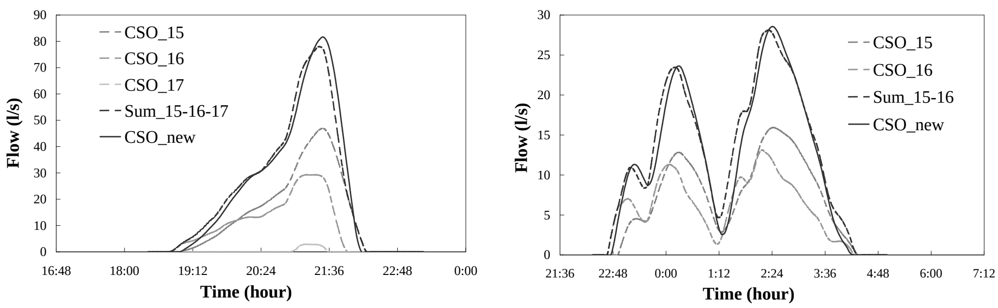


Figure 5.4: Model reduction example of multiple CSOs into one CSO. The figures contain two main overflows in the catchment during a year of simulation. They depict the simulated overflows from the individual CSOs as found in the sewer system, the sum of the latter and the simulation results of the new, calibrated CSO.

5.2.4 The Bleesbruck sewer model in WEST®

After collecting the available data and having reduced the number of CSOs from around 64 known CSOs down to 16, the overall model for the Bleesbruck catchment consists of 21 catchment units, 22 pipe units with a total of 138 tanks-in-series, 6 pumps, 4 storage volumes and is shown in Figure 5.5.

The influent rain file contains rain data, registered at 10-minute time intervals near the town of Ettelbruck. As no other high resolution weather station was available in the region, all subcatchments are fed with the same rain data.

5.2.5 Hydraulic model calibration using InfoWorks^{TM}CS

Considering that no flow data from the collector was available, a quantitative local calibration of the sewer model against measurements becomes impossible. For this integrated study, where emission loads and immission concentrations in the river are assessment criteria, the objective of this calibration step is to make sure that mass balances of outflow and overflow from the sewer are as close to reality as possible.

As already mentioned in Chapter 3, hydrological modelling of the sewer system using tanks-in-series lacks the ability to simulate the system's behaviour when pressurised flow occurs. Hence, conditions upstream will not influence flows downstream and neither will an obstructing structure downstream provoke any water accumulation upstream. However, to improve the model in terms of model structure, a conceptual backflow model was implemented to ensure that some of the witnessed backwater effects could be represented in the main collector. The principle of the used backflow model is described in Chapter 3 section 3.3.2.1 and uses a maximum flow parameter Q_{back} in each pipe stretch. The latter sets a maximum value to the flow in a pipe and any excess water, instead of flowing downstream to the WWTP, gets sent back into the previous pipe stretch and so forth, until it is discharged by a CSO structure. To calibrate Q_{back}, we use InfoWorks^{TM} CS simulations of the exact same model of the collector as was implemented into WEST®. The former is depicted in Figure 5.6. A detailed geometric model using exact pipe and manhole coordinates could also have been used, but this data was not available at the time.

The method is illustrated on the southern part of the 'Bleesbruck' collector model, which is schematically represented in Figure 5.7. It contains only one overflow structure at the beginning of the collector to ensure backwater effects. By inspecting simulation results from the model in InfoWorks^{TM}CS, the maximum outflows from these pipes are used as Q_{back} limits for the KOSIM-WEST pipes. Because the catchment runoff models are not the same in the two softwares, it was decided to use KOSIM-WEST catchment outflows as inflows to the collector in Infoworks^{TM}CS. This way the method of comparing flows through the pipes could be more rigorously applied as their possible differences will not be caused by differences in catchment runoff predictions.

For the individual pipe stretches, Table 5.1 shows geometric data (L = length, D = diameter, s = slope), calculated KOSIM-WEST model parameters (n = number of tanks-in-series, k = retention parameter, Q_{max} = maximum outflow in gravity flow) and the calibrated backflow parameter Q_{back}. A first observation will be that, Q_{back} values are increasing from down- to upstream. This reflects that there cannot be more water coming from upstream than can pass

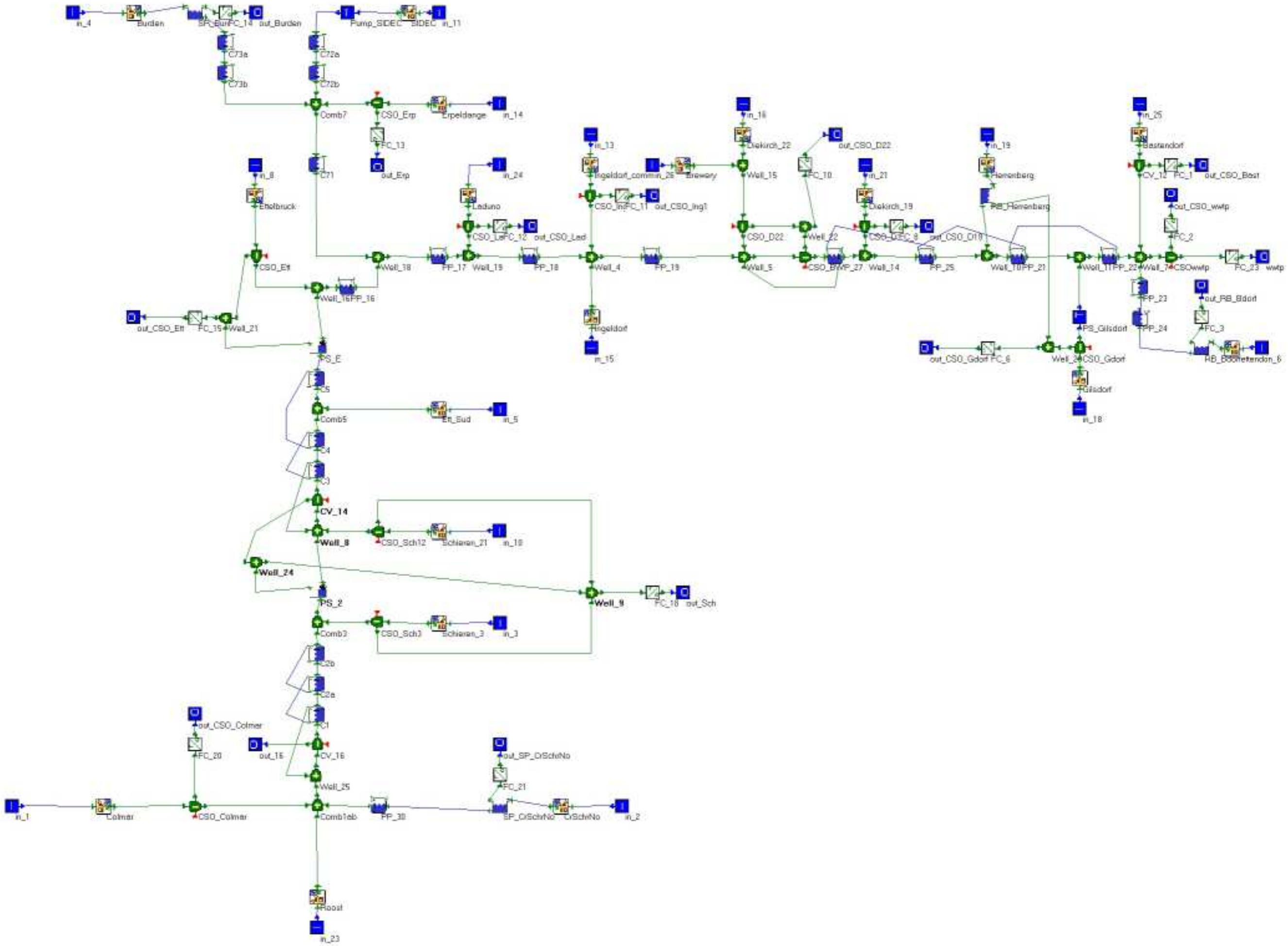


Figure 5.5: Bleesbruck urban catchment model configuration in WEST®. The structure of the model is reflected in the data compilation sheet in Figure 5.2. WEST®icons are found in Figure 5.1.

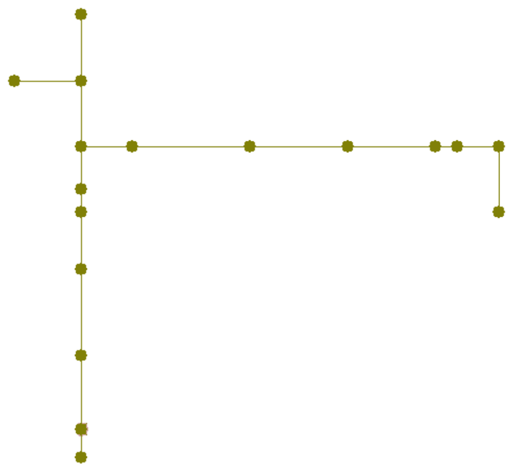


Figure 5.6: Model configuration of the Bleesbruck collector in InfoWorks^{TM}CS.

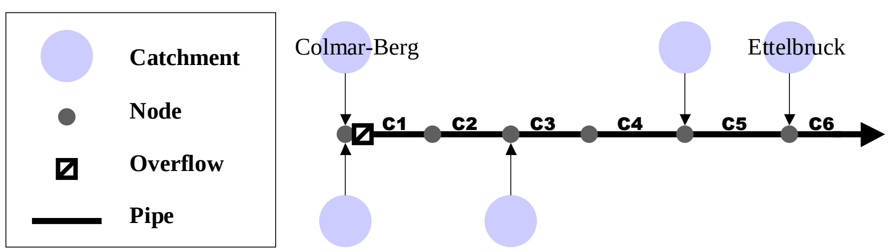


Figure 5.7: Southern part of the modelled 'Bleesbruck' collector.

Table 5.1: Parameter values for pipe stretches of the southern Bleesbruck collector (backgrounds: white: geometric data, grey: calculated using Kalinin-Miljukov, light blue: calibrated).

	L [m]	**D [mm]**	**s [-]**	**n [-]**	**k [s]**	**Q_{max} [l/s]**	**Q_{back} [l/s]**
C1	425	300	0.002	7	80.1	44	75
C2	1100	400	0.002	13	92.5	94	75
C3	1310	400	0.001	8	253.9	66	90
C4	875	500	0.002	8	103.5	169	130
C5	350	800	0.002	2	122.6	584	210
C6	645	500	0.001	3	288.5	119	700

Table 5.2: Mass balances of Infoworks^{TM}CS and WEST® over one year.

		Outflow(%)	**Overflow(%)**
Infoworks^{TM}CS		81.5	18.5
KOSIM-WEST	Without backflows	91.3	8.7
	With backflows	83.8	16.2
	With fine-tuning	81.7	18.3

downstream. Hence, maximum flow rates downstream have influence on possible flow-through quantities upstream. For stretches C1, C3 and C6, $Q_{back} > Q_{max}$ as pressure will allow for higher flows than the gravity maximum flow. $Q_{back} < Q_{max}$ occurs if the immediate upstream stretch fixes the flow and there is no or not enough water brought from a catchment inflow (i.e. C2, C4 and C5).

Table 5.2 shows values of simulated outflow volumes over a year of simulation. KOSIM-WEST results are given for 3 cases: without backflows, with backflows according to Infoworks^{TM}CS outflows from every pipe, and finally with further fine-tuning of Q_{back} values to further increase these backflows. Adjusting Q_{back} values by inspecting outflows in Infoworks^{TM}CS does allow getting a good approximation. The fine-tuning method allows to further fit mass balances, but whether this fine-tuning is really necessary and appropriate is still open for debate. In fact, it consists of further decreasing Q_{back}, thereby cutting peak flows that might be important in the subsequent evaluation of the model results. Figure 5.8 shows the simulated outflows for this simple system and it can be seen that, generally, a good visual fit is obtained. However, for single events and peaks it is not possible to achieve a complete match. Differences appear in the amplitudes of the wet weather peaks (KOSIM-WEST flows are higher than Infoworks^{TM}CS) and in the tailing part of a peak; unlike the hydraulic model, the hydrological model cannot activate storage volume, which contains the water that will be sent after the main water peak is passed. This leaves a spot for further improvement of the backflow model by, for example adding tanks inside the backflows that would simulate the ignored storage volume in collectors.

The backflow model can be considered as an attempt to refine the hydrological model towards better mass balances and description of dynamics. According to the same principle as outlined above, backflow calibration has been applied to the rest of the collector using Infoworks^{TM}CS (model in Figure 5.6).

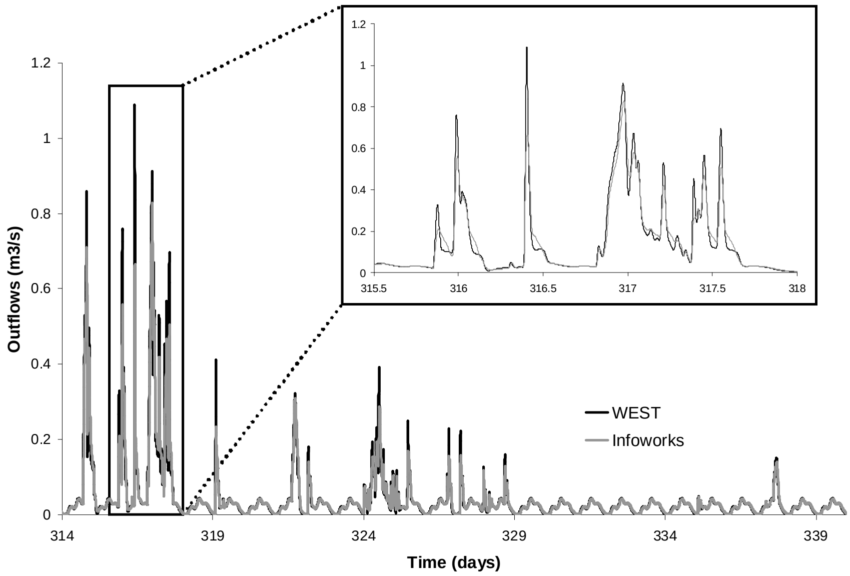


Figure 5.8: Outflows from the southern Bleesbruck collector in Infoworks^{TM}CS and in WEST® for 5 days in autumn.

5.2.6 Calibration using WWTP influent data

After application of the above described backflow model to the whole collector, simulations could be performed to calibrate dry and wet weather water quantity and quality at the inlet of the WWTP. As the impacts of individual operation scenarios will be analysed by long-term integrated model simulations, it becomes important to do a one-year calibration so as to include effects of seasonal effects of importance (sewer infiltration, evaporation, tourism,...). The main calibration objectives were the overall mass balances of flow and pollutants, and the shape of peaks.

As described in Chapter 4 section 4.1.1, the available data at the WWTP are flow and COD (measured after the sand removal/degreaser unit), ammonium, nitrates and orthophosphates (all measured after the first biology). Before using the data collected at the WWTP, they were inspected for their quality. While comparing flows to the treatment plant with limnimetric data of the Sûre, a correlation between the two was noticed. This observation was confirmed by the operator arguing that river water possibly intrudes into the sewer through low-lying CSOs during high flows in the river. Hence, as no data was available, this phenomenon cannot be included into the integrated model and it was decided to omit months where the likely-hood of river intrusion is high, i.e. November, December, January, and February. However, this should not affect the conclusions we want to draw from scenario analysis, as we are primarily interested in the high concentration periods in the river, i.e. spring, summer and autumn. Although scenario simulations will be carried out for the period March 2005 to October 2005, the simulations performed in the calibration process only run from March to mid-September as the measurements at the WWTP were interrupted due to construction works. Nevertheless, this should be sufficient data over time for calibration of the urban catchment model.

5.2.6.1 DWF calibration

First of all, calibration was performed on the DWF pattern. The default KOSIM pattern for 0-5000 inhabitants (see Chapter 3 section 3.3.1.5) was modified such that the simulated dry weather flow would fit the inflow pattern at the treatment plant. Apart from having a less pronounced mid-day peak and being shifted to the right by one hour, its shape was found to be quite similar (see Figure 5.9). Also, night values were chosen to be higher than zero, assuming some night activity and taking into account eventual flow times before entrance of the combined sewage into the main collector. Indeed, nightly flows might be linked to industrial activity. The water use was calibrated to be 120 l/d/PE. Weekend effects are taken into account by reducing flow by a factor 0.7 and the tourist season factor is left to one, as no systematic water increase in DWF could be identified for 2005.

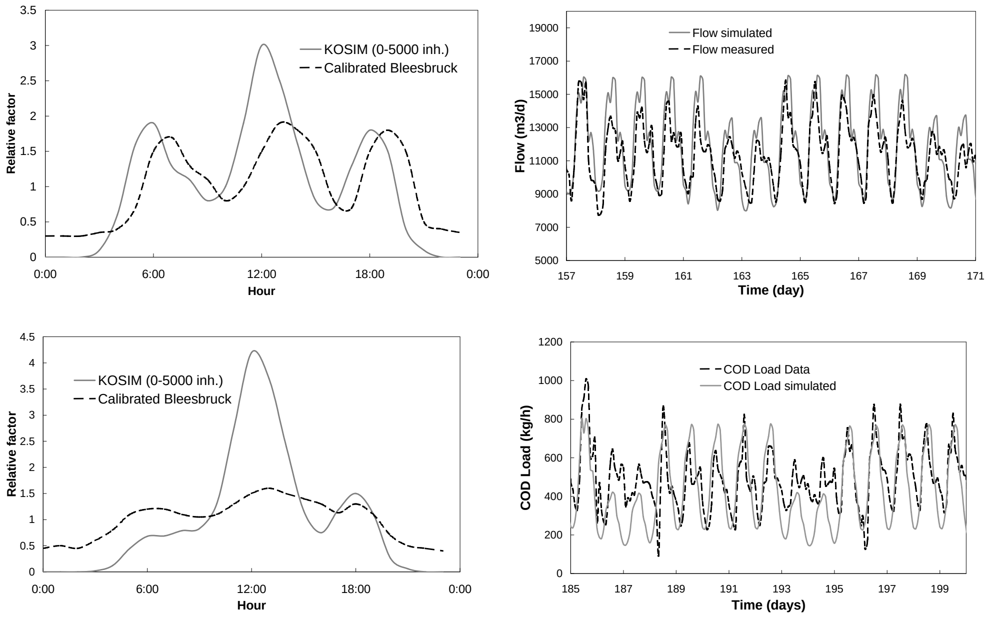


Figure 5.9: Calibrated DWF pattern for flow (top) and pollution (bottom) at the inlet of the Bleesbruck WWTP for 2005 and 2003 respectively. Figures on the left show the default KOSIM pattern (grey) and the calibrated pattern (black) and those on the right represent the simulated (grey) and the measured (black) data.

Once daily flow pattern and their amplitudes were fitted at the measured flows at the WWTP, the yearly mean infiltration was found to be 0.116 l/s/ha. Monthly factors of infiltration could be left as described in the case study Chapter 4.

Within the current KOSIM-WEST model, DWF pollution pattern is the same for all pollution components and COD was chosen to serve as calibration component. Indeed, the ammonia concentration measured at the WWTP after the first AS unit contains the sewage ammonium plus the ammonium in the sludge water stemming from centrifuges operated from 9 to 16 o'clock during week-days, so that the pattern does not correspond to what stems from the sewer (see

Table 5.3: Catchment parameter values in WEST®. Default values are taken from ATV-DVWK (2000).

Dry weather flow parameters			
Parameter	**Default**	**New**	**Units**
$\overline{Q}_{PE}$	180	120	l/d/PE
C(COD$_{part}$)	120	108	g/d/PE
C(COD$_{sol}$)		12	g/d/PE
C(TN)	11		g/d/PE
C(TP)	1.8		g/d/PE
C(NH_4)	9		g/d/PE
C(PO_4)	1.2		g/d/PE
Tourism		165-274	day
Tourism Water Factor	1		-
Tourism Pollution Factor	1	1.1	-
W-E Water Factor	1	0.7	-
W-E Pollution Factor	1	0.5	-
Infiltration	-	0.116	l/s/ha

Figure 4.4 for location of measurement points at the WWTP). Moreover, as some nitrification is taking place in the COD-removing unit of the WWTP (shown in the next section 5.3), ammonia concentrations can be expected to be a little higher before the first biological unit than after it. Orthophosphates are measured after chemical precipitation so that the influent pattern could not correspond to the observation at this location. However, for COD, online measurements of 2005 are very irregular and no pattern could be distinguished, so that data from 2003 were used and the pattern is shown in Figure 5.9.

Once the pattern was determined, *mean pollutant concentrations* could be calibrated and these are listed Table 5.3. Water use per capita was found to be lower than expected, but pollution parameters could be left at their default values.

Figure 5.10 shows collected results during the weeks of the CD4WC measurement campaigns. Online measurements of COD are not displayed due to poor quality as already mentioned above. For ammonium, simulation results are compared to point measurements, as they were taken at the inlet of the plant and do not include the high concentration sludge water as do the online data.

5.2.6.2 WWF calibration

As no measurements were available inside the network to locally calibrate the model, the only reference for the catchment models was the confirmation by the system's operator SIDEN that simulated 'critical' CSOs are indeed known for discharging regularly. Default values (see Appendix A) were used for catchment properties and, after hydraulic calibration of the collector, WWTP inflow data seemed to fit well enough. Results for flows at the WWTP inlet are shown in Figure 5.11 and the bias measure for mass balances $B = 0.99$ and the correlation coefficient $R^2 = 0.70$. It is however observed that in the simulations event flows at the WWTP tend not to be as extended over time as they appear in reality. This was already observed when hydrologic

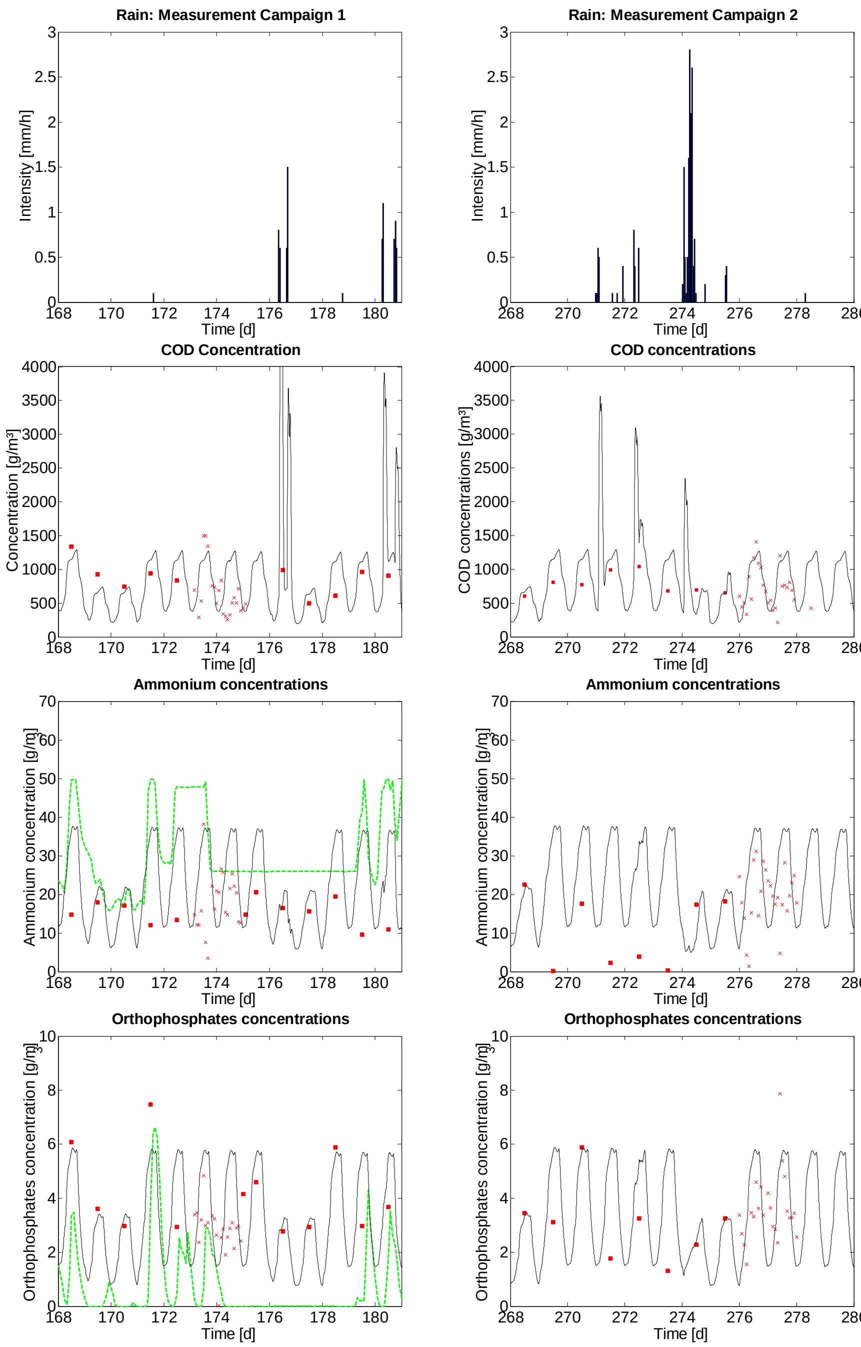


Figure 5.10: Rain data (top). Measurement campaign data (red, ■ = daily mixed samples, × = 2 hour samples) and simulated data (thin black lines) from the sewer to the WWTP for COD, ammonia, and orthophosphates during June and September 2005. Online measured data (dashed green lines) are measured in the first AS unit (including sludge waters and after phosphorus precipitation).

Table 5.4: Sludge water concentrations in mg/l.

$C_S(COD_{part})$	$C_S(TN)$	$C_S(TP)$	$C_S(COD_{sol})$	$C_S(NH_4)$	$C_S(PO_4)$
600	450	25	200	350	20

model results were compared to hydrodynamic simulation results in section 5.2.5 and might be due to inaccuracy of the model structure of the hydrological model. Peaks are much narrower in simulations, probably due to the fact that storage in the collector is not properly used.

Simulation results for ammonium from Figure 5.11 include the added sludge water. Just before the WWTP model, an input file sends $100m^3$/h of sludge water between 9 and 16 o'clock during weekdays. Concentrations of the sludgewater are based on 4 measurements (2 x morning and afternoon) and are given in Table 5.4. Orthophosphates were the most difficult to adjust due to the quite irregular peaks. These partly come in through the delivery of sludges from other treatment plants. The order of magnitude of the incoming P-concentrations seems good, however.

5.2.7 Conclusions

Data was collected to build the Bleesbruck catchment and collector model in the newly implemented KOSIM-WEST. To reduce model complexity, CSOs and their respective drained surfaces have been lumped together. Further model simplification can be done when elaborating simple control strategies by, for example, lumping together wastewater sources without control potential. For more accurate verification of simulation results, measurements from one or more CSOs to calibrate subcatchment models could be useful. In this study, rain events could not be properly calibrated, neither for flow nor for pollution and eventual first flush effects.

The backflow model is a good tool to limit flows in the main collector, giving the modeller an additional parameter in hydrological modelling of sewer systems. It improves mass balances in terms of out- and overflows from the sewer system, and a useful extension could be to add a model component that reflects storage in the system, especially once studies of a system go towards control strategies within the sewer system. However, in case multiple long-term simulations are planned, the model calculations should not become more time-consuming than hydrodynamic model calculations.

Although dry weather flow could be represented in a satisfactory way (as will be confirmed when calibrating the WWTP), modelling of industry wastewater remains difficult. Due to its randomness in time and quantity, constructing input files for pollution and especially industry-related pollution, using a random function based on data (see Ort *et al.* (2005)) can help to better reproduce pollution dynamics. This can be especially important for evaluation of emission loads during a year and for testing in more depth the treatment plant performance during fast dynamic changes in influent pollutant concentrations.

The sewer model results will be used to feed the WWTP model validation and the 8-months calibration. Hence, the calibrated sewer pollutant concentrations can be checked for plausibility through WWTP simulations.

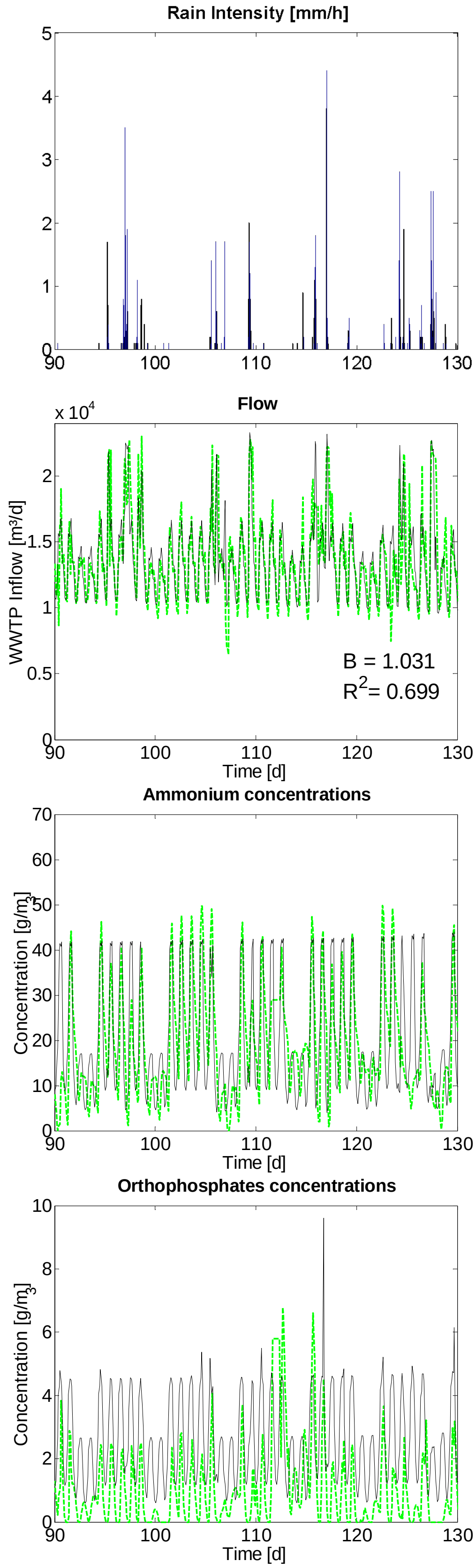


Figure 5.11: Rain data (top). Online measured (dashed green lines) and simulated data (thin black lines) from the sewer to the WWTP for flow, ammonia and phosphate for April 2005 (simulations now include sludge water).

5.3 WWTP Model

In this section, the construction and calibration of the WWTP model is presented. An existing online model is translated into the WEST® software. The biochemical processes are simulated using the ASM2d model, which is calibrated and then validated using two measurement campaigns. In line with the further needs of the model for long-term integrated scenario simulations, the model is then calibrated for 8 months using online data from the WWTP.

5.3.1 ASM model formulation

As already pointed out in Chapter 3, the IWA activated sludge models (Henze *et al.* (2000)) are all included inside the WEST® model base and are the most commonly applied mathematical models for modelling activated sludge compartments of wastewater treatment plants. These models have become widely used and applied as they represent a good compromise between complexity and simplicity, and have shown to make good predictions of the dynamic behaviour of plants.

In WEST®, a set of differential equations describes transport and conversion of components concentrations in the treatment plant unit (see Chapter 3 section 3.4.1). The conversion term of the ith component is given by

$$r_i = \sum_{j=1}^{m} \nu_{ij} \rho_j \tag{5.7}$$

where ν_{ij} is the stoichiometric coefficient and ρ_j is the kinetic process rate for process j. Tables 5.5 and 5.6 list the ASM2d variables and processes applied in this study. Process kinetics are mainly based on Monod kinetics and parameters for stoichiometry and kinetics can be found in (Henze *et al.* (1999)) .

5.3.2 On-site implemented 'LIFE' model calibration

Within an EU Life project (LIFE98 ENV/L/000582, Schosseler *et al.* (2003)), the ASM1 model (Henze *et al.* (1987)) was applied to the here considered treatment plant in order to optimise plant operation. During the same project, the treatment plant was equipped with online measurement devices. Next to temperature, pH and conductivity, total COD is measured after the sand and grease removal unit. Total solids and dissolved oxygen are monitored inside the 2 activated sludge (AS) units and quality parameters like ammonium, nitrate and phosphate are measured at their outflows. The treatment plant layout and its equipment are shown and described in Chapter 4 section 4.1.2.

The original model (see Figure 5.12) was implemented into the SIMBA software (ifak System GmbH, Germany) and model predictions are currently used to regulate the aeration in the second biology (Schosseler *et al.* (2003)). The use of two bioreactors in series had been found appropriate to model the hydraulics of the first AS unit. The clarifiers are modelled using point settlers, as the focus during the Life project was laid on dissolved pollution. An additional point settler 'SurfAB' is added to retrieve the floating sludge that is constantly taken away from the first 2

Table 5.5: State variables of the ASM2d model.

State variable	**Description**
S_{O2}	Dissolved oxygen
S_F	Fermentable, readily biodegradable organic substances
S_A	Fermentation products, considered to be acetate
S_{NH4}	Ammonium plus ammonia nitrogen
S_{NO3}	Nitrate plus nitrite nitrogen
S_{PO4}	Inorganic soluble phosphorus, primarily orthophosphates
S_I	Inert soluble organic material
S_{ALK}	Alkalinity of the wastewater
S_{N2}	Dinitrogen
X_I	Inert particulate organic material
X_S	Slowly biodegradable substrates
X_H	Heterotrophic organisms
X_{PAO}	Phosphate-accumulating organisms
X_{PP}	Poly-phosphate
X_{PHA}	Cell internal storage product of PAOs
X_{AUT}	Nitrifying organisms
X_{TSS}	Total suspended solids
X_{MeOH}	Metal-hydroxides
X_{MeP}	Metal-phosphate

Table 5.6: Processes of the ASM2d model.

	Process
1	Aerobic Hydrolysis
2	Anoxic Hydrolysis
3	Anaerobic Hydrolysis
4	Aerobic growth on S_F
5	Aerobic growth on S_A
6	Anoxic growth on S_A
7	Anoxic growth on S_A, denitrification
8	Fermentation
9	Lysis (heterotrophs)
10	Storage of X_{PHA}
11	Aerobic storage of X_{PP}
12	Anoxic storage of X_{PP}
13	Aerobic growth of X_{PAO}
14	Anoxic growth of X_{PAO}
15	Lysis of X_{PAO}
16	Lysis of X_{PP}
17	Lysis of X_{PHA}
18	Aerobic growth of X_{PAO}
19	Lysis (nitrifiers)
20	Precipitation
21	Redissolution

clarifiers. The weir between the two biological units is modelled using a data-derived function relating the inflow of the WWTP to the overflow.

The influent to the treatment plant is fractionated into ASM1 variables according to 2 measurement campaigns conducted during the project and fractionation values f_{trans} are given in Table 5.7.

Table 5.7: Fractionation of the influent composition.

Measured variable	**ASM1 variable**	f_{trans}	**given value**	**ASM2d variable**	f_{trans}	**given value**	**unit**
COD soluble	S_S	0.75		S_F	0.48		$gCOD/m^3$
				S_A	0.32		$gCOD/m^3$
	S_I	0.25		S_I	0.2		$gCOD/m^3$
COD part. = COD total - COD soluble	X_S	0.77		X_S	0.8		$gCOD/m^3$
	X_I	≈0.13		X_I	≈0.04		$gCOD/m^3$
	X_{BH}	0.1		X_H	0.16		$gCOD/m^3$
	X_{BA}	0.001		X_{TSS}	0.75		$gTSS/m^3$
	X_P	0.001					$gCOD/m^3$
NH4-N	S_{NH4}	1		S_{NH4}	1		gN/m^3
Norg = TKN - NH4-N	S_{ND}	≈0.15					gN/m^3
	X_{ND}	≈0.85					gN/m^3
PO4-P				S_{PO4}	1		gP/m^3
	S_O		1	S_O		0.01	$-gO_2/m^3$
	S_{NO3}		0.1	S_{NO3}		0.01	gN/m^3
	S_{ALK}		5	S_{ALK}		5	mol HCO_3^-/m^3
				S_{N2}		0.01	gN/m^3
				X_{AUT}		0.01	$gCOD/m^3$
				X_{PAO}		0.01	$gCOD/m^3$
				X_{PHA}		0.01	$gCOD/m^3$
				X_{PP}		0.01	gP/m^3
				X_{MeP}		0.01	$gMePO_4/m^3$
				X_{MeOH}		0.01	$gMe(OH)_3/m^3$

The on-site SIMBA model has been translated into WEST® (see Figure 5.13) during the here presented project so that the integrated sewer-WWTP-river model can run be in one software. For this integrated study, due to the presence of phosphorus removal by chemical precipitation and the interest in phosphorus regarding emissions and immission concentrations in the river, the ASM2d model (Henze *et al.* (1999)) has been chosen to replace the used ASM1 model. Indeed, next to biological processes for chemical oxygen demand (COD) and nitrogen removal, ASM2d includes variables and processes to model biological and chemical phosphorus removal.

Differences between the SIMBA and the WEST® models are:

- Conversions of components are now modelled with ASM2d instead of ASM1 (see section 5.3.1), due to the presence of chemical phosphorus removal at the plant and the importance of phosphorus in the assessment procedure of the IUWS, and also to be in line with other parallel projects where ASM2d was used,

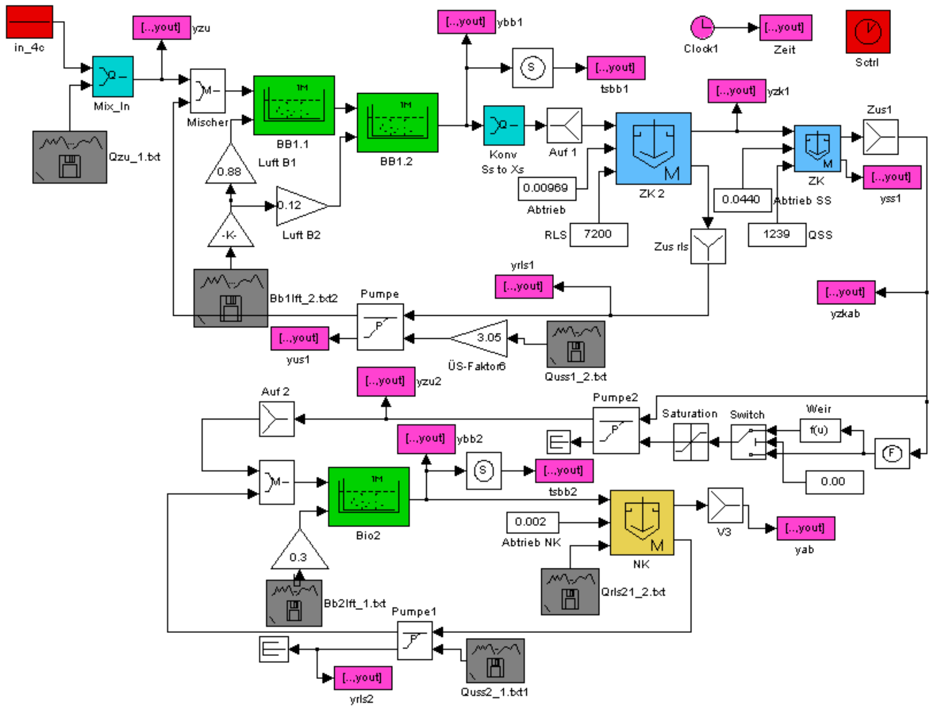


Figure 5.12: Model of the Bleesbruck WWTP in SIMBA.

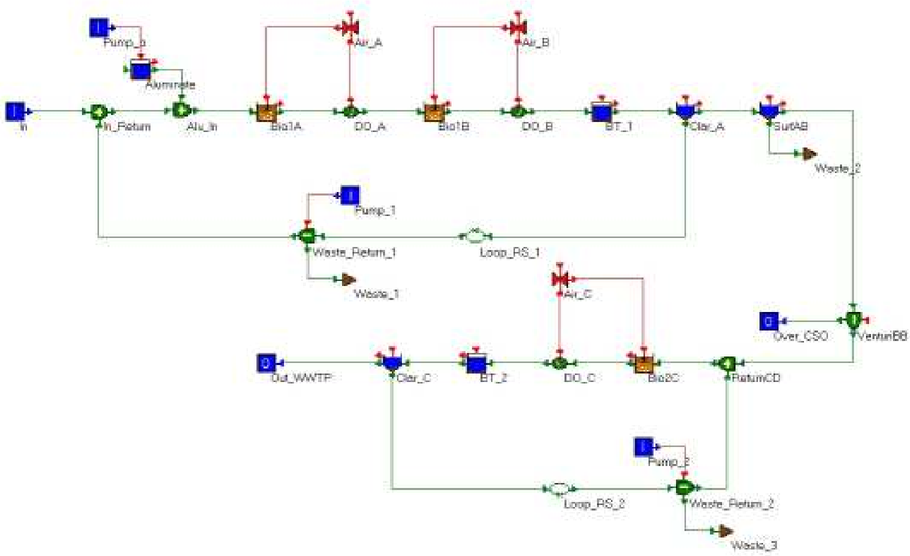


Figure 5.13: Model of the Bleesbruck WWTP in WEST®.

- whereas in SIMBA, one parameter set is used for the two biological compartments, in WEST®, two different sets can and will be used,
- in the 'new' model configuration, PI controllers for aeration were implemented into WEST® , the return sludge flows are still specified in data files as in the SIMBA model,
- chemical phosphorus removal is taking place as aluminium oxide is added to the first biology tank, based on orthophosphate concentrations that are measured after Biology 1 and that are exceeding 3 mg/l. Pumping data of aluminium oxide liquid are used,

To check whether the newly implemented model version corresponded to the SIMBA model configuration, and before ASM2d parameters were calibrated, water flows of the SIMBA and the WEST® model were checked and confirmed to produce the same results. The WEST® model version was calibrated on a one-week measurement campaign from June 2001, which had also been used for the SIMBA model calibration.

Although influent composition of COD had been determined from measurements during the EU Life project, two ways of calibration were tested for the WEST® model version: (a) changing kinetic parameters in the Peterson matrix, or (b) changing the influent file composition. First point of concern was the sludge balance and applying (b) to decrease sludge concentrations without changing default kinetic parameters did not allow to fit dissolved oxygen concentrations nor for satisfying simulation of nitrification in the first biology. It came out that both, small changes in influent composition with respect to previous ASM1 fractionation (see Table 5.7) as well as tuning of some of the kinetic parameters in the first AS unit was necessary to produce satisfying results. Table 5.8 shows default and new values for four critical parameters in the first biology. Indeed, only changing with b_H (already doubled) could not achieve the desired results for sludge balance, i.e. reduce sludge concentrations, so that X_I had to be decreased (sludge wastage is fixed from pumping data). It was expected that a correct simulation of the oxygen dynamics would allow for good further kinetic parameter calibration so that parameters of PI oxygen controllers were set such that oxygen concentration dynamics in the activated sludge tanks matched the measured data.

Table 5.8: Default and new parameter values for the first biology.

Parameter		**Default**	**New**	**Units**
Growth rate of heterotrophs	μ_H	6.0	3.0	d^{-1}
Rate for lysis and decay of heterotrophs	b_H	0.6	1.2	d^{-1}
Growth rate of autotrophs	μ_{AUT}	1.0	1.5	d^{-1}
Reduction factor for denitrification	η_{NO_3}	0.8	1.0	–

Figures 5.14 and 5.15 show the results in order as proceeded for calibration: together with setting the aeration control parameters such that the dissolved oxygen concentration behaviour could be reproduced by the model, the solids balance was checked, and then, then COD removal fitted. Next, the model was adjusted to fit ammonia and nitrate concentrations and, last, the phosphorus concentrations were checked.

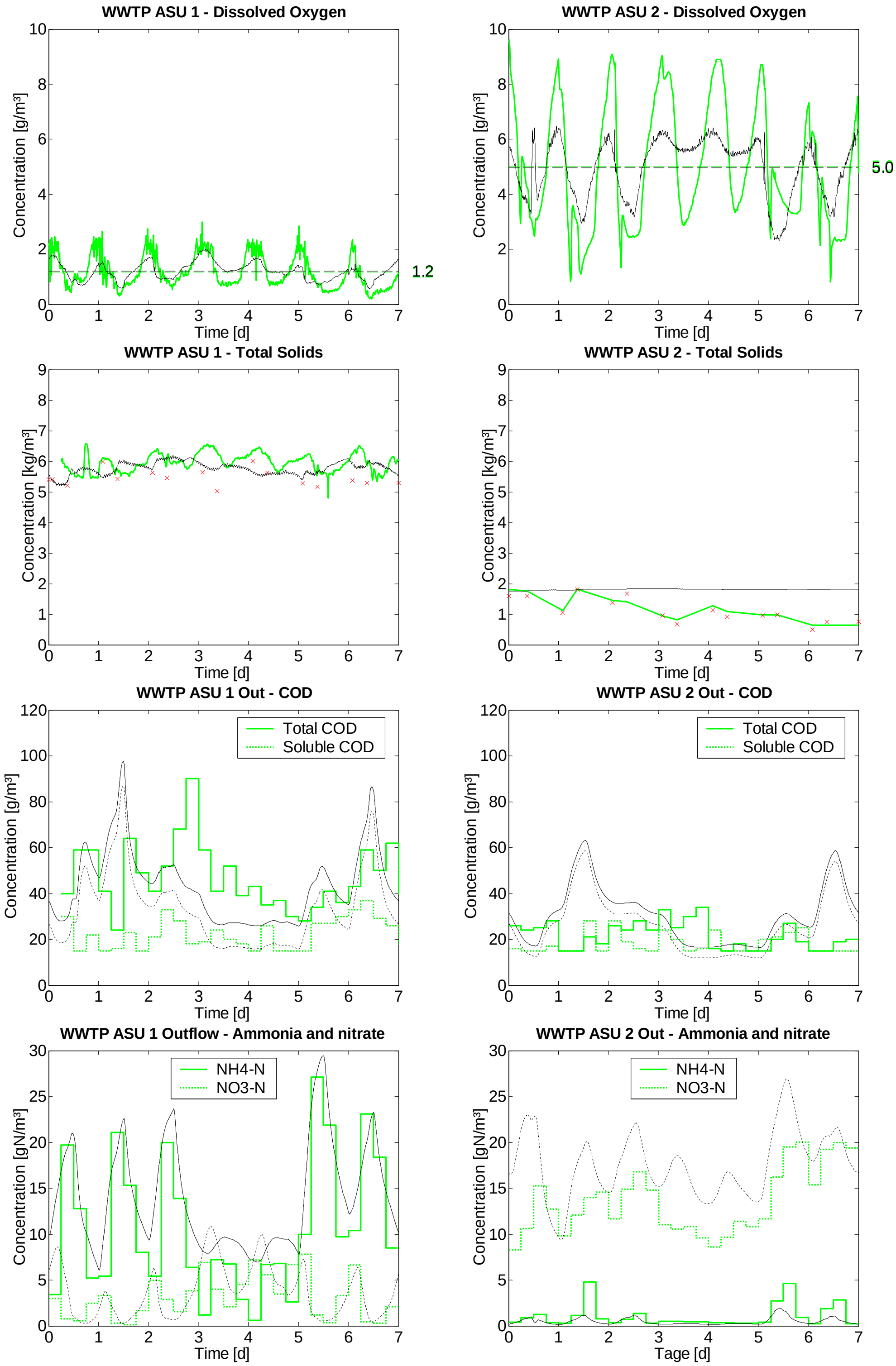


Figure 5.14: Simulation results in WEST® (black lines) versus online measurements (green lines) and manual point measurements (red ×).

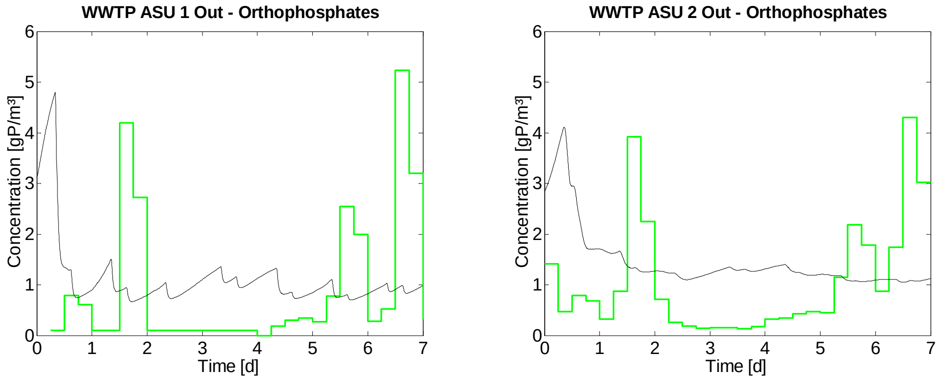


Figure 5.15: Simulation results (black lines) in WEST® versus online measurements (green lines).

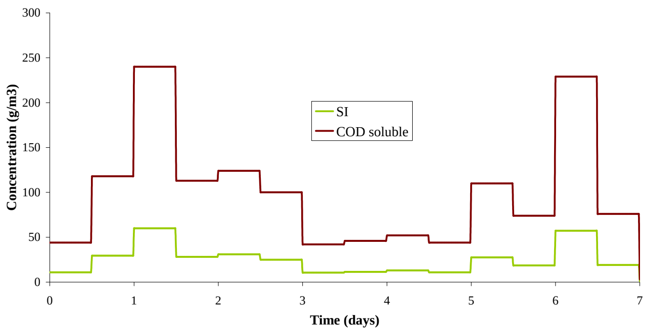


Figure 5.16: Influent measurements for soluble COD; fractions are inert non-biodegradable organics S_I, readily biodegradable substrate S_F and fermentation products S_A.

It can be concluded that results for oxygen represent the dynamics well enough in both AS units, even if in AS unit 2, simulated DO concentration peaks do not reach measured ones. However will such high levels not effect biological activity. The sludge levels could not follow measurements exactly in the second AS unit, although actual pumping data was used. The measured low TSS concentrations in the second biology cannot be explained apart from the fact that underdimensioning of the units causes low residence time and therefore low sludge age. Figure 5.16 depicts the measured influent soluble COD and the derived inert fraction S_I. The two-peak appearance is visible throughout the simulated biological units and explains the pattern for COD concentrations in the calibration results. The decrease of the growth rate μ_H along with an increase of death rate b_H for heterotrophs is supposed to represent the low sludge age (3-5 days) of the first AS unit. Autotrophs were given an increased growth rate μ_{AUT} so that we can reproduce some nitrification and the reduction factor η_{NO_3} for denitrification was increased to its maximum value to allow for some denitrification. The latter might be interpreted by the fact that aeration in the sludge unit is not very uniform, bringing about anaerobic zones where denitrification can take place. In the first biology, especially ammonia and nitrates show a good fit within measurement errors.

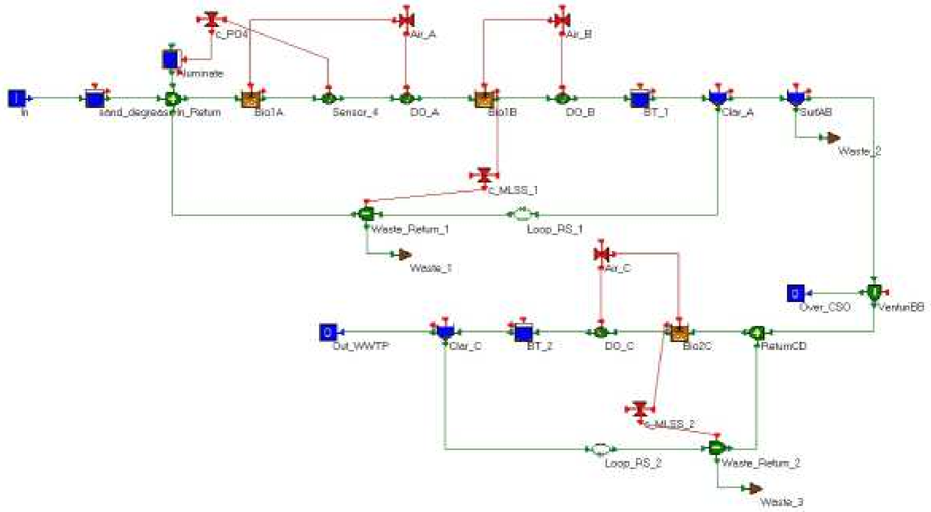


Figure 5.17: Model configuration in WEST® as used in the future of this project as compared to Figure 5.13.

In the second biology, the parameters were left with ASM2d default values (Henze *et al.* (1999)). Results for nitrates are not totally satisfactory, as they are a little high compared to measurements. However, before changing parameter values, it was decided to check the current model settings in the 'CD4WC' model validation presented below. Results for orthophosphates are shown in Figure 5.15. Simulations do not fit measurements at all: It seems that some of the peaks cannot be eliminated by the implemented control system at the WWTP. This weakness of the control at the WWTP is related to the fact that aluminate pumping into AS unit 1 happens only once the orthophosphates concentration rises at the outlet of the AS unit. The more regular phosphorus removal in the model is probably due to the immediate mixing of the precipitation liquid with the bulk AS in the model.

5.3.3 'CD4WC' model validation

The 'CD4WC' model calibration is both a sort of validation for the 'LIFE' calibration and a preparation for the 8-months calibration that is to follow. The adapted model version for this validation is shown in Figure 5.17 and 'new' features include

- a volume for the sand and grease removal unit to include retention after the influent,
- dynamic data from simulation of the calibrated sewer system model are used as input to the WWTP model, (see section 5.2.6),
- controllers instead of pumping data for return and waste sludge are implemented. Their

settings are also data inspired, but as we want to use the model for testing different scenarios with the integrated model on the long-term, it is useful that the model does not depend on the data,

- the ASM2d model becomes temperature dependent, i.e. kinetic parameter values of the last calibration are changed using

$$K_T = K_{T_{ref}} \cdot e^{\beta_K (T - T_{ref})} \tag{5.8}$$

where $K_{T_{ref}}$ is the value of the parameter at a reference temperature T_{ref} (20°C), β_K is the temperature correction factor for parameter K. T was calculated as an average temperature measured on-site over the simulated 2 weeks.

- for chemical phosphorus precipitation, an on/off controller is used to pump the aluminium oxide into the AS unit model on phosphate measurements in the first tank,

Results for AS units 1 and 2 using 2 measurement campaigns in spring and autumn 2005 (see Chapter 4 section 4.2) are shown in Figures 5.18 and 5.19. Next to the on-line monitoring data, the data from the CD4WC summer measurement campaign for COD, ammonia, nitrates and orthophosphates are used to compare to simulations (see Chapter 4 section 4.2 for details on the available data and the measurement campaign). Apart from changes due to temperature, kinetic and stoichiometric parameter values were not modified from the previous calibration.

Oxygen dynamics in the 2 AS units are similar for measured and simulated data. Especially for ammonium and nitrates, the validation lies within measurement errors. As the WWTP model influent data are simulated data from the sewer model and not real measurements, some of the measured peaks at the outlet might not have been simulated and vice versa. Note that the online ammonium sensor cannot measure values above 20 mg/l. It seems that the measurements of particulate COD at the outflow of the WWPT are partly higher than those simulated. This can be adjusted by decreasing the settling capacity of the secondary clarifiers for example. However, more measurements at the WWTP would be useful, before new adjustments are done. For phosphate precipitation modelling, it was impossible to find the appropriate controller model and parameters to match the measured number and shape of peaks.

5.3.4 8-months calibration

The calibration was performed for the months of March to October 2005 using the online data and the CD4WC autumn measurement campaign, as from 15 September online data is disrupted due to ongoing construction works at the WWTP. An influent file for the WWTP model was generated for these 8 months by the sewer model, both for water quantity and water quality components. The main reason to exclude the winter period is due to the difficulty of modelling the sewer system during that period. Indeed, as mentioned in section 5.2.6, Sûre water is intruding into the system and there is no exact data available neither about the locations of the intrusion nor about the quantity coming in. Additional features to this model version are:

- online monitored temperature data for 2005 is fed into the model,

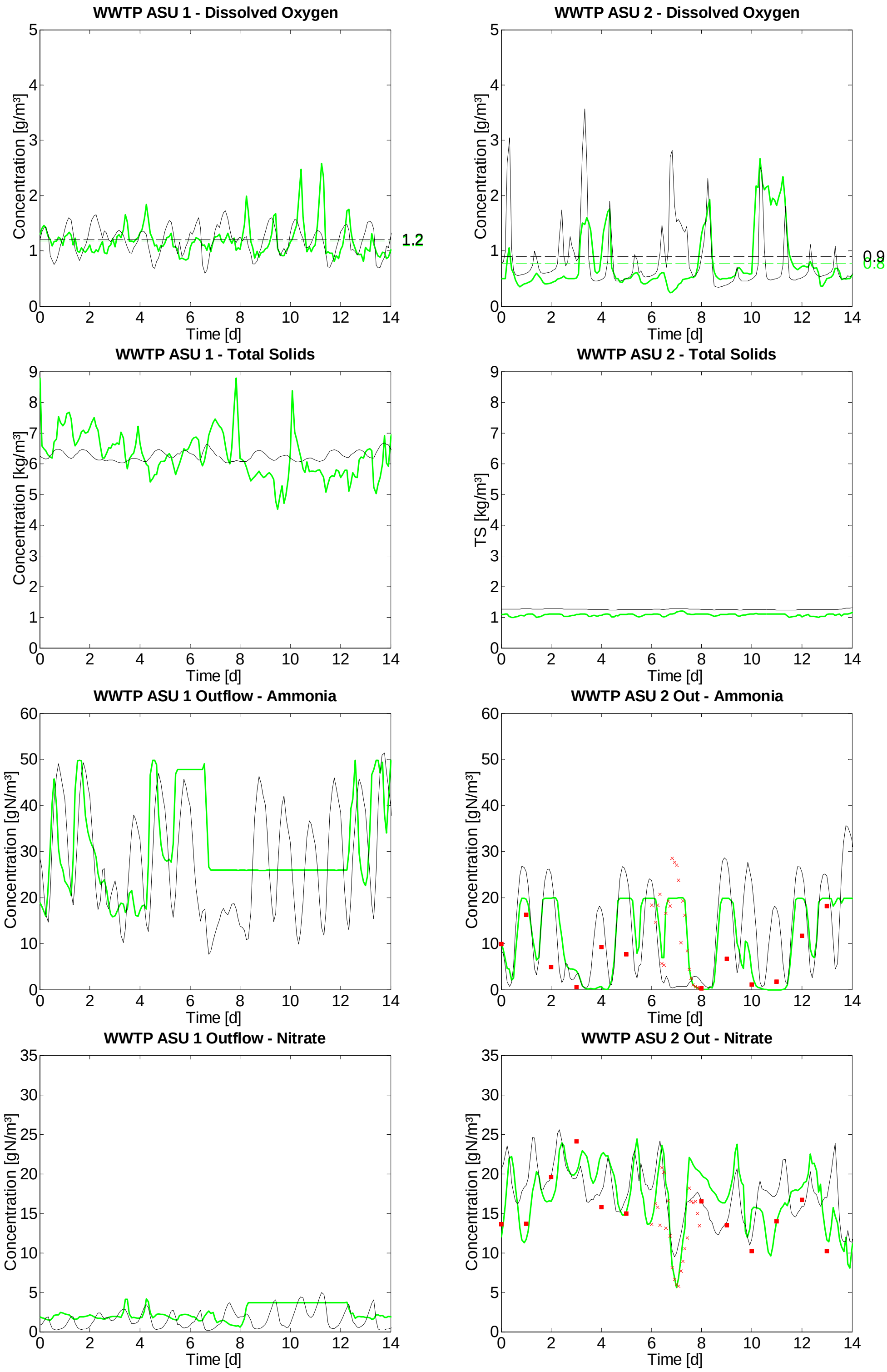


Figure 5.18: Simulation results in WEST® (black lines) versus online measurements (green lines) in AS unit 1 and 2, during the 'CD4WC' spring measurement campaign. Also included are manual point measurements (red ■ for daily mixed samples, red × for 2-hourly grab samples.

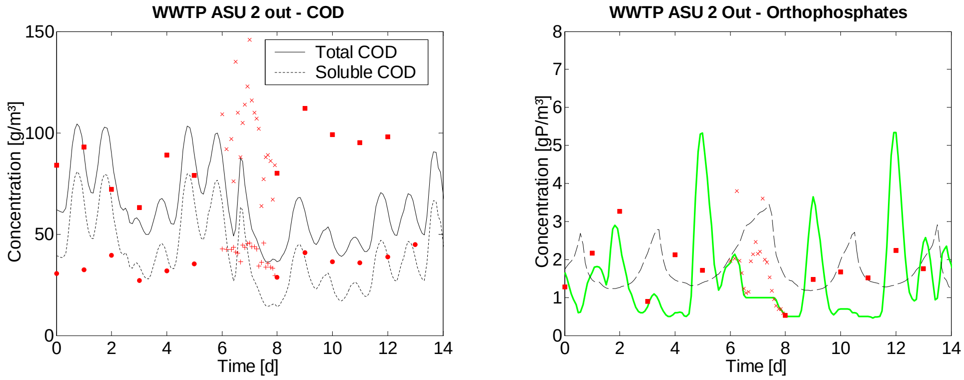


Figure 5.19: Simulation results in WEST® (black lines) versus online measurements (green lines) at the outflow of the WWTP, during the 'CD4WC' spring measurement campaign. Also included are manual point measurements (red ■, • for daily mixed samples, red ×, + for 2-hourly grab samples for total and dissolved COD respectively.

- a supplementary file was generated to represent external sludges, content of sceptic tanks, etc. arriving at the WWTP every day. It was constructed using a random number generator mimiquing the operators experience on frequency of arrivals and ranges of volumes and concentrations for COD, nitrogen and phosphorus. The additional load is relatively small but should nevertheless represent the short time shock loads the plant is subjected to.

- difficulties arose when running the simulations with the same settings as for the 'CD4WC' calibration. This comes from the fact that the measured TS concentrations for both the AS units 1 and 2 were not the same over the whole simulation period (lower in winter than in summer). Therefore sludge concentration controllers acting on the sludge wastage rate were implemented allowing to define two set points, one for winter and one for summer periods.

None of the parameter values from the initial calibration were changed. First results showed that at low temperatures the model could not reproduce desired concentrations for ammonium and nitrate. Indeed, nitrification activity is much lower in the colder periods of the year and when using the ASM models, temperatures are expected to be in the range between 10 to 25°C (Henze *et al.* (1999)). For the months of March and April, the temperature at the WWTP is near that lower limit. Similar difficulties with WWTP simulations have been encountered by Achleitner (2006). In our case, oxygen control settings were adjusted for winter and early spring such that through reducing the proportional gain and hence system response, reduced nitrification could be imitated.

Simulation results versus measurements are shown in Figure 5.20 for the whole simulation period, Figure 5.21 for April 2005 and Figure 5.22 for September 2005, containing data from the 'CD4WC' autumn measurement campaign for COD, ammonia, nitrates and orthophosphates (see Chapter 4 for details on the available data and the measurement campaign).

The higher ammonium and nitrate concentrations in summer suggested by Figure 5.20 are due to lower infiltration of parasite waters into the sewer network. Measured nitrate concentrations

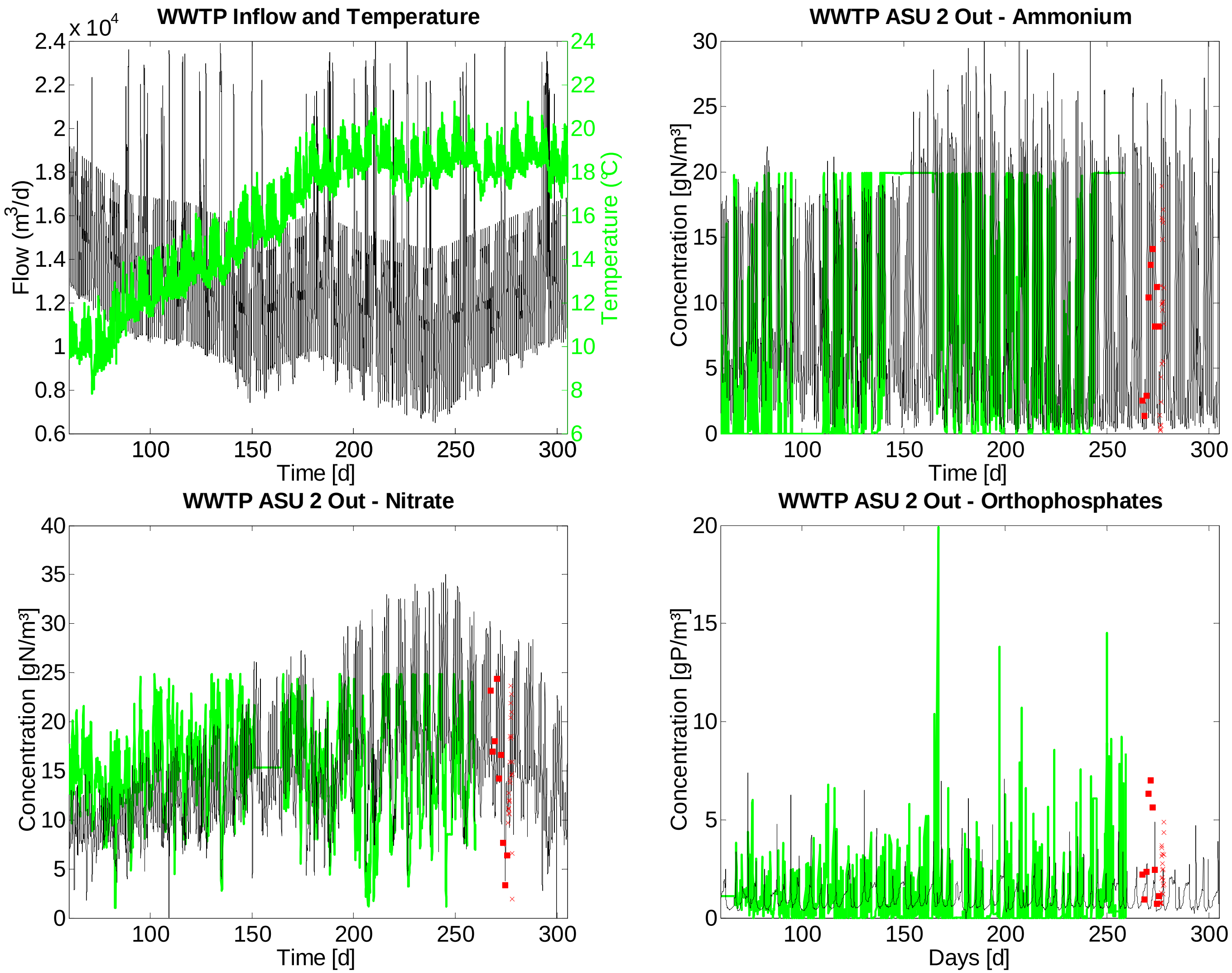


Figure 5.20: 8 month of simulation results (black lines) in WEST® versus online measurements (green lines) at the outflow of the WWTP. Note that online sensors do not measure above concentrations of 20 mg/l.

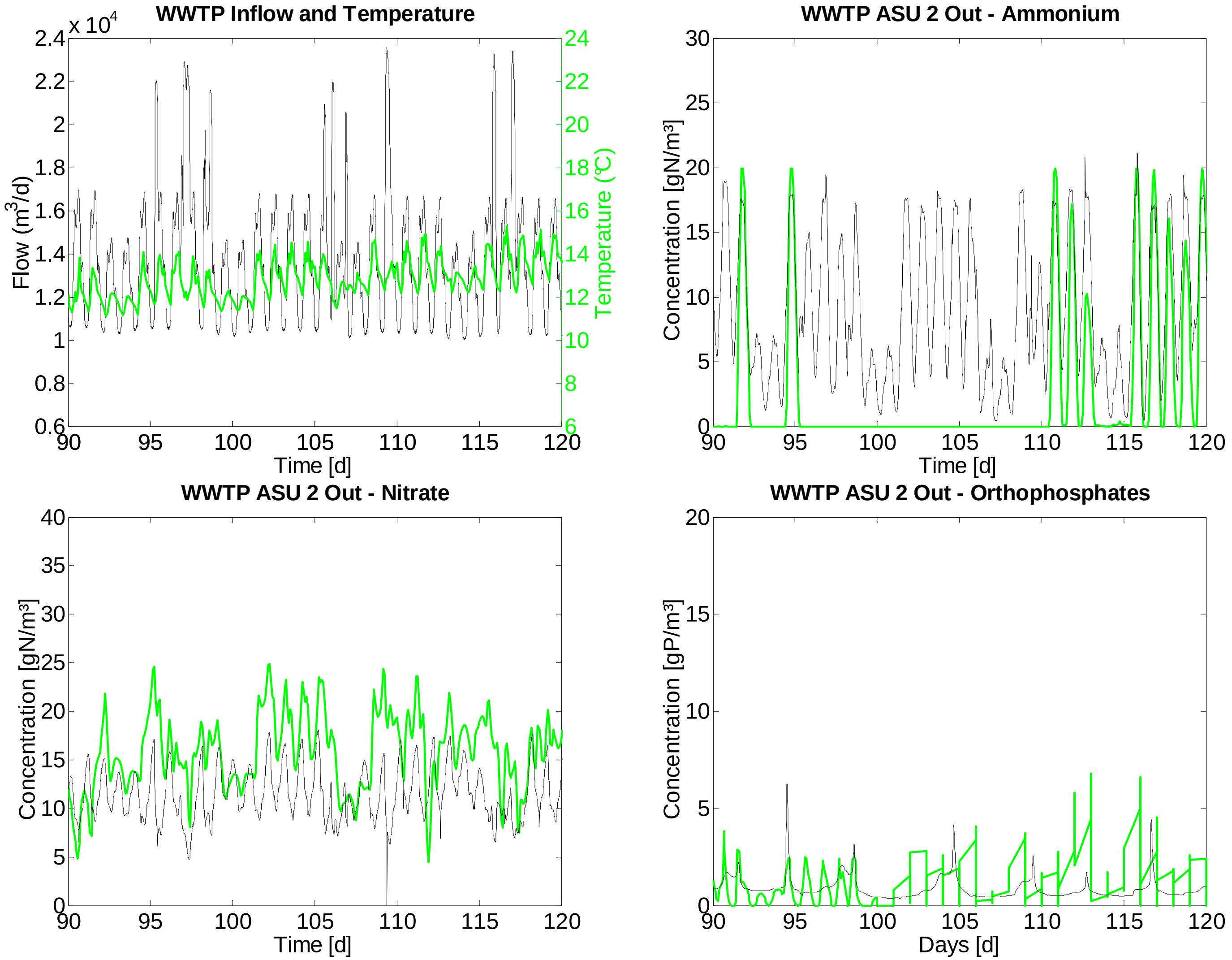


Figure 5.21: April simulation results (black lines) in WEST® versus online measurements (green lines) at the outflow of the WWTP. Note that online sensors do not measure above concentrations of 20 mg/l.

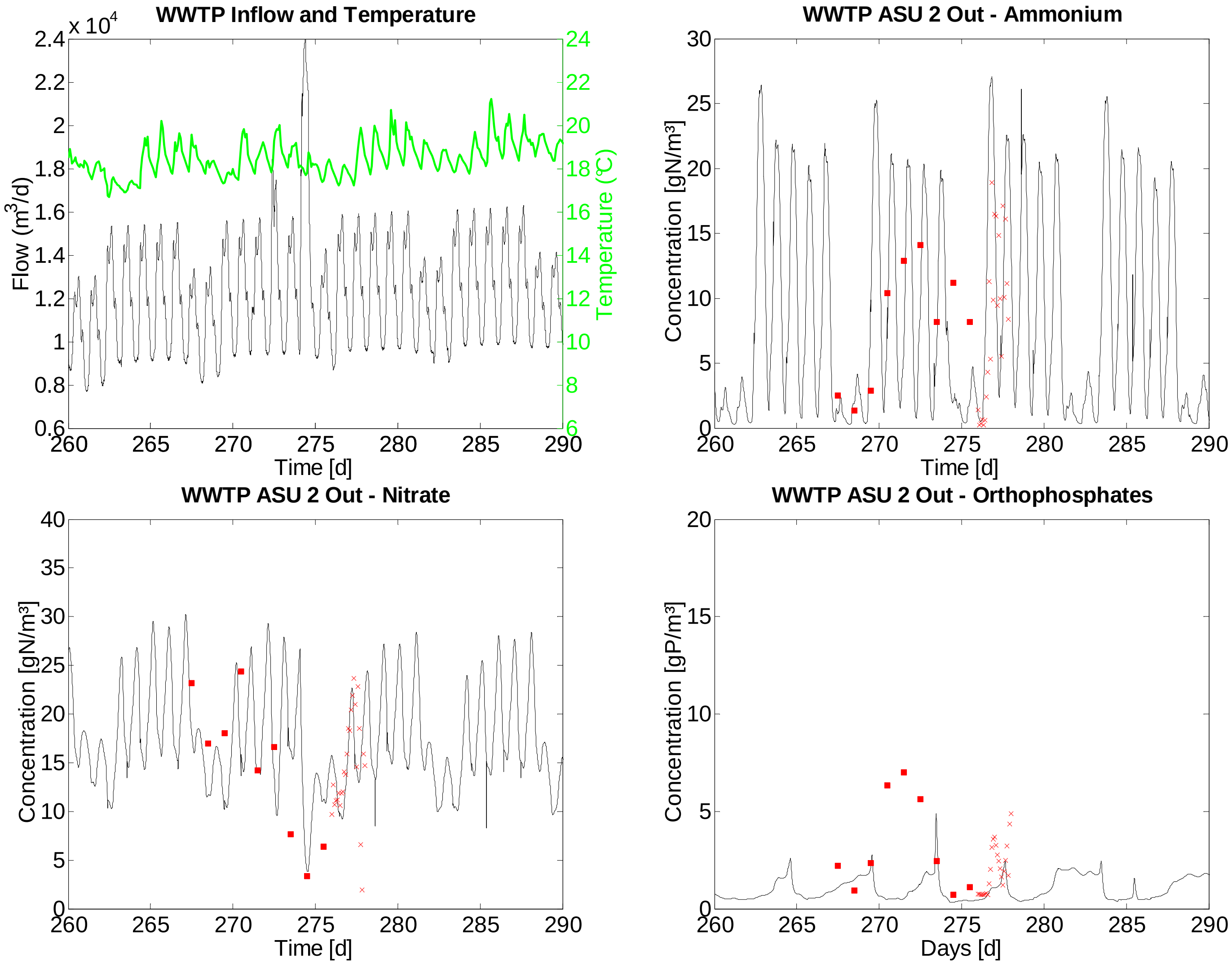


Figure 5.22: September simulation results (black lines) in WEST® versus autumn manual point measurements (red □, ● for daily mixed samples, red ×, + for 2-hourly grab samples. Note that online sensors did not work during that period.

in winter are still slightly higher than those simulated, but tuning parameters did not improve these results. This suggests not enough nitrification or too much denitrification considering that ammonium values are within measured ranges. However, an extra measurement campaign would be required for the winter season to fill the knowledge gaps for further model development. The comparison with sampling data from the 'CD4WC' measurement campaign in Figure 5.22 gives good results especially for the 2-hourly grab samples. The orthophosphates concentration peaks could not be matched with data, which is thought to be due to the unusual control onsite.

5.3.5 Conclusions

From an already available model in SIMBA, a new WWTP model was constructed in WEST®. The aim of this translation was to have the model in the same software than models for sewer and river. Calibrations were performed with data using measurement campaigns' data and online monitoring data. The aim of the calibration was to have a model reproducing the plant's behaviour over the spring, summer and autumn periods so that integrated scenario simulations can be tested.

The sewer model results proved to produce realistic incoming data for the WWTP model. Even though individual peaks and lows might be omitted, overall tendencies and magnitudes are represented within measurement errors. Difficulty for the 8-months calibration arose regarding reduced bacteria activity in winter and early spring. The ASM model seems to perform less well at low temperatures. Unfortunately, phosphorus concentrations could only be simulated to the order of magnitude.

Compared to the other systems, more data was existing for the WWTP, allowing for good calibration, especially for ammonium and nitrates. Certainly more data would be required regarding the COD fractionation at the influent, the settling capacity in the settlers or even for determination of kinetic parameters using respirometry (Vanrolleghem *et al.* (1999a)). To discuss the influence of the WWTP in terms of particulate COD could certainly be improved by using a more elaborate model for the settlers. It was however estimated that the system's behaviour is sufficiently well represented by this model so that it can be used in a comparative scenario analysis.

5.4 River Model

This section presents the models used for river hydraulics and river water quality. It explains how they are applied to model the rivers Attert, Alzette and Sûre of the 'Bleesbruck' case study. System boundary conditions are defined and calibration of hydraulics and water processes is presented and commented.

5.4.1 RWQM model formulation

Good overview on the different processes in river waters and how to model them can be found a.o. in Gromiec *et al.* (1983) or Chapra (1997). Commonly available standard in-stream water quality model softwares are QUAL2K (US Environmental Protection Agency (EPA), Chapra *et al.* (2006)), WASP7 (EPA, US), ISIS (Wallingford Software, UK), MIKE 11 (DHI Software, DK), AQUASIM (Reichert (1998)), DUFLOW (STOWA, NL). A comprehensive review on some of these can be found in Cox (2003) and Rauch *et al.* (1998b).

The river water quality model RWQM1 (Reichert *et al.* (2001)) was developed within an IWA Task Force with the aim of being compatible with the existing Activated Sludge Models (ASM1, ASM2 and ASM3, Henze *et al.* (2000)). Hence, the model state variables are of the same kind as in the ASM models, characterising organic material, organisms (bacteria, algae and consumers), nutrients, oxygen and inorganic materials. The main development however is that the elemental composition of the model components in terms of C, H, O, N and P (and X, summarising all the rest) gives a rigorous theoretical base to the model.

Previous case studies in which the model has been applied so far are a.o. described in Martin *et al.* (2006), Deksissa (2004), Meirlaen *et al.* (2001), Reichert *et al.* (2001), Reichert (2000). The model structure allows to easily exclude processes of less importance to the study or to add additional relevant ones; i.e. again the purpose of study determines on the processes to be included in the model and the required data set. Hence river basin models will certainly require less detail than a quality model applied in the vicinity of a point source. Holvoet (2006) has extended the model to include pesticide modelling.

5.4.1.1 Hydraulics and Transport

Assuming that longitudinal accelerations are more significant than transverse or vertical ones, river hydraulics can be modelled in 1-D. This can be done using the full St.Venant equations for energy and momentum conservation (see Equations 3.1 and 3.2), or, through simplification of the equations by ignoring the acceleration or pressure terms.

In this case, the hydraulic routing will be simplified to a model of Continuously Stirred Tank Reactors (CSTRs) in series. It is the same model as was used for runoff concentrations of rainwater and for subsequent sewer transport in collectors (see section 3.3). Each tank receives the output from the previous tank, and the contents of the tanks are supposed to be instantaneously mixed. Equations 5.9 and 5.10 assure for mass conservation on one hand and linear behaviour of the outflow with respect to the volume of water contained in the tank on the other hand:

$$\frac{dV}{dt} = Q_{in}(t) - Q_{out}(t) \tag{5.9}$$

$$Q_{out}(t) \quad = \quad \frac{1}{k_n} V(t) \tag{5.10}$$

where V is the volume, Q_{in} and Q_{out} are the in- and outflow of the tank and k_n is the linear reservoir constant of tank n. The advantages of such a simplified conceptual model are partly discussed in Chapter 2 section 2.3.3. As the model will not simulate any backwater effects, a model correction could be implemented similar to the one for the collector system (see 5.2). Meirlaen (2002) describes a procedure for calibrating the river hydraulics in tanks-in-series by using a complex hydraulic model in Aquasim (Reichert (1998)). It was assumed that such supplementary feature was unnecessary for the time being, as focus will lie on river water quality at low flows.

The water quality submodel is integrated into the above simple hydrological model, so that concentrations in a mixed variable volume can be expressed by:

$$\frac{d(V(t)\mathbf{c}(t))}{dt} = Q_{in}(t)\mathbf{c}_{in}(t) - Q_{out}(t)\mathbf{c}(t) - V(t)\mathbf{r}(\mathbf{c}(t), \mathbf{p}) \tag{5.11}$$

where Q_{in} and Q_{out} are factors for flow, $\mathbf{c}$ and $\mathbf{c}_{in}$ are the component concentration vectors inside the considered tank and in the inflow to the tank respectively. $\mathbf{r}$ is the conversion rate vector, which is a function of the actual concentrations $\mathbf{c}$ and the model parameters $\mathbf{p}$.

Particulate material is modelled in the same way as solute material and sedimentation is not taken into account. Martin *et al.* (2006) present the use of a sedimentation factor f, thereby separating hydraulic from solids retention time.

The dispersion is typically decreased by increasing the number of tanks and there exists an optimum to best represent pollutant propagation (Meirlaen (2002)). In the end, the model accuracy is determined as a compromise between accuracy and calculation time, depending on the model purpose.

5.4.1.2 Simplified RWQM1

For the biochemical conversions, a simplified version of the IWA River Water Quality Model (RWQM1, Reichert *et al.* (2001)) is used. This model version was also applied in a study for the river Nete (Belgium, Ghermandi (2004)) and in Benedetti (2006).

The full RWQM1 contains $n = 24$ state variable and $m = 23$ processes. Like the ASM models (see section 5.3.1), the model can be represented in a matrix form, giving an overview of which variables are subject to what processes, each reaction term in equation 5.11 can be expressed as:

$$r_i = \sum_{j=1}^{m} \nu_{ij}\rho_j \tag{5.12}$$

where r_i is the reaction term for the ith component, ν_{ij} is the stoichiometric coefficient and ρ_j is the kinetic process rate expression for process j, typically based on Monod kinetics.

The model can be simplified to only include the processes of importance to the given application. In this case all processes related to the state variable 'consumers' were eliminated (as no data was available) and processes describing any pH-related reactions have been omitted too as

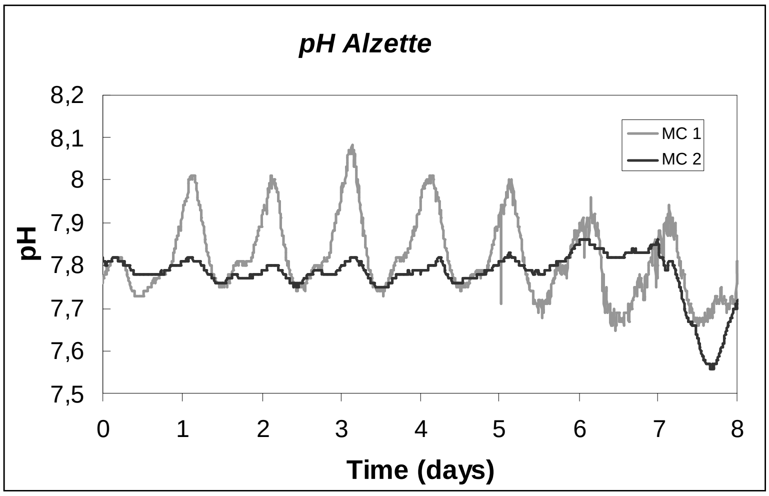


Figure 5.23: Data from measurement campaigns, where one of the probes in the Alzette was equipped with a pH sensor. MC1 = measurement campaign 1 in spring 2005; MC2 = measurement campaign 2 in autumn 2005.

the pH is known to be constant in the river stretches over the simulation period. The Alzette river basin is dominated by carbonate rich sedimentary rocks and groundwater contributions are mostly from the Luxembourg Sandstone, a carbonate rich substrate. Hence the Alzette is well buffered with high bicarbonate levels and pH is weakly influenced by photosynthesis and respiration (see Figure 5.23) . The river Sûre has a rather low alkalinity due to the devonian schist rock background from its catchment, but improves its buffering capacity at the confluence of the Alzette in Ettelbruck. The investigated reaches are therefore rather stable with respect to natural pH variation, although algal presence will influence pH with consumption and production of CO2. However ammonium dissociation is not expected to be impacted in a significant matter by these variations.

Hydrolysis, bacterial and algal growth are functions of water temperature. A heat balance model was implemented in the river model to consider the effect of atmospheric changes on water temperature. Based on the model concept (see Figure 5.24) of Talati & Strenstrom (1990), the model includes the effects of solar radiation, atmospheric radiation, surface evaporation and surface convection as a function of water surface and time series of daily incoming water temperature, radiation intensity, air temperature, wind speed, relative humidity. An addition was made to include the contribution of base flow coming from groundwater, characterised by quantity and temperature of incoming groundwater. In the actual implementation of the model, Hp, Hb and Htw were neglected since they are of minor relative importance (Gillot & Vanrolleghem (2003)).

The temperature dependency of kinetic parameters like heterotrophic growth rate, nitrifier growth rate, algae growth rate and hydrolysis is expressed by:

$$K_T = K_{T_{ref}} \cdot e^{\beta_K (T - T_{ref})} \tag{5.13}$$

Table 5.9: State variables in the simplified version of RWQM1.

State variable	Description
S_I	Inert soluble COD
S_S	Readily biodegradable soluble COD
S_{O2}	Dissolved oxygen
S_{NO2}	Nitrite nitrogen
S_{NO3}	Nitrate nitrogen
S_{PO}	Phosphate
S_{NH}	Ammonia nitrogen
S_{ALK}	Alkalinity
X_I	Particulate inert COD
X_S	Particulate organic matter
X_H	Heterotrophic biomass
X_{N1}	First stage nitrifying bacteria
X_{N2}	Second stage nitrifying bacteria
X_{ALG}	Algae and macrophytes
X_P	Phosphate adsorbed to particles
X_{II}	Particulate inorganic matter

Table 5.10: Processes in the simplified version of RWQM1.

Process	
1	Aerobic growth of heterotrophs with ammonia
2	Aerobic growth of heterotrophs with nitrate
3	Anoxic growth of heterotrophs with nitrate
4	Anoxic growth of heterotrophs with nitrite
5	Aerobic endogenous respiration of heterotrophs
6	Anoxic endogenous respiration of heterotrophs
7	Growth of first stage nitrifiers
8	Aerobic endogenous respiration of first stage nitrifiers
9	Growth of second stage nitrifiers
10	Aerobic endogenous respiration of second stage nitrifiers
11	Growth of algae with ammonia
12	Growth of algae with nitrate
13	Aerobic respiration of algae
14	Death of algae
15	Hydrolysis
16	Aeration

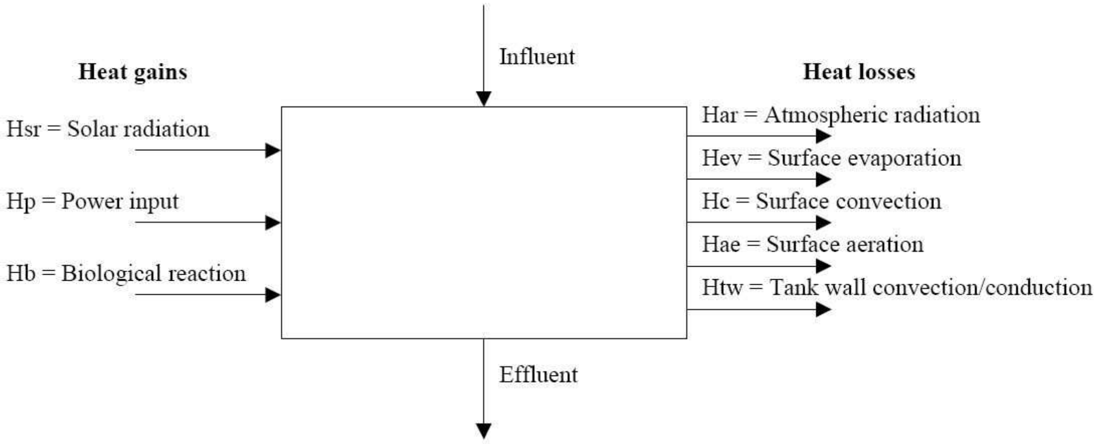


Figure 5.24: Overview of the heat exchanges over the river.

where T is the water temperature, $K_{T_{ref}}$ is the value of the parameter at a reference temperature T_{ref} (20°C) and β_K is the temperature correction factor for parameter K.

5.4.2 River model construction

The 'Bleesbruck' river system model is shown in Figure 5.25. It is a one-dimensional longitudinal segmentation of the 3 rivers Attert, Alzette and Sûre into 23 compartments according to physical river characteristics (e.g. very low velocity compartments due to dams or an electric power station) and considering locations of incoming point pollution sources like CSOs or the WWTP. The length of the tanks varies between 600 and 1800m and widths are 10m for the Attert (*Att*) and Alzette (*Alz*) and 20m for the Sûre (*S*).

Using the meteorological data collected at a measurement station in Ettelbruck (maintained by ASTA, Administration des services techniques de l'agriculture, Luxembourg), input blocks feed the following variables into the model in half hour timesteps: solar radiation, wind speed, relative humidity, air and water temperature.

5.4.2.1 Upstream river boundary conditions

Influent files containing river flow and component concentrations are needed at *in_Sûre*, *in_Alzette* and *in_Attert*. Flows were calculated from quarter hourly level meter data with their associated rating curves.

For concentrations, mainly data from the measurement campaigns described in Chapter 4 in section 4.2 were used. Some of the measurements directly correspond to model components, e.g. S_O, S_{NO3}, S_{NH} and S_{PO}. However, not all model components can be measured and need to be derived from lumped measurements. For fractionation of COD, we used:

$$COD_{Tot} = COD_{sol} + COD_{part} \tag{5.14}$$

$$= (S_S + S_I) + (X_I + X_S + X_H + X_{N1} + X_{N2} + X_{ALG}) \tag{5.15}$$

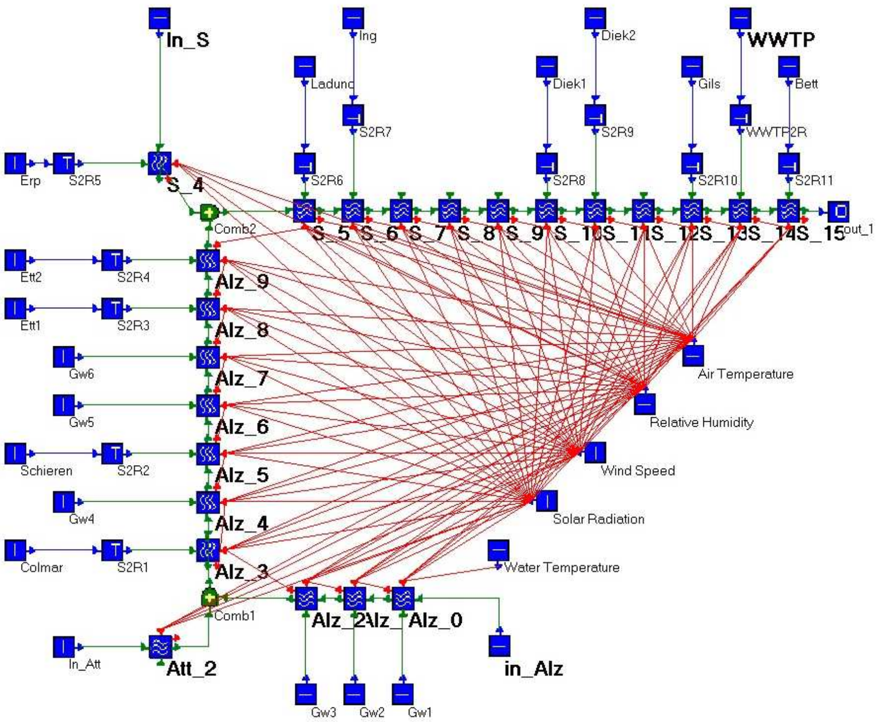


Figure 5.25: Sûre (S), Alzette (Alz), Attert (Att) river system model in the WEST® configuration builder environment (Gw1, Gw2,... =Groundwater; S2R1, S2R2,...= Sewer to river connectors).

Table 5.11: Fractionation of the influent COD for the RWQM.

Measured variable	RWQM1 variable	f_{trans}	given value	unit
COD soluble	S_S	0.6		$gCOD/m^3$
	S_I	0.4		$gCOD/m^3$
ChlA	X_{ALG}	0.4167		$gCOD/m^3$
	X_H		2	$gCOD/m^3$
	X_{N1}		0.4	$gCOD/m^3$
	X_{N2}		0.2	$gCOD/m^3$
X_S + X_I= COD total -	X_S	0.65		$gCOD/m^3$
COD soluble - X_{ALG} - X_H - X_N1 - X_N2	X_I	$\approx$0.35		$gCOD/m^3$

Measured ChlA was converted into algae biomass using a factor 0.4167 from literature (Jorgensen *et al.* (1991)). Table 5.11 summarises the values and fractionation used for RWQM1 variable conversion. Values were taken from previous model applications (Ghermandi (2004) and Benedetti (2006)). Using a river tank, mean daily values from measurement campaigns were used to generate dynamic data that can be fed into the model.

A contribution to river flow that has been taken into account is groundwater intrusion. As no data regarding quantity and quality was available, groundwater flow and nitrate concentrations became calibration parameters, while the other pollutant components were supposed to be insignificant compared to the river concentrations. Note also that no direct surface runoff is taken into account here. Hydrological modelling of the contributing river basins was foreseen and started in collaboration with the Centre de Recherche Public Gabriel Lippman (Luxembourg) using a conceptual model. However, already at the beginning of the modelling exercise, it was noticed that data was not sufficient to obtain good enough calibration. It was estimated that modelling errors would exceed errors generated from omitting direct surface runoff and that feeding in inflow data of the simulation year at boundaries was enough.

5.4.2.2 Urban catchment boundary conditions

From previously performed sewer network and WWTP simulations, the emissions from CSO structures and WWTPs are fed in at their respective locations. However, the variables from the sewer and the WWTP models need to be converted into RWQM variables. To convert sewer variables into RWQM1 variables, the same fractionation was used as to convert them into ASM2d variables (see Table 5.7). For conversion from ASM2d to RWQM1, so-called interfaces (Vanrolleghem *et al.* (2005a)) were applied. The principles of the continuity-based interfacing method (CBIM) are explained in Chapter 3 section 3.4.2. Composition matrices and transformation variables are shown in Tables 5.26 and 5.27.

5.4.3 River hydraulics and transport calibration

As already mentioned above, information on flows upstream of the considered river stretches in the model were evaluated from level meter readings and used as inflows to the model. Two level

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	S_A	S_{ALK}	S_F	S_I	S_{N2}	S_{NH4}	S_{NO3}	S_{O2}	S_{PO4}	X_{AUT}	X_H	X_I	X_{MeOH}	X_{MeP}	X_{PAO}	X_{PHA}	X_{PP}	X_S	S_{h+}	S_{H2O}
	(gCOD m^{-3})	(mol m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gN m^{-3})	(gN m^{-3})	(gN m^{-3})	(-gCOD m^{-3})	(gP m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(g$Fe(OH)_3$ m^{-3})	(g$FePO_4$ m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gP m^{-3})	(gCOD m^{-3})	(gH m^{-3})	(gH m^{-3})
Balance (g/gMatter)																				
α_C	0.407	0.197	0.570	0.610	0.000	0.000	0.000	0.000	0.000	0.520	0.520	0.610	0.000	0.000	0.520	0.558	0.000	0.570	0.000	0.000
α_H	0.051	0.016	0.080	0.070	0.000	0.222	0.000	0.000	0.010	0.080	0.080	0.070	0.059	0.000	0.080	0.070	0.000	0.080	1.000	0.111
α_O	0.542	0.787	0.280	0.280	0.000	0.000	0.774	1.000	0.667	0.250	0.250	0.280	0.941	0.674	0.250	0.372	0.608	0.280	0.000	0.889
α_N	0.000	0.000	0.060	0.030	1.000	0.778	0.226	0.000	0.000	0.120	0.120	0.030	0.000	0.000	0.120	0.000	0.000	0.060	0.000	0.000
α_P	0.000	0.000	0.010	0.010	0.000	0.000	0.000	0.000	0.323	0.030	0.030	0.010	0.000	0.326	0.030	0.000	0.392	0.010	0.000	0.000

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
	S_s	S_I	S_{NH4}	S_{NO2}	S_{NO3}	S_{HPO4}	S_{O2}	S_H	S_{OH}	X_H	X_{N1}	X_{N2}	X_{ALG}	X_S	X_I	X_P	S_{H2O}
	(gCOD m^{-3})	(gCOD m^{-3})	(gN m^{-3})	(gN m^{-3})	(gN m^{-3})	(gP m^{-3})	(-gCOD m^{-3})	(gH m^{-3})	(gH m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gP m^{-3})	(gH m^{-3})
Balance (g/gMatter)																	
α_C	0.570	0.610	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.520	0.520	0.520	0.360	0.570	0.610	0.000	0.000
α_H	0.080	0.070	0.222	0.000	0.000	0.010	0.000	1.000	0.059	0.080	0.080	0.080	0.070	0.080	0.070	0.010	0.111
α_O	0.280	0.280	0.000	0.696	0.774	0.667	1.000	0.000	0.941	0.250	0.250	0.250	0.500	0.280	0.280	0.667	0.889
α_N	0.060	0.030	0.778	0.304	0.226	0.000	0.000	0.000	0.000	0.120	0.120	0.120	0.010	0.060	0.030	0.000	0.000
α_P	0.010	0.010	0.000	0.000	0.000	0.323	0.000	0.000	0.000	0.030	0.030	0.030	0.060	0.010	0.010	0.323	0.000

Figure 5.26: Elemental composition matrices for origin and destination models, for ASM2d (top) and RWQM1 (bottom).

transformations \ components	S_A	comp C S_{ALK}	S_F	S_I	S_{N2}	comp N S_{NH4}	S_{NO3}	S_{O2}	S_{PO4}	X_{AUT}	X_H	X_I	X_{MeOH}	X_{MeP}	X_{PAO}	X_{PHA}	X_{PP}	X_S	S_{h+}	S_{H2O}
	(gCOD m^{-3})	(mol m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gN m^{-3})	(gN m^{-3})	(gN m^{-3})	(-gO$_2$ m^{-3})	(gP m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gFe(OH)$_3$ m^{-3})	(gFePO$_4$ m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gP m^{-3})	(gCOD m^{-3})	(gH m^{-3})	(gH m^{-3})
Sa_fromASM2d	-1	0.00471	0	0	0	-0.03352	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Salk_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sf_fromASM2d	0	0	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Si_fromASM2d	0	0	0	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sn2_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Snh4_fromASM2d	0	0	0	0	0	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sno3_fromASM2d	0	0	0	0	0	0	-1	0	0	0	0	0	0	0	0	0	0	0	0	0
So2_fromASM2d	0	0	0	0	0	0	0	-1	0	0	0	0	0	0	0	0	0	0	0	0
Spo4_fromASM2d	0	0	0	0	0	0	0	0	-1	0	0	0	0	0	0	0	0	0	0	0
Xaut_fromASM2d	0	0.00014	0	0	0	0.02191	0	0	0	-1	0	0	0	0	0	0	0	0	0	0
Xh_fromASM2d	0	0.00014	0	0	0	0.02191	0	0	0	0	-1	0	0	0	0	0	0	0	0	0
Xi_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	-1	0	0	0	0	0	0	0	0
Xmeoh_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Xmep_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Xpao_fromASM2d	0	0.00028	0	0	0	0.04382	0	0	0	0	0	0	0	0	-1	0	0	0	0	0
Xpha_fromASM2d	0	0.00124	0	0	0	-0.03352	0	0	0	0	0	0	0	0	0	-1	0	0	0	0
Xpp_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	0	0	-1
Xs_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	0	0
Sh+_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	0
Sh2o_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1

transformations \ components	S_s	S_I	S_{NH4}	S_{NO2}	S_{NO3}	S_{O2}	S_{OH}	X_H	X_{N1}	X_{N2}	X_{ALG}	X_S	X_I	X_P	comp P S_{HPO4}	comp Ch S_H	comp O S_{H2O}
	(gCOD m^{-3})	(gCOD m^{-3})	(gN m^{-3})	(gN m^{-3})	(gN m^{-3})	(-gCOD m^{-3})	(gH m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gCOD m^{-3})	(gP m^{-3})	(gP m^{-3})	(gH m^{-3})	(gH m^{-3})
Sa_fromASM2d	1	0	0	0	0	0	0	0	0	0	0	0	0	0	-0.00559	0.00675	0.0161
Salk_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sf_fromASM2d	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Si_fromASM2d	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sn2_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Snh4_fromASM2d	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sno3_fromASM2d	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
So2_fromASM2d	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
Spo4_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Xaut_fromASM2d	0	0	0	0	0	0	0	0	0.15	0.35	0	0.42	0.08	0	0.00654	-0.001	-0.00252
Xh_fromASM2d	0	0	0	0	0	0	0	1	0	0	0	0.42	0.08	0	0.00654	-0.001	-0.00252
Xi_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0
Xmeoh_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Xmep_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Xpao_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0.84	0.16	0	0.01309	-0.00201	-0.00505
Xpha_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	1	0	0	-0.00559	0.00328	0.00221
Xpp_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0.06452	0.93548
Xs_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
Sh+_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
Sh2o_fromASM2d	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1

Figure 5.27: Transformation matrices for origin and destination models, for ASM2d (top) and RWQM1 (bottom).

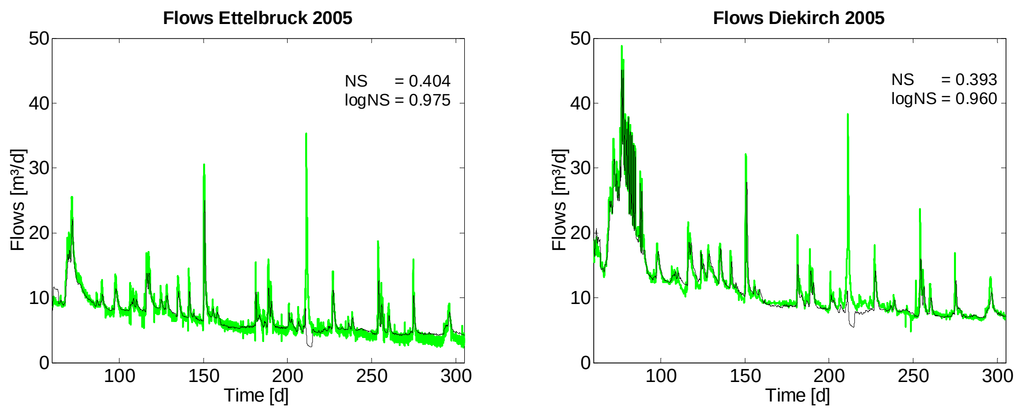


Figure 5.28: Flow calibration (green lines = measured, black lines = simulated) after Tanks Alz_7 and S_10. The table depicts values of the Nash-Sutcliffe and the logarithmic Nash-Sutcliffe coefficient.

meters are installed within the modelled river stretches *Alz_7* and *S_10* (see Figure 5.25). As the Attert has a relatively small contribution compared to the Alzette, intrusion of groundwater into the Attert was omitted, and for the Sûre, no groundwater intrusion had to be included in the region of the modelled stretch. For the Alzette, incoming groundwater quantities were calibrated for winter and early spring. Regarding flow times, the only calibration parameter is k_n, which represents the residence time of the water in a given tank n. These times were estimated from 4 tracer tests at different locations. Figure 5.28 shows the flow calibration for the Alzette and the Sûre. It can be observed that high flows are less well reproduced, which is both due to the fact that $k's$ were fixed for low flow (no tracer test performed during a rain event) and that small tributaries and surface runoff were not taken into account. This is confirmed by the Nash-Sutcliffe and logarithmic Nash-Sutcliffe values (see Equations 5.4 and 5.5). For transport of solute material, similar results were compared and dispersion could not be represented well by the model, as a too high number amount of tanks would have been required, making the computation times too long. Hence, the dispersion in the model is overestimated.

5.4.4 River biochemical processes calibration

Calibration of the river model could not be done over the whole 8-months simulation period as was done for the sewer system and the WWTP. Measured data only cover the last 2 weeks in June 2005 and the 2 weeks at the end of September 2005 (see Chapter 4, section 4.2). Daily average values from the measurement campaigns were used to create input files to the three rivers as has been described above in section 5.4.2. The campaigns also included 2-hourly intensive measurements over 2 days that could be used for calibration. Calibration locations are situated approximately 5 km before and 3 km after the WWTP effluent, i.e. at the outflows of river stretch S_8 (Ingeldorf) and S_{15} (Bettendorf) in the model. Overall, calibration of the conversion model turned out not to be straightforward in this case. Table 5.12 contains the chosen calibrated parameters.

The calibrated nitrate groundwater concentration is the only groundwater quality compo-

Table 5.12: Default and new parameter values for the river model.

Parameter		**Default**	**New**	**Units**
Maximum specific growth rate for algae at reference temperature	$k_{gro,Alg}$	2	20	d^{-1}
Base value for kla	kla_{base}	1.0	2.0	d^{-1}
Exponent for velocity in kla calculation	v_{pow}	0.97	0.5	–
Surface area reduction factor	A_{s_red}	1	0.2	–
Saturation coefficient for the growth of algae on light	K_I	500	200	Wm^{-2}
Groundwater nitrate concentration	$C_{gw}(NO_3\text{-}N)$	0	20	gm^{-3}

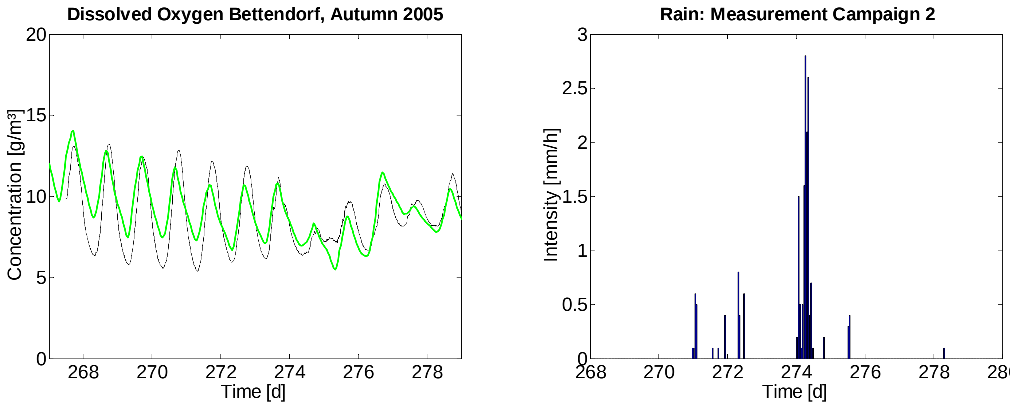


Figure 5.29: Left: Oxygen calibration after Tank S_15 for measured (black) and simulated (green) DO concentrations. Right: Measured rain during that period.

nent supposed to be of significance towards the river component concentrations, i.e. all other groundwater concentrations are supposed to be zero.

A major concern of the calibration was to fit the oxygen dynamics as these represent overall dynamics of the river conversions. The only way to find a parameter set that would reproduce the DO concentration and at the same time reflect concentration ranges for other variables was to suppose very high growth rate of algae in the model. Simulated DO concentrations are depicted in Figure 5.29 and show that depletion of the oxygen concentration after a rain event on day 274 could also be described. Unfortunately, spring campaign measurements at the S_{15} calibration point could not be used due to problems with the temperature measurements on the probe, so that dissolved oxygen concentrations could not be calculated. Further results are shown in Figure 5.30.

The need for such high algae growth rate can probably be explained by the fact that measured algae concentrations (i.e. the values used as boundary conditions for the model and the ones used for calibration) only correspond to the dissolved algae matter, whereas the default model parameters represent the total algae mass, i.e. floating and sessile algae. Indeed, by keeping $k_{gro,Alg}$=2d^{-1}, simulation results fit measured algae concentrations, however dissolved oxygen concentrations could not be reproduced. As the model only contains dissolved algae that are

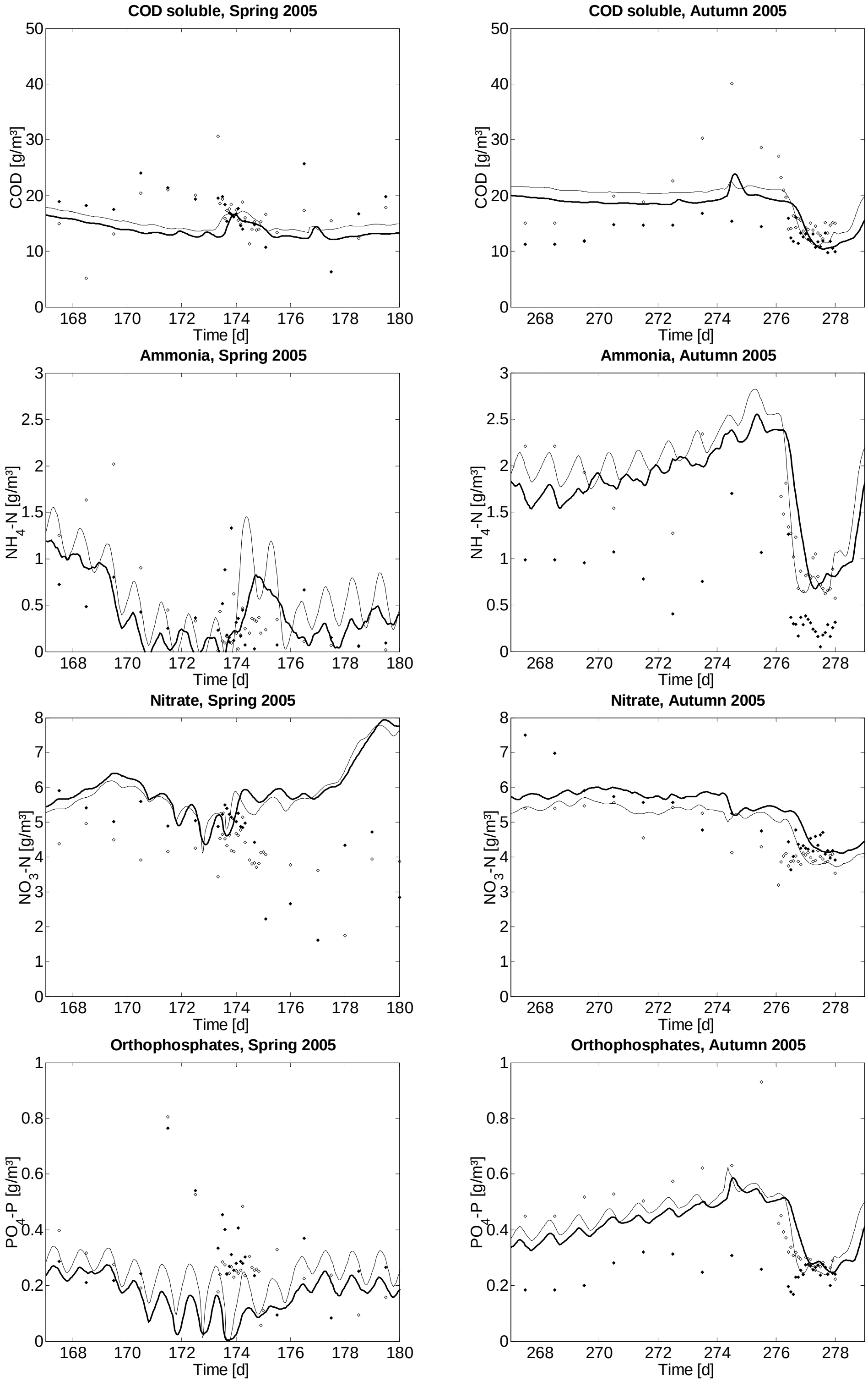


Figure 5.30: Measured (markers) and simulated data upstream (thin lines) and downstream (thick lines) of the WWTP for COD, ammonia, nitrate and orthophosphates in spring and autumn 2005.

constantly washed out, we need to simulate fast growing algae, i.e. $k_{gro,Alg}$=20d^{-1}, so that they can make up for the missing sessile algae. It hence seemed justified to simulate more algae than what was measured and as in the further analysis of scenarios, DO concentrations is one of the evaluation criteria we need good simulation results for this parameter.

The choice of such an extreme value for one of the key parameters was certainly expected to have an influence on the general model behaviour and possibly entail further necessary tuning of other parameters. Indeed modified parameters are directly linked to the oxygen concentration (reaeration rate, equation 5.16) or to algae growth (nutrient uptake, equation 5.17). The reaeration rate is given by,

$$Kla = Kla_{base} \cdot \mathrm{v}^{\nu} \cdot \mathrm{d}^{\delta} \cdot (Kla_{temp})^{T_w - 20} \tag{5.16}$$

where Kla_{base} is the base value for Kla, v the water velocity, d the water depth and T_w the water temperature. Kla_{temp} is the temperature coefficient, ν and δ are exponents for velocity and depth. The growth of algae with NH_4 and NO_3, as given in the RWQM1, is in both cases a factor of the following function

$$f(k_{gro,Alg}, K_I, I) = k_{gro,Alg} \frac{I}{K_I} exp\left(1 - \frac{I}{K_I}\right) \tag{5.17}$$

where I is the solar intensity and K_I is the saturation constant. As is shown on Figure 5.25, solar intensity is fed into the model. Supposing unfortunate positioning of the measurement station and therefore erroneous measurements, a different set of incoming data for solar intensity was tested if it would make K_I tuning unnecessary. However, simulation results did not change significantly.

Ammonia concentrations appear to be smaller in late spring, which might be due to higher uptake through algae. Also, especially for late summer, concentrations downstream are lower than upstream concentrations and this seems to be confirmed by measurements. This indicates that we have faster nitrification of ammonia after the WWTP, i.e. faster increase of NO_3, hence promoting algae growth. Orthophosphates were difficult to calibrate with any set of parameters.

Figure 5.31 shows results for the intense measurement days and it should illustrate that days where dynamic measurements are available produce best results from the river model. Model calibration becomes a trade off between model objectives and available resources to conduct measurement campaigns. In this case it can be concluded that the collection of both daily and hourly samples was useful. The former are more informative for the modeller and were subsequently used to produce missing data for influent to the model. The latter were also integrated to the influent file and served in the calibration check (see 5.31). To calibrate for different seasons more 2-day campaigns could have been useful. The most important parameter however seems to be the DO concentration, as it determines overall dynamics and especially in eutrophied rivers it represents the presence of algae and other oxygen consuming substrates. Hence, using online probes DO measurements at many locations and for longer times can significantly contribute to characterisation of a system and to model calibration purposes. However attention has to be paid regarding the placement of the probe as DO concentrations might vary across the cross-section of the river.

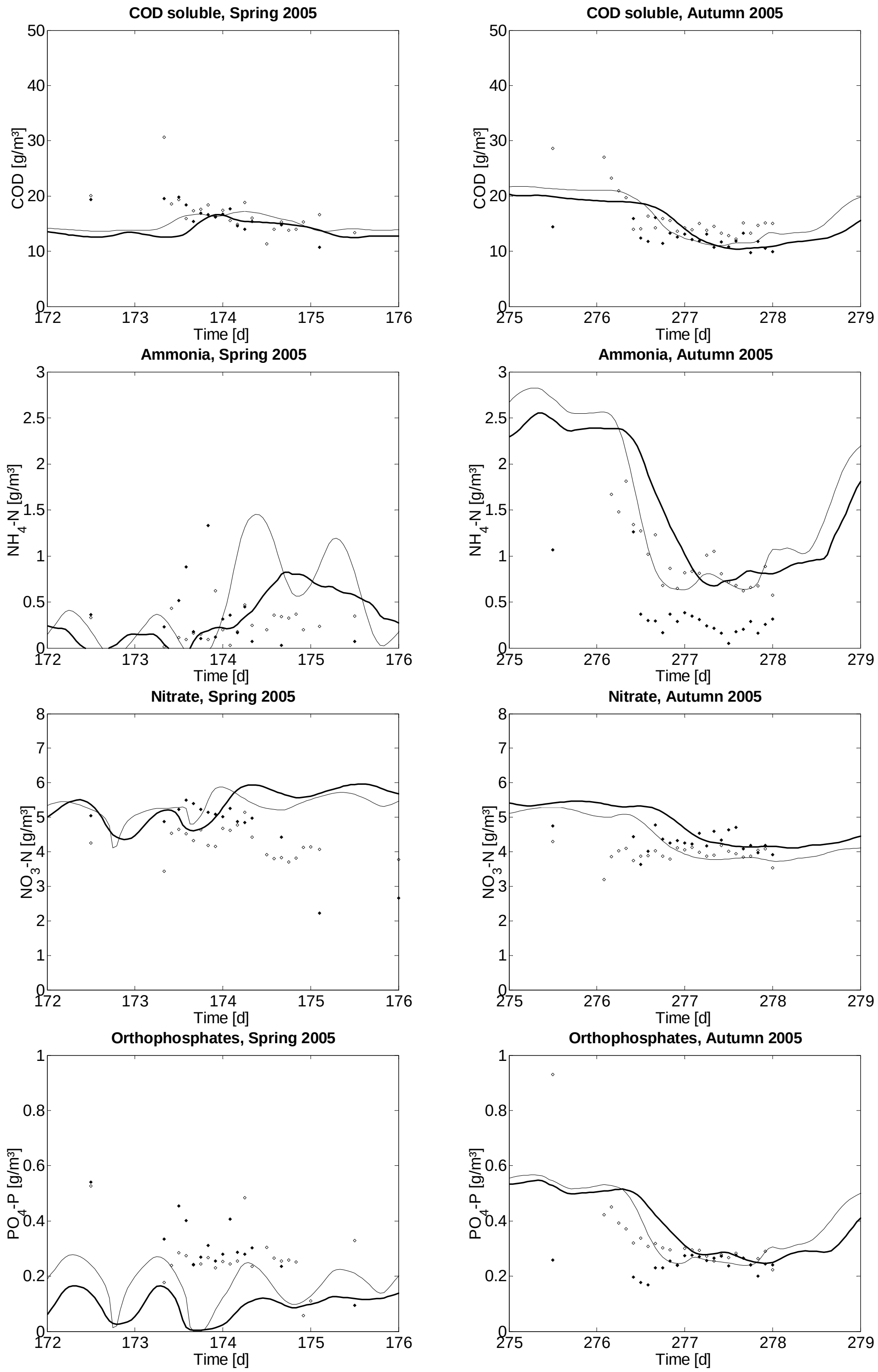


Figure 5.31: Zoom of the previous Figure: Measured (markers) and simulated data upstream (thin lines) and downstream (thick lines) of the WWTP for COD, ammonia, nitrate and orthophosphates in spring and autumn 2005.

A reason that, next to the fact that a river system is a complex system, possibly impedes easy calibration might be related to the fact that we are dealing with 3 individual rivers with different hydromorphology and water quality, suggesting the use of 3 different parameter sets for the river system model. However, supplementary measurement points before the confluence of Attert into Alzette and Alzette into Sûre would have been necessary to properly calibrate parameters for Attert and Alzette in a first place. Interesting would also be to find out whether hydrodynamic modelling could have improved results.

5.4.5 Conclusions

A 3 river system was modelled in WEST® using CSTRs in series for transport and the RWQM for conversion processes. The study shows that the construction of such a complex system is not straightforward and requires a lot of data. Two measurement campaigns in June and September were used to calibrate the model. Calibration periods were low flow situations, which we are primarily interested in as these periods present the highest concentrations of pollutants. In case of wet weather investigations, further measurement campaigns would have been needed and fractionation of measured COD could possibly have become different. The modelling of sediments and the sediment compartment, which was omitted in this case.

However, it is assumed that for a first calibration and for the further use in scenario analysis the river model can be used as such. Indeed, more results and findings from river model simulations are presented in Chapter 6.

5.5 The Integrated Model

5.5.1 Integrated simulations

Simulations of the integrated model have been performed in Tornado and were executed sequentially to save simulation time. For the scenario analysis, the 3 subsystems do not need to 'communicate' as no feedback to upstream subsystems was required. Hence, the results of one subsystem were fed as input into the downstream submodel. Simulation periods were 8 months (March to November) and calculation times ranged from 8 minutes for the river model to 20 minutes for the WWTP model to 25 minutes for the sewer model. Simulation times for simultaneously simulating submodels were tested by assembling the sewer and the WWTP model, and calculation time reached 500 minutes!

This is due to different timescales of the sewer and the WWTP, which is a so-called "stiff system". Simulating a stiff system with a regular solver such as Runge-Kutta is by definition very inefficient. The rate of change of the fastest changing variable is used to determine the stepsize, ending up with a very small stepsize, which is applied to all variables, even to the ones that are stable over longer periods, resulting in lots of irrelevant computations. In order to tackle this problem, special so-called "stiff solvers" have been developed, which do not merely use one stepsize, but a different and appropriate stepsize for each variable. In this way, irrelevant computations for slow variables can be avoided. The most well known textbook example of a stiff system is the VanDerPol system. Simulation with a regular solver can take hours, versus only a couple of seconds with a stiff solver. Tornado contains two stiff solvers: Rosenbrock and CVODE, of which the latter is certainly the better one. One problem with stiff solvers though is that they are sensitive to discrete changes and time was not available to extensively test these solvers on the Bleesbruck case study, but this will certainly happen in the near future.

The integarted model as it is now seems an adequate approximation to the data at hand and to make sure the model represents the system on the long-term, no calibration was performed on a single event. In such case the model might much more accurately reproduce that particular data set but the predictions over the long term would probably be off due to a lack of generalization power. Calibration was completed not as an objective in itself but to obtain the best possible representation of the system. In this case, the long-term calibration for sewer and WWTP was considered to be sufficient for the subsequent comparative scenario analysis.

An important issue in this context is that in this case we have 3 subsystems, with different calibration issues. A WWTP is a much more confined, compact system than the sewer or river system and the IWA ASM models are state of the art, for which applications are manifold. This is not the case for sewer and river systems, especially not for water quality variables. The sewer acts on short time scales, even in DWF conditions so that its dynamics, both for water flow and pollution are subject to more random fluctuations (**??**. We agree that model predictions do generally exceed measurement uncertainties, but no rigorous evaluation of the calibration performance (Reichert (1998)) was performed as results were estimated good enough for the following scenario analysis. In that context, Reda & Beck (1997) found that in their case the robustness of the ranking of scenarios stayed the same with changes of calibration parameters.

5.5.2 Conclusions

Although all of the 3 submodels are run separately, this Chapter tried to show that the same approach for model construction and calibration was applied. The level of complexity and need for data of the models is expected to be uniform for the 3 models. An integrated measurement campaign at the WWTP and in the river was performed to give information in order to both construct and calibrate river and WWTP models. The integrated model will now be used in a scenario analysis, for which it was constructed and calibrated.

Also, before refinement of submodels through more data and/or more measurements, it is useful to use the model, critically analyse its results and to understand its behaviour. This way it can provide useful information, point to eventual errors in construction of the model and help in the interpretation of certain phenomena (see Chapter 6).

Chapter 6

Scenario Analysis

The Chapter first presents the 15 scenarios that, in the context of a combined immission-emission approach, have been tested via simulations of the integrated urban river system model described before. Costs of scenarios are discussed and an evaluation method, dealing with the large amount of simulated data, is presented. The first scenario analysis, done for immission and emissions at different locations, assesses impact of the urban catchment Bleesbruck on the receiving rivers and identifies more and less suitable scenarios. In a second scenario analysis, a semi-hypothetical case study is analysed where the quite high original background pollution of the Bleesbruck receiving waters is set to comply with WFD requirements.

The following chapter is partly developed from the following article:

Solvi, A.-M., L. Benedetti, V. Vandenberghe, S. Gillé, P. M. Schosseler, A. Weidenhaupt and P. Vanrolleghem (2006). Implementation of an integrated model for optimised urban wastewater management in view of better river water quality. A case study. *IWA World Water Congress and Exhibition*, 11-15 September 2006, Beijing, China.

6.1 Scenarios

Simulated scenarios are listed in Table 6.1 and have been chosen within the framework of the EU project CD4WC (www.tu-dresden.de/CD4WC) and with regard to the deficits and pressures of the case study described in Chapter 4 section 4.3. They include *source control* to reduce or flatten incoming wastewater and pollution, *construction* measures to increase uptake capacities within the system, *operation* change scenarios to test system performance under alternate system configurations and *river measures* to directly act upon the processes in the receiving water.

6.1.1 References

Scenario **Ref** represents the existing situation and simulates it using the calibrated model described in Chapter 5. The **None** scenario assumes that the urban catchment and its WWTP do not exist; hence only the river model is simulated. As it is very difficult to determine the state of a river in absence of anthropogenic influence by just using data, this scenario offers another reference state to evaluate how far pollution could theoretically be reduced.

The other scenarios, described in the following paragraphs represent a modified version of the reference integrated model and represent alternative management of the existing system.

6.1.2 Source Control

FlatDWF suggests storage buffer tanks at housing, urban infrastructure or industry level, to temporarily store wastewater to flatten peaks so as to send a more constant water and pollution flow towards the treatment plant. This can improve the treatment plant performance as such buffering eliminates unexpected sudden pollution peaks. In the case study, such pollution peaks especially come from industries like the brewery or the slaughterhouse (see Chapter 4 section 4.1.1.2). To represent this within the model, all DWF patterns from households and industries are set to constant values such as to send uniform water flow and pollution concentrations with the same overall load. **FlatNH** represents the hypothesis that households are equipped with separation toilets, so that urine is separated from grey and black water and stored in small household tanks. The latter are emptied into the sewer system by constant outflow, thereby assuring a constant load to the WWTP at day and night, which is expected to lead to better stability of nitrification. This is especially useful as capacities at the WWTP 'Bleesbruck' are limited (see Chapter 4 section 4.1.2). Such separation technologies are increasingly tested and evaluated, knowing that yellow water (urine), black water (faeces) and grey water (wastewater without toilet) have different composition and might need different treatment and individually serve as energy source or for reuse (Otterpohl (2002)). The modelling and control of such urine tanks is discussed in Rauch *et al.* (2003) and shows that such waste design reduces nitrogen peaks and can even reduce ammonium discharges if the control is extended to keep urine in the tanks during rain periods. Figure 6.1 shows simulated flow and ammonium concentrations at the inlet of the WWTP for FlatDWF and FlatNH.

Table 6.1: List of simulated scenarios with acronyms for later reference and short description.

Domain	Acronyms	Description of scenario
References	**Ref**	Current situation, i.e. calibrated integrated model
	None	No urban catchment present, i.e. simulation of the river model without any input from sewer or WWTP
Source control	**FlatDWF**	Dry weather flow pattern flattened by buffer tanks at households and industries
	FlatNH	Dry weather flow pattern for ammonium flattened by urine separation and buffer tanks at households
	ImpRed	Impervious surface reduction (-25%) by decoupling from the combined sewer network
	InfRed	Mean infiltration reduction (-50%) by sewer network rehabilitation
Construction	**RetBas**	Construction of retention basins at 3 critical locations (3000+700+500m^3)
	SluBu	Construction of incoming sludge buffer tank (100m^3)
	SluWT	SHARON-Anammox treatment on-site
	NitVol	Increase ($\times$2) of nitrification volume (1100 m^3)
Operation	**OvLo**	Increase of WWTP hydraulic load (+33%)
	ImprN	Nitrification cascade control with DO controller set-point controlled by NH_4 measurements
	ImprP	Improved phosphorus control by reducing dosage delay
River measures	**Sha**	Shading in the river by means of tree plantation along the banks, i.e. reduced solar radiation (-33%)
	Reae	Artificial reaeration in the river after the confluence of Sûre and Alzette, i.e. through significant increase of *kla* in Tank S_6 of the model

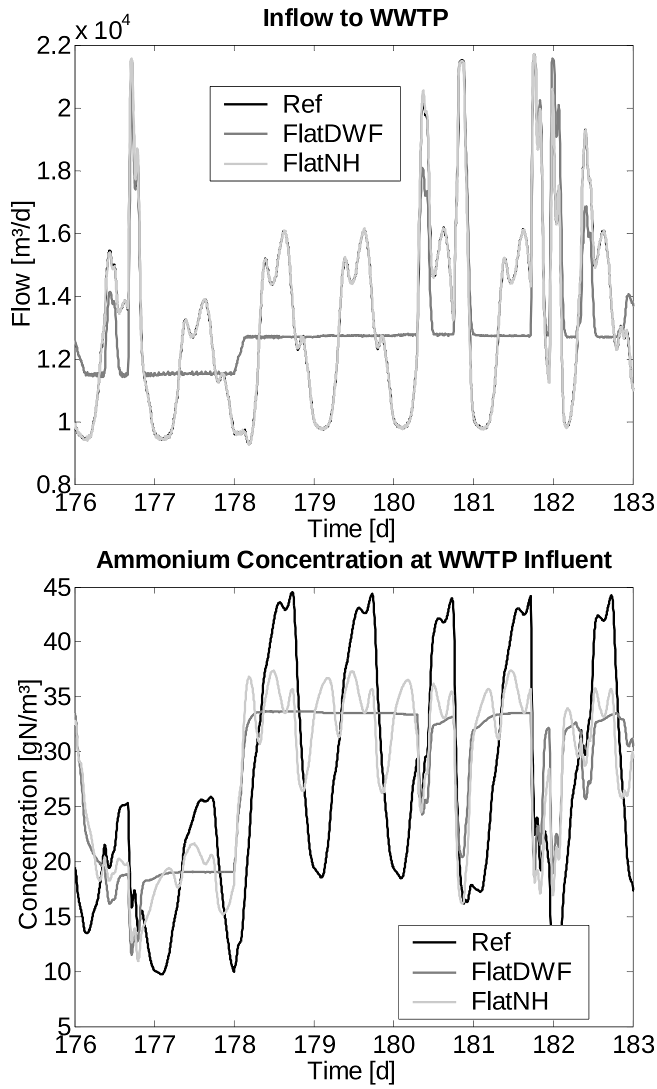


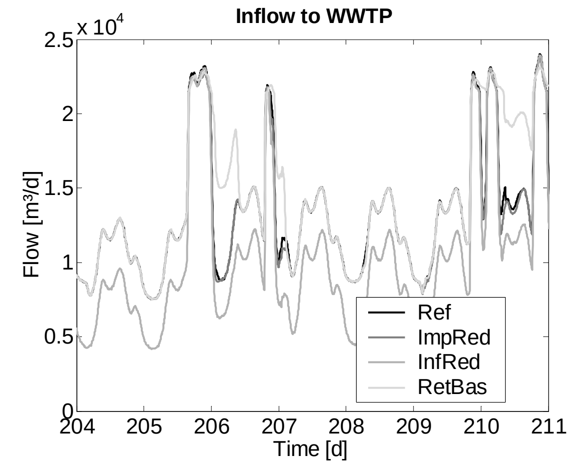
Figure 6.1: Simulated flow and ammonia concentrations at the WWTP inlet.

The **ImpRed** measure should primarily reduce the hydraulic load both in the sewer system and at the WWTP during wet weather, as impacts of storm events on urban wastewater systems are numerous (a.o. Schütze *et al.* (2002), Krebs (2003)). They include increased flow within the system, and in combined sewer systems, wastewater concentrations are diluted. Sending diluted water to the WWTP provokes deterioration of separation efficiency in settlers as well as perturbation of the biochemical equilibrium of the plant with possible washout of sludge. Direct discharges of untreated wastewater to receiving waters are another direct consequence. The reduction of impervious or sealed surfaces by replacing them with pervious surfaces like lawn, will promote local infiltration of rainwater into the soil. The scenario simulates a transformation of 25% of impervious into pervious surfaces, as this was estimated to be a realistic percentage where especially parking lots would be refurbished. Hence, some of the rainwater can infiltrate into the soil before runoff into the sewer system (see Chapter 3 section 3.3.1.2 for the modelling approach) and hydraulic peaks from rain events are reduced. Figures 6.2 and 6.3 show that although the scenario does not deviate significantly from the reference scenario at the WWTP, it does reduce overflow volumes considerably.

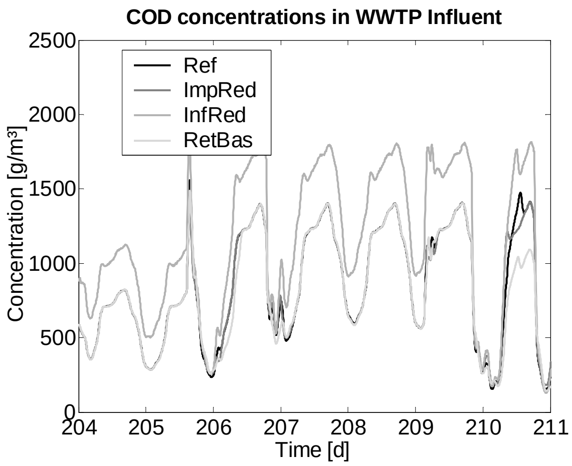
Scenario **InfRed** reduces the amount of infiltration into the local sewer systems by 50%. Parasite waters can intrude through defective joints or cracks in pipes and may vary according to seasons. Inspecting and repairing affected stretches would achieve such infiltration reduction. Infiltration water can be seen as 'clean' water diluting the combined wastewater and consequences are destabilisation of sewer facilities, overloading of treatment facilities or disruption of water treatment processes (Joannis *et al.* (2002)). It means increase of operation costs and, as parasite waters increase emptying times of retention tanks, they increase spilled volumes and loads to the river. Such measure hence reduces the amount of conveyed water and at the same time increases pollutant concentrations, thereby improving process efficiencies at the WWTP (Brombach *et al.* (2002), Fuchs *et al.* (2003)). Authorities do however often consider outflow concentrations (which would increase if less dilution is occurring), and although, as will be shown in the scenario analysis, it reduces overall load discharged, infiltration reduction is often not considered a valuable option. Luxembourg's Management Authority cofinances such measures by 50%, thus considering them as a valuable option. Figure 6.2 illustrates the reduced flow and increased concentrations at the WWTP, and also that there is practically no change for the overflow compared to the reference scenario due to the small proportion of DWF in wet weather conditions.

6.1.3 Construction Measures

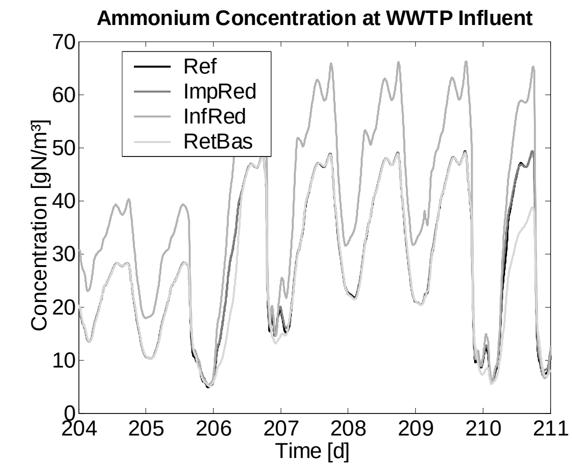
Scenario **RetBas** models additional storage tanks in the sewer network. It is a very common local construction measure applied to reduce the volume, intensity and frequency of CSOs, to retain the first flushes and to reduce the amount of discharged particulates through sedimentation in the tank. Also, as the time lag between urban sewer and river responses is usually large, through storage of the pollution at least at the beginning of an event, the receiving water has some time to raise its flow so that the dilution with the sewage is higher in case of a discharge (Lessard & Beck (1991)). It allows to send a controlled flow (e.g. maximum WWTP design inflow) to the WWTP, and one of the negative influences can be the prolonged wet weather flow



(a) Flow

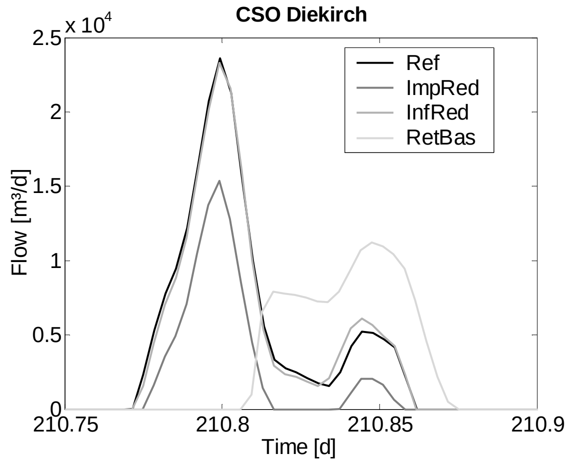


(b) COD particulate

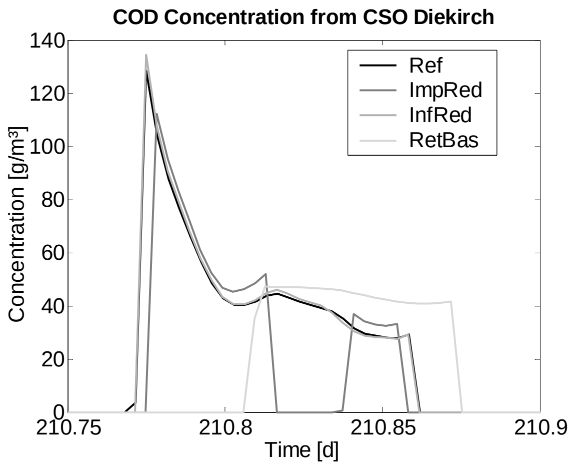


(c) Ammonium

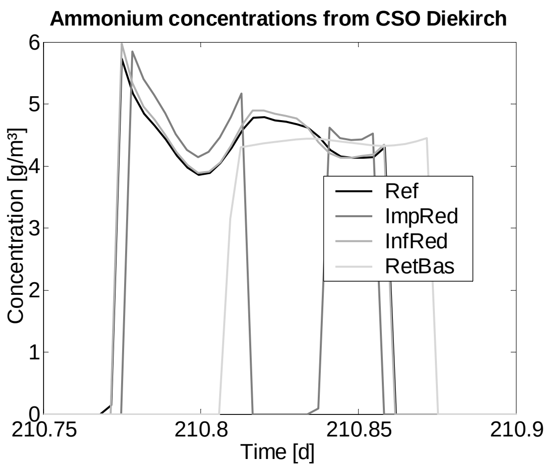
Figure 6.2: Flow, particulate COD, ammonium for different upgrade scenarios at the source and the sewer level at the WWTP inlet (left).



(a) Flow



(b) COD particulate



(c) Ammonium

Figure 6.3: Flow, particulate COD, ammonium for different upgrade scenarios at the source and the sewer level at the CSO Diekirch.

to the WWTP, which can provoke deterioration of WWTP effluent quality as it affects both secondary clarifiers as well as the nitrification process (Rauch & Harremoës (1997), Lau *et al.* (2002), Ashley *et al.* (2002)). For the Bleesbruck urban catchment, several retention volumes are currently being built or were completed in 2006. Three stormwater tanks have been implemented inside the model: one tank after the town of Ettelbruck (3000m^3), and two tanks for the town of Diekirch (700+500m^3). The simulated stormwater tanks are fully mixed sedimentation tanks and simulate 25% of sedimentation (see Chapter 3 section 3.3.2). The sedimented pollution is flushed out of the tank once the volume goes below a certain threshold, which represents the operation of a flushing gate inside the tank. Figure 6.2 shows the prolonged increased inflow and hence reduced COD and TNH concentrations at the WWTP. CSO overflow figures illustrate the reduced discharge volumes and that high concentrations of particulate COD at the beginning of an event can be retained. As the throttle outflow from this tank is controlled and lower than the CSO outflow in the reference scenario, once the tank is full, the tank overflow volume is higher than in the reference case.

Construction measures at the WWTP are **SluBu**, **SluWT** and **NitVol**. The scenario **SluBu** is specific to the case study and the model includes a newly built storage volume for incoming sludges (100 m^3), meant to dampen shock load peaks, in view of improved treatment. **SluWT** suggests on-site treatment of the sludge waters from centrifuges, which in the existing configuration of the WWTP are just sent back and mixed with the sewage entering the WWTP. Through partial nitrification (SHARON) of ammonia into nitrite and subsequent denitrification of nitrite to dinitrogen in an anoxic ammonia oxydation (ANAMMOX) process, NH_4 is removed from ammonium rich wastewater with a minimum of COD and energy consumption (van Dongen *et al.* (2001)). Hence, in the model of this scenario, the recycle of sludge water is omitted by taking away the sludge waters from the influent file. An increase in nitrification volume (**NitVol**) increases the retention time inside the treatment plant. This will allow for a higher sludge age, i.e. a more stable population of autotrophs and therefore higher nitrification capacity. Figure 6.4 shows that the sludge buffer tank scenario will not affect treatment capacity in terms of ammonia, and is just meant to avoid short peaks. The decoupling of the sludge water circuit reduces both ammonium and nitrate concentrations in the effluent and the increase of nitrification volume decreases ammonium and increases nitrate concentrations.

6.1.4 Operational Changes

Operational measures are **OvLo**, **ImprN** and **ImprP**. The **OvLo** supposes that, during wet weather conditions, the overflow limit of the CSO structure just before the WWTP is increased so that the allowed treatment plant inflow during WWF is 33% higher than it is now. By increasing the WWTP inflow above the design load, it has been shown that emission limits might still be met by the WWTP (e.g. Lessard & Beck (1990), Meirlaen *et al.* (2002), Seggelke & Rosenwinkel (2002)), but such overload can also bring about negative impact on oxygen to the river due to sludge loss from the plant (Rauch & Harremoës (1996)). Scenario **ImprN** improves nitrification through a new control strategy. The on/off controller for aeration implemented in the reference model is replaced by a cascade control through measurements of oxygen and ammonium concentrations (Olsson & Newell (1999)). The ammonium level is kept at a value

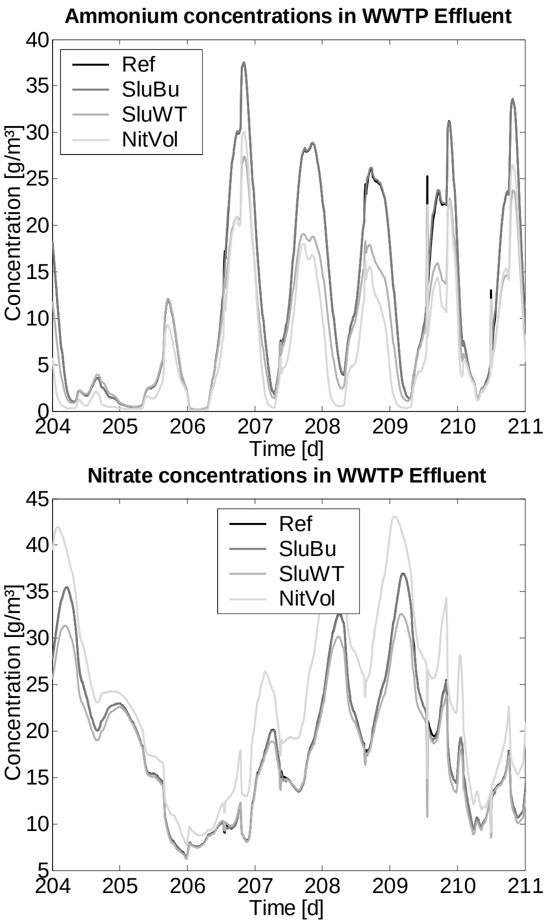


Figure 6.4: Simulated ammonium and nitrate concentrations at the WWTP outlet for different upgrade scenarios involving construction measures.

of 3 mg/l in the effluent of the tank by a supervisory P controller (master) that determines the set point of a PI DO controller (slave). For scenario **ImprP**, phosphorus control is improved by replacing the on/off by a P controller. Figure 6.5 shows for illustration that, compared to the reference scenario, the concentrations of soluble COD in the WWTP effluent increase during the wet weather overload scenario, ammonium decreases for improved ammonium control and similarly for orthophosphate concentrations with improved phosphorus control. However, the processes within the model are complex, non-linear and correlated so that each scenario will bring about more subtle changes than are shown and discussed here, but this goes beyond the scope of this section and thesis.

6.1.5 River Measures

The **Sha** scenario suggests planting of trees or scrubs along the river bank. Such shading decreases solar radiation and therefore the growth of algae (a.o. Hill (1996), Mosisch *et al.* (2001), Ghermandi (2004)). An immediate consequence is that oxygen variations are not so pronounced anymore, which, during nights, increases the risk of DO concentrations going below a threshold that represents suffocation danger for fish populations. Higher DO concentrations do also increase a streams capacity to assimilate organic wastes from sewer, WWTPs or diffuse sources. Another effect of shading is that it reduces water temperatures, which entails higher oxygen uptake capacities of the river water, DO and decreases rates of bacterial breakdown of organic matter, thus reducing DO consumption. The scenario is simulated by reducing incoming solar radiation intensity by a third. **Reae** will simulate artificial reaeration in one of the river tanks. Such measure can be implemented on specified stretches in standing waters or eutrophied rivers, where dissolved oxygen levels could go below thresholds (Vandenberghe & Vanrolleghem (2005)). In this case, the aerator was simulated by controlling the dissolved oxygen in stretch *Alz_8* to stay above 8 mg/l. This tank was chosen as it represents a river stretch before a dam so that it is likely to have zones of slowly flowing or even standing waters with low natural reaeration. Figure 6.6 shows the reduction of the algae concentration and the decrease in dissolved oxygen for the shading scenario (for more information, see section 6.4.1.1). It also shows that the reaeration in one of the stretches upstream will help to keep the DO concentration above a certain level.

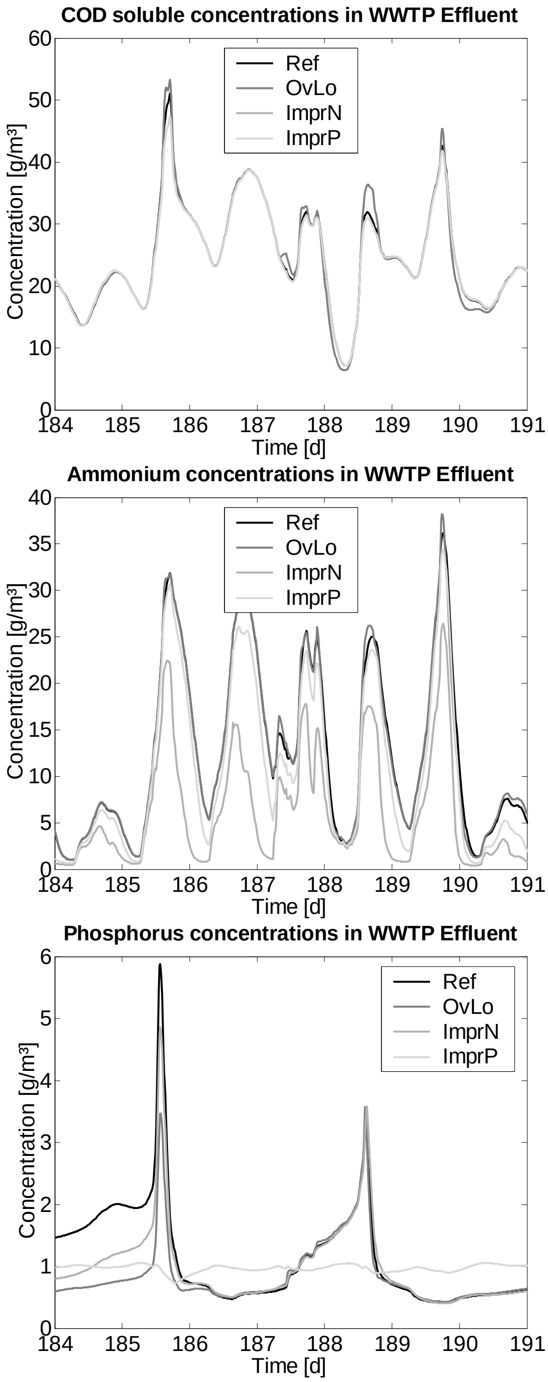


Figure 6.5: Simulated COD, ammonium and orthophosphate concentrations at the WWTP outlet for the different operational measures.

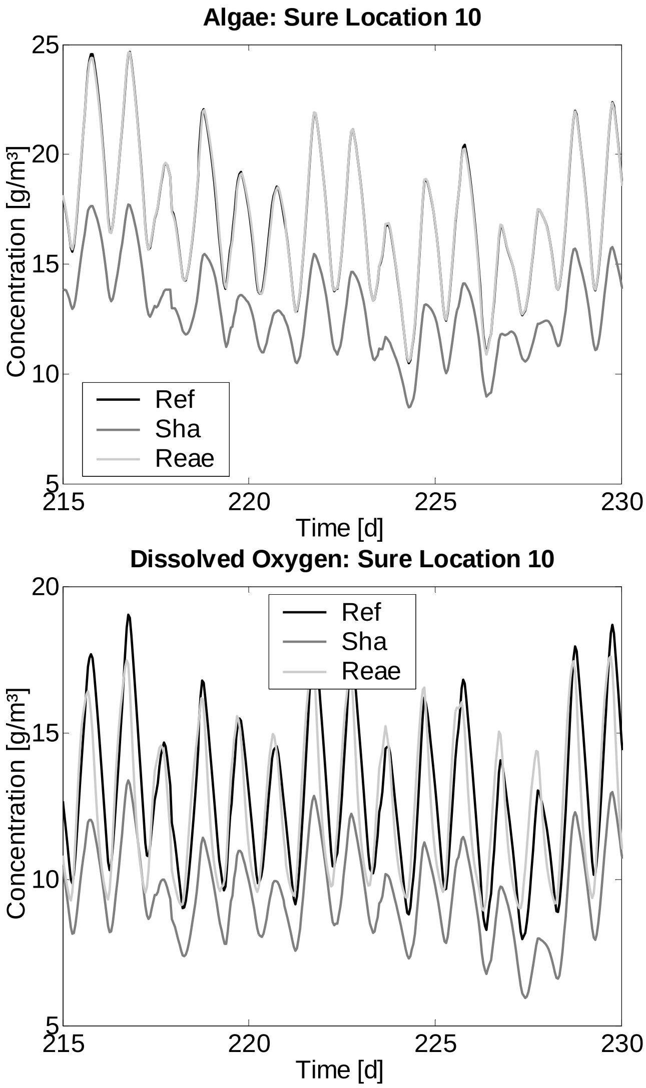


Figure 6.6: Simulated algae and DO concentrations at location 10 in the Sûre for the river shading and reaeration scenarios.

6.2 Costs

Costs will certainly play an important role in a decision process by stakeholders and will vary for each case study. More than indicating any exact evaluation of costs, this section aims to provide orders of magnitude for costs of each scenario for the 'Bleesbruck' example so that they can be compared to each other (see Table 6.2). In this evaluation, costs include investment costs to implement a measure and, additional operation and personnel costs. Information on costs were taken from CD4WC (2006), Benedetti (2006), Haeck (2006), Hillenbrand & Böhm (2003), Günthert & Reicherter (2001) or from personnal communication with the local operator SIDEN. To compare what the implementation of each measure would mean in terms of additional financial resources, costs of the **Ref** scenario are put to zero. Operation costs include sludge treatment, chemicals, oxygen transfer (Benedetti (2006)) and pumping energy costs and savings. They were estimated from simulated variables like waste sludge, pumped chemical, oxygen concentrations and volumes of water in the system. Investments for replacement of machinery and infrastructures due to ageing over the years are not included. It should be noted that the polluter-pays-principle has also not been accounted for (e.g. through effluent taxes), and not upgrading a system according to legislation might entail environmental penalty costs.

Table 6.2: Estimated scenario costs for the case study 'Bleesbruck'. Operation and personnel costs are given for the additional costs compared to the costs in the reference case.

	Investment (€)	Additional costs for operation, maintenance & personnel (€/y)
Max	20,000,000	340,000
Min	0	0
Ref	0	0
FlatDWF	12,480,000	100,000
FlatNH	20,000,000	250,000
RedImp	5,000,000	<0
InfRed	6,000,000	30,000
RetBas	4,200,000	33,600
SluBu	30,000	>0
SluWT	500,000	50,000
NitVol	120,000	45,000
OvLo	0	>0
ImprN	20,000	7,000
ImprP	15,000	5,000
Sha	?	10,000
Reae	100,000	340,000

FlatDWF requires the construction of retention volumes at industry or housing level. The Bleesbruck catchment approximately produces around $52000\text{PE} \times 0.12\text{m}^3/\text{PE/d} = 6240\text{m}^3/\text{d}$ of wastewater, including new prospects for industry and population. Supposing that we want to store all of it for one day and that industries might have days where they produce more and days where they produce none, we assume a required capacity of twice the above volume, i.e. 12480m^3. From sewer storage tank investment costs, we estimated a cost of 1,000 Euro/m^3, i.e.

12,480,000 Euro. Maintenance and operation of these tanks requires cleaning and electricity for outflow control and they were estimated in the order of 100,000 Euros (around 8 Euros/m^3/year, Hillenbrand & Böhm (2003)). **FlatNH** will require a separation toilet and extra drainage for each household (1,000 Euros) (personnal communication during pilot project visit in SolarCity, Linz (A)), and a small tank to collect the urine (1,000 Euros). For an estimated 25000 inhabitants, supposing 2.5 persons per household, the investment costs become 20,000,000 Euros. Maintenance costs, whether it will be paid from public taxes or private households will certainly involve no less than 25 Euros/year per installation.

The rainwater management related costs are inspired from Hillenbrand & Böhm (2003). The **ImpRed** requires the transformation of impervious surfaces to pervious surfaces, in this case 25% of the total impervious surface, i.e. 0.25×200 ha. Costs for laying of infiltration surfaces in newly built areas can be lower than those for impervious areas, but to replace existing surfaces costs were taken to be 10 Euros/m^2 on average, depending on the kind of new surface. The maintenance costs are non-existing to very low, and operation costs are just slightly reduced through electricity savings for pumping and aeration therefore supposing none or a little less costs than in the existing situation. **InfRed** would require the location of cracks and leaks in the sewer system and a rule of thumb (Günthert & Reicherter (2001)) gives that the cost for pipe rehabilitation per meter is equal to its diameter in millimetres. Therefore, supposing that half of the sewer network is old and subject to infiltration, and that half of this is checked and rehabilitated, i.e. 15 km out of a total of 60, we obtain total costs of 6,000,000 Euros, supposing a mean diameter of 400mm. Due to improved performance of this scenario, more sludge is produced that requires treatment and related costs were estimated to be around 30,000 Euros, including savings for reduced pumping. Scenario **RetBas** foresees 4000m^3 of volume construction. An average of 1,000 Euros/m^3 gives a total of 4,000,000 Euros of investment costs. Maintenance and operation costs involve personnel and cleaning costs, which can be omitted by including a flushing system upon construction of the tank. We used 8 Euros/m^3/year for operating costs including electricity for pumps and gates, i.e. 33,600 Euros/year (Hillenbrand & Böhm (2003)).

SluBu buffer tank construction costs around 30,000 Euros (personnal communication from the operator). For the **SluWT** scenario, the investment costs for a SHARON-ANAMMOX treatment with a capacity of 1200kg of NH_4-N/day are given to be 2,000,000 Euros (www.stowa.nl). At Bleesbruck, the NH_4-N load in the recycle sludge water is estimated around 250 kg/d. Hence, investment costs are estimated around 500,000 Euros and investment into energy, methanol and lye are 50,000 Euros/year. **NitVol** doubles nitrification volume, i.e. it adds 550 m^3 to the existing volume and the investment was estimated to be 550,000 Euros based on costs per volume as for retention basins. In this case aeration will have to provide air to double the volume. These costs were calculated using the additional oxygen consumption and were estimated around 45,000 Euros/year (Benedetti (2006)).

The investment costs for **OvLo** will certainly be very low as it only involves a modification of the incoming CSO structure in order to increase the possible inflow to the WWTP. The costs required to treat the additional water are small as, only during wet weather, it requires a little more pumping. The control scenario **ImprN** needs, supposing a complete installation

(5,000 Euros) of new DO sensor (3,000 Euros) and NH sensor plus a controller (12,000 Euros), investment for about 20,000 Euros. For service contracts and maintenance, one should foresee around 7,000 Euros (Haeck (2006)). **ImprP** investments are expected to be in the same order of magnitude as for the nitrogen control, i.e. 17,000 Euros and the same is true for its operation costs (simulations show that a reduction of introduced chemicals, hence costs, can be achieved).

Costs for implementation of the **Sha** scenarios can include the acquisition of land, where prices can be high especially in urban areas. In a second step, costs are determined by the types of plants chosen and a certain maintenance will have to be provided during the first years after plantation (thinning, pruning, weed control), supposing one person once a week (salary = 50,000 Euros/y). **Reae** scenario costs for acquisition and installation of the aerators were estimated as 100,000 Euros. Assuming a fine bubble aerator is working during the critical months of the year for DO concentrations, operation and maintenance costs were evaluated to be around 340,000 Euros/year (CD4WC (2006)).

6.3 Evaluation Criteria

The complexity of the scenario analysis lies within the choice of criteria and the interpretation of results due to the very large number of possibilities given to the modeller to analyse modelling outcomes and differences in the scenario simulations with respect to the reference case. In the end, based on many types of information (scientific, cost-benefit, risk-related,...), decisions have to be taken by stakeholders and to do this, multi-criteria decision analysis (MCDA) can be applied in several contexts ranging from social to environmental (e.g. Linkov *et al.* (2006)).

Hence, the choice of appropriate criteria lies within the objectives of the study and of course the available resources. A major aim of the scenario evaluation was to perform the analysis in a combined emission-immission approach as required by the WFD. Within this case study, the evaluation of model results will have two purposes:

- understand effects of the tested measures within the integrated system. This is done via inspection of simulation results of variables at different locations in the system from within the model, which does not only provide understanding of the system but also serves to check whether produced results seem plausible and are numerically stable, before they are used in the evaluation matrix described in the next section,
- identify the appropriateness and implementation feasibility of certain scenarios for this case study. This is done by comparing each scenario with the reference scenario, using a multi-criteria matrix with colour scheme for ease of evaluation.

6.3.1 The evaluation matrix

Table 6.3 shows the here applied *evaluation matrix* for *long-term* assessment of simulation results. *Locations* for performance evaluation are interfaces, i.e. CSOs and the WWTP, and several locations in the river model. As already mentioned, within the context of the WFD implementation, *immission* concentrations as well as *emission* loads and concentrations will be

Table 6.3: Example of the evaluation matrix at some location within the integrated system.

	Variable 1					Variable 2					…	
Emission	Load	Mean	Max	D*thr*	F*thr*	Load	Mean	Max	D*thr*	F*thr*	…	…
Max												
Min												
Ref												
Scenario 1			0.85			0.81	0.81	0.91	0.78	0.83		
Scenario 2			0.94			0.92	0.89	0.95	0.93	1.06		
…												

compared to the reference scenario. Chosen quality related *variables* and related *thresholds* will depend on legislative criteria, or, in the river, on toxicity for fish etc. To get an idea of the overall performance of a chosen scenario over a whole simulation period of 8 months, the following *criteria* for variables were chosen: means, maxima and minima concentrations in mg/l, exceedance durations D in days (i.e. time spent above/below certain concentration thresholds) and the number of exceedances F (i.e. how often the concentration threshold is crossed over the simulation period). The numerical value after letters D and F gives indication of the respective variable's threshold (denoted by '*thr*' in the table). Total emission loads over the simulation period of chemical oxygen demand (COD), total ammonium (TNH), total nitrogen (TN) and orthophosphates (PO) are given in tons unless indicated otherwise, and in m^3 for volumes of water from CSO structures.

Relative values of all the above named criteria are calculated for each scenario with respect to the reference case, i.e. the integrated system as it exists now. The outcomes are filled into a table and a colour scheme is applied to allow for visual evaluation of scenarios. To account for uncertainties and eliminate possible evaluation of differences originating from numerical calculations during simulation runs, cells containing relative values in the range $0.95 < x < 1.05$ are shaded in grey (and values are omitted from the table). To mark improvement between 5 and 15 %, the concerned cells are shaded in a light grey. For further amelioration, i.e. >15%, the cell will be left in white. Negative influence from the measure will colour the cell in black, see Table 6.3. Hence, as an example, if a measure provokes a considerable increase of the TNH concentration in the river and the relative value goes above 1.05 the cell is coloured in black. Note however that for DO, the same relative value will produce a light grey cell, as 'more' oxygen in the river means an improvement compared to the reference case.

Above the relative values cell block, absolute values are given for the *reference* scenario, as well as for maximum and minimum of scenarios, in order to get an idea about the order of magnitude for variables and criteria.

Interpretation of cells needs to be done with care as often the isolated consideration of such a relative value can mislead the overall evaluation of a scenario. In that sense, for example, the number of exceedances always needs to be considered in parallel to the duration of exceedance: an increased number of exceedances does not necessarily mean that a certain measure has negative impact. It might just mean that, if the duration above the limit has decreased, the threshold value was crossed more often than in the reference scenario. Another criterion to be interpreted carefully concerns the maxima and minima. One should know that these criteria focus on one single value within the simulation results and the 'extreme event' in the reference scenario might not correspond to the 'extreme event' in the considered scenario. Hence, direct comparison cannot be done. Also, although a scenario might reduce peaks in general, it might not be able to reduce the extreme event peak. Maxima and minima criteria are however particularly useful to check the simulations for their numerical stability: when decreasing maxima time stepsize accuracy, simulation results should not be changing anymore.

The results are evaluated as qualitative trends and should by no means be interpreted as absolute values. No uncertainty analysis was done.

6.3.2 Criteria and thresholds for the Bleesbruck case study

For emissions, the following thresholds were chosen according to the Urban Treatment Wastewater Directive (CEC (1991)) for a WWTP having capacity between 10'000 and 100'000 PE: *Total COD<125mg/l, TN<15mg/l* and *TP<2mg/l*. As the Bleesbruck treatment plant does not comply with TN emission standards for most of the time, total TNH has been added as criterion with the same limit of 15 mg/l. For emissions from the CSO structures, water volumes, duration and frequency, as well as particulate COD, ammonium and orthophosphate loads were chosen. For all of these variables also mean and maxima were considered.

For immission-based evaluation, we have chosen *dissolved oxygen, total ammonium, orthophosphate and total COD*. Dissolved oxygen is certainly one of the most important variables to assess river status. It is relevant to many processes like growth and when it drops below certain thresholds it can directly endanger fish populations. In this study for a lowland river, a threshold of 5 mg/l was chosen, which constitutes a good average value for occurring fish species (FWR (1998)).

Ammonium and phosphorus are both nutrients for algae and, in case of too high concentrations, can overfeed the algae resulting in eutrophication. This excess growth can impact on water quality directly (e.g. unsightly scums, clogging of the water course) or indirectly by exacerbating other problems (e.g. oxygen depletion, ammonia toxicity,...) (Chapra (1997)). Further secondary effects are for example loss of submerged aquatic vegetation due to the shading provoked by the plants and consequently the disturbance of the balance of organisms. Total ammonium (TNH) is contained in the river under ionised and unionised form (NH_4^+ and NH_3), but only the latter is toxic for fish. From the UPM manual (FWR (1998)), Table 6.4 indicates frequency and duration for un-ionised ammonium thresholds that should not be crossed and when DO goes below 5mg/l as well, a correction factor smaller than 1 is multiplied with these limits. The table applies to the cyprinid fish family, which the barbel fish, frequently found in these regions of Western Europe, is a member of. The amount of available NH_3 can be derived from TNH using temperature and pH, so that

$$c_{\mathrm{NH_3}} = \left(\frac{1}{1 + 10^{\mathrm{pKa}-\mathrm{pH}}}\right) c_{\mathrm{TNH}}, \text{ where } \mathrm{pKa} = -\log\left[\exp\left(\frac{-6344}{\mathrm{T}}\right)\right]. \qquad (6.1)$$

Table 6.4: Fundamental intermittent standards for un-ionised ammonia - concentration (mg/l)/duration thresholds not to be breached more frequently than shown for an ecosystem suitable for cyprinid fishery (FWR (1998)).

Return period	1 hour	6 hours	24 hours
1 month	0.150	0.075	0.030
3 months	0.225	0.125	0.050
1 year	0.250	0.150	0.065

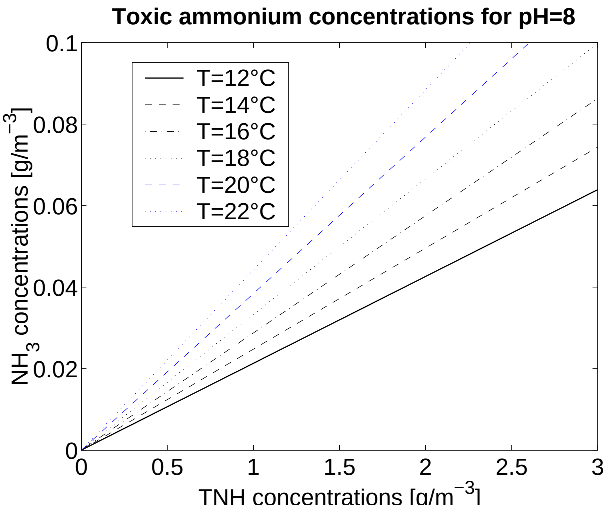


Figure 6.7: Toxic ammonium concentrations against total ammonia for various temperatures at pH=8.

T is the temperature in Kelvin. From Table 6.4 and Figure 6.1, the immission thresholds for ammonium have been chosen to be 2 mg/l. This way, it can be tested whether daily discharge peaks from the WWTP can raise concentrations in the river to reach toxic ammonia concentrations of 0.075 mg/l.

Both TN and TP can be the limiting factor to algae growth. A rough rule of thumb for assessing which nutrient is limiting, relates to the nitrogen-to-phosphorus ratio (Borchardt (1996)). Ambient TN:TP ratios greater than 20:1 are considered phosphorus limited, and ratios smaller than 10:1 N-limited. In the case of the Bleesbruck rivers, the ratio is above 20 and therefore phosphorus seems to be the limiting nutrient. It should hence be reduced as much as possible and the threshold for duration and frequency calculations was set to be 0.4mg/l.

Total COD is used as another criterion indicating pollution. It represents organic carbon whose decomposition might also lead to oxygen depletion and toxicants preferentially associate with it (Chapra (1997)). It is difficult to estimate baseline natural conditions for a river, and criteria have to be defined according to each situation.

Next to chemical criteria, economic criteria will certainly play an important role in the decision process on whether or not implementation of a certain measure is feasible or not. Such costs were discussed in section 6.2.

6.4 Scenario Analysis 1

The scenario analysis will evaluate the performance of the individual scenarios described in section 6.1 and help to understand interactions within the integrated system. The evaluation matrix and applied colour schemes for comparison were discussed in section 6.3. Some general remarks are the following:

- The **Ref** scenario is represented by the calibrated model of the 'Bleesbruck' catchment-sewer-river system as described in Chapter 5 and all other scenarios are portrayed according to changes within this model.
- The **None** case will indicate the maximum improvement possible with respect to chemical river water quality as it completely excludes any influence from a catchment or WWTP.
- Not all single cells in the evaluation matrix that show criteria improvement or degradation for a specific scenario will be commented in the analysis, as it was felt that only the most relevant results should be discussed, without loosing the overall picture.

The evaluation will start with immission results after the WWTP discharge point, followed by emissions from the WWTP. The same will be done for 2 'critical' CSOs, followed by a look at the total emissions from the system sewer plus WWTP.

In section 6.5, a second scenario analysis is performed for the hypothetical case that all of the receiving rivers have 'good' chemical water quality, which is not true in the existing situation. It has the purpose to evaluate impacts of the urban catchment under study once the WFD is implemented in the concerned river basins.

6.4.1 WWTP discharges and effects

6.4.1.1 Immission

Immission results after the WWTP discharge point are summarised in Table 6.5. The cells representing means for variables from the **None** scenario reveal that possible improvement in the river after the WWTP discharge point through measures in the catchment or the WWTP is relatively small. Except for ammonium, the proportionally little hydraulic contribution of the WWTP and the CSOs compared to the river flow ($< 3\%$) cannot significantly influence the mean river concentrations. Also, the pollutant base concentrations are already relatively high due to the bad river water quality of the river Alzette upstream the considered catchment, so that mixing with the higher concentrated wastewater will not have as much effect as in the case of low base pollution concentrations in the river (see section 6.5).

Starting off with *DO* immission results after the WWTP, Table 6.5 clearly shows that, only in-stream measures like **Sha** and **Reae** have considerable effect on the river DO concentration.

Table 6.5: Evaluation matrix for immission after the WWTP: Simulation results with DO threshold 5mg/l, NH_4-N threshold 2 mg/l, PO_4-P threshold 0.4mg/l (see section 6.3.1 for explanation of the matrix).

	DO				NH				PO				COD	
Immission	Mean	Min	D5	F5	Mean	Max	D2	F2	Mean	Max	D04	F04	Mean	Max
Max	12.4	6.8	20.5	28	1.40	2.76	29.5	26	0.38	0.65	97.5	50	37.6	65.2
Min	7.9	3.1	0.0	0	0.99	2.56	8.5	5	0.32	0.55	47.4	27	33.1	55.9
Ref	11.5	4.6	4.6	10	1.17	2.72	18.1	16	0.33	0.60	60.8	45	37.5	62.7
None			0.81		0.85	0.94	0.47	0.31		0.90	0.78	0.71		
FlatDWF						0.94	0.77	0.50						
FlatNH							0.89	0.69				1.09		
RedImp								0.94				0.89		
InfRed			0.91				0.83	0.94		1.08	0.92	0.84		
RetBas								1.19						
SluBu														
SluWT							0.77	0.50						
NitVol					0.94		0.69	0.50						
OvLo								1.06				1.07		
ImprN			0.95		0.92		0.67	0.50						
ImprP							0.94					1.11		
Sha	0.69	0.68	4.47	2.80	1.21		1.63	1.63	1.14		1.60	0.60	0.88	0.89
Reae	1.07	1.47	0.00	0.00				1.06						

Measured and simulated DO concentrations indicate that the river is in a state of supersaturation, i.e. that oxygen levels mostly stay above saturation concentrations (>8mg/l) (see Figure 6.8). Due to the presence of high algae concentrations, DO concentrations can reach more than 12mg/l during the day so that, even at night, concentrations do not often go below a DO concentration that cause fish suffocation.

The **Reae** scenario considerably increases the minimum DO concentrations, and therefore constitutes a favourable option for improvement of water quality. Not only are DO concentrations improved locally, i.e. in the river stretch the aerator is placed in, but the simulations also show improved oxygen concentrations in the river part downstream the WWTP (see Figure 6.8). Although costs (see section 6.2) for implementation of reaeration are relatively high regarding maintenance and operation, it is a useful measure in case of high eutrophication and fish suffocation at night. Obviously a more thorough analysis of the river system is then required, along with onsite measurements, to determine best locations and operation schemes for reaeration.

With the **Sha** scenario, average DO concentrations decrease significantly and ammonium as well as phosphorus concentrations increase due to reduced consumption by the lower algae mass present in the river, which is reflected in the calculated mean and maxima COD decrease. Also less ammonium will be nitrified due to decreased transformation rates caused by the temperature drop (less solar radiation). We find that the minimum DO values have decreased and the time of exceedance (time fraction below threshold) of minimum DO concentrations has increased. This is an unexpected result as with the reduction of solar radiation and algae mass, day DO concentrations were expected to decrease and vice versa for night DO concentrations. However, reducing the algae mass will take away supersaturation, which, at night, prevented DO levels to drop too low. Due to the high income of substrate at the Alzette model boundary, and lower DO

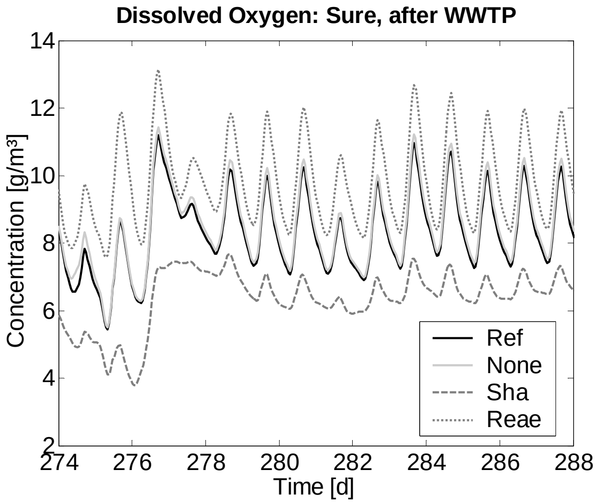


Figure 6.8: DO concentrations at locations 8 (Alzette) and 15 (Sûre).

concentrations with shading during day (see Figure 6.8), oxygen concentrations can go below a critical threshold during night due to oxygen consumption by bacteria. It seems that shading is only appropriate in cases where the incoming COD is small, for example in case of diffuse phosphorus pollution at the source of a river.

With scenarios **InfRed** and **ImprN**, the duration below the DO concentration threshold is affected although only by little, and interpretation of these scenario results becomes more speculative. It should be noted however, that the improvement reaches up to 50% of the improvement possible in the case of no emissions at all (**None**). Looking ahead at emissions in Table 6.6, we find that infiltration reduction will reduce emission loads for every considered component, and is the only scenario reducing COD discharge loads. The improved nitrogen control has best performance regarding discharge of ammonium in 4 out of 5 NH emission criteria, suggesting less nitrification and oxygen consumption in the river. Figure 6.9 shows DO immission concentrations for a period of 4 days and illustrates the infinitesimal effect of the catchment on the river, which slightly more pronounced during the rain event.

The influence of the WWTP on immission concentrations is most visible for *ammonium*. In the river after the WWTP discharge point, concentrations of total ammonium can stay above 2 mg/l and can result in toxic unionised ammonium concentrations under the right temperatures and pH conditions. Therefore, although overall mean concentrations cannot be improved, the shortened time span for which ammonium levels stay above thresholds can reduce risks for ammonia toxicity (**FlatDWF**, **FlatNH**, **InfRed**, **SluWT**, **NitVol**, **ImprN**). Figure 6.10 shows ammonium concentrations after the WWTP and that, with these scenarios, one or the other exceedance can indeed be avoided. Very visible are the diurnal variations of river concentrations and the increased influence of the WWTP during the week as opposed to the week-end (days 268-269 and 275-276). During rain events (e.g. day 274), the effect of higher concentration in the **InfRed** scenario is more pronounced in the river due to the higher discharges of water. Although

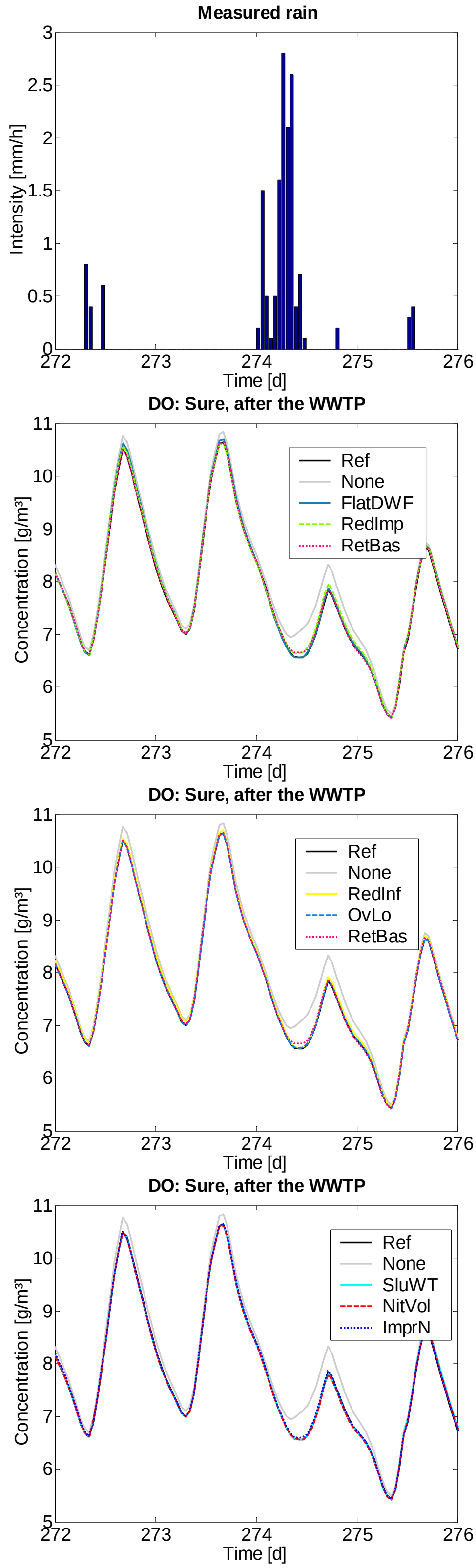


Figure 6.9: DO concentrations in the Sûre after the WWTP. Curves overlap as the effect of scenarios on immission concentrations is negligible.

Table 6.6: Evaluation matrix for emissions from the WWTP: NH_4-N and TN thresholds 15 mg/l, PO_4-P threshold 2 mg/l (see section 6.3.1 for explanation of the matrix).

	COD					NH					TN					PO				
Emission	Load	Mean	Max	D125	F125	Load	Mean	Max	D15	F15	Load	Mean	Max	D15	F15	Load	Mean	Max	D2	F2
Max	111.1	52	89.7	0	0	35.6	13.4	48.0	100.8	195	68.5	40.3	108.2	238.0	59	2.6	1.2	8.0	31.7	65
Min	80.2	42	61.5	0	0	15.9	5.5	28.5	16.0	103	57.0	24.6	64.3	216.6	14	1.8	0.9	1.5	0.0	0
Ref	106.2	43	72.3	0	0	33.0	12.9	37.8	94.0	178	67.1	28.5	79.4	219.8	52	2.4	1.0	6.8	21.9	63
FlatDWF			0.85			0.81	0.81	0.91	0.78	0.83			0.94		0.46	1.07	1.09	1.18	1.45	0.92
FlatNH						0.92	0.89	0.95	0.93	1.06					0.60					
RedImp																		0.93	1.05	
InfRed	0.76	1.21	1.16			0.71		1.27	1.07	1.10	0.87	1.41	1.36	1.08	0.27	0.73	1.17		1.32	
RetBas			1.10			1.08									1.10				0.83	
SluBu																		0.75	0.88	
SluWT						0.75	0.75	0.75	0.66	0.94	0.85	0.86	0.81		1.13			1.11		
NitVol						0.59	0.55		0.45	0.94					0.75	1.08	1.09			0.83
OvLo			1.24							1.05					1.06			1.10	0.75	
ImprN						0.48	0.43		0.17	0.58			1.08					1.05		0.92
ImprP						0.86	0.85		0.86							0.91	0.93	0.22	0.00	0.00

by improving nitrification at the WWTP, a small change in mean concentrations in the river can be obtained, the base pollution is so high that investment needs to be done upstream of the Bleesbruck catchment. Actually major WWTPs are currently being rebuilt, so that upstream pollution in the Alzette will soon be reduced.

As before for nitrogen, the river's *phosphorus* content stems from upstream rather than this urban catchment. Apart from **ImprP**, phosphorus concentrations are affected especially again by the **InfRed** scenario (see Figure 6.10). The temporarily higher concentrations in orthophosphate for the improved control can be explained by looking at WWTP emission concentrations. These show that, with the on/off control in the reference scenario the amount of aluminate released in one time is so high that it gets concentrations extremely low for longer time so that it stays below the controlled concentration of the improved control scenario (also see section 6.4.1.2).

6.4.1.2 Emissions

Table 6.6 shows that in terms of WWTP emissions, *ammonium* results demonstrate the most significant changes for scenarios. Before any investigation of single scenarios, it is pointed out again that the WWTP nitrification does not funtion in a satisfactory way and exceeds TN emission values during 90% of the time (220 days out of 245 simulated days). The scenario **InfRed** is the only scenario that, although it slightly increases effluent concentrations, reduces the emission loads for all components. The increased concentrations of the incoming wastewater increase reaction rates at the WWTP. This is an interesting scenario as it does not represent an improvement in terms of emission concentrations as asked for by the urban wastewater directive (CEC (1991)), but it performs well within the WFD context.

Figure 6.11 shows 2 bargraphs representing relative values for concentrations and loads of ammonia, nitrate and dinitrogen. The significant increase of nitrate and dinitrogen concentrations and loads for the scenario **NitVol** show the improved nitrification and denitrification taking place through the higher residence times in the biological unit. **ImprN** and **NitVol** reduce am-

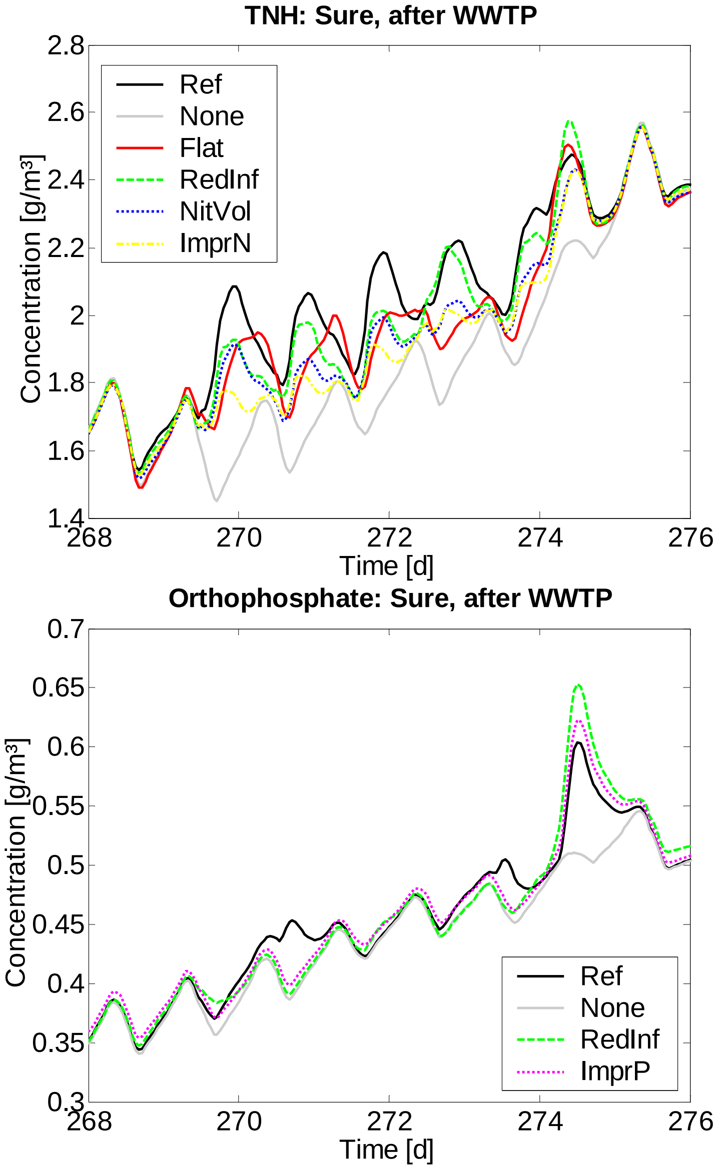


Figure 6.10: Total ammonium (left) and orthophosphate (right) concentrations in the river.

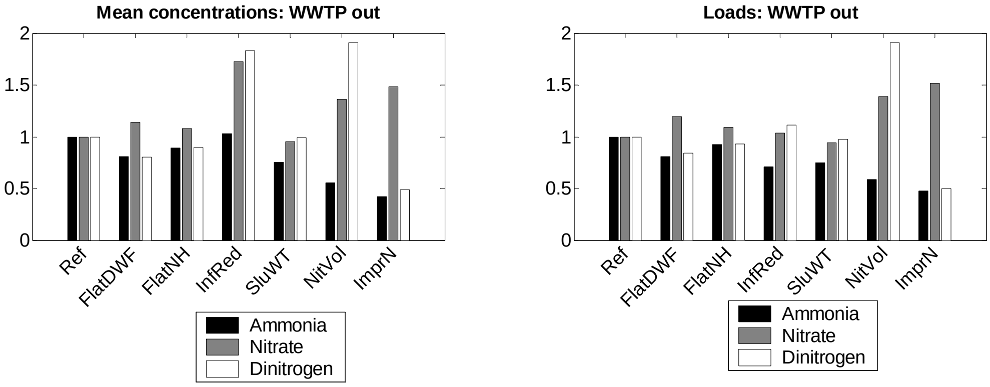


Figure 6.11: Emission concentrations (left) and loads (right) of ammonia, nitrate and dinitrogen from the WWTP for different scenarios.

monium emissions more than half and a little less than half respectively, both in terms of loads and concentrations. The reduction in *TN* emissions in scenario **SluWT** is easily explained by the fact that a certain quantity of ammonium is taken out of the modelled system by assuming onsite sludge reject water treatment. For the **InfRed** scenario, more nitrogen ends up in the waste sludge. **FlatDWF** and **FlatNH** do improve overall treatment efficiency at the WWTP.

Phosphorus emission loads and mean concentrations are only little decreased by **ImprP**. This is due to the fact that the improved control was designed to react quicker to incoming peaks so as to be able to eliminate them, which is reflected in the duration and frequency criteria (see also Figure 6.12). More precise dosage of aluminate is performed so that concentrations in the tank will stay at the fixed value of 1 mg/l and simulated pumping data revealed that more than 25% of the aluminate can be saved (see Figure 6.12). **RetBas** and **OvLo** do not significantly influence emissions but reduce the duration above threshold for orthophosphates, which is linked to the dilution of peak concentrations.

6.4.2 CSO discharges and effects

The analysed CSOs are the 2 most important ones in the sense that their connected drainage areas are the largest in the 'Bleesbruck' catchment and that they will be or have recently been connected to a storage tank. The 'Ettelbruck' CSO is an often overflowing CSO as its overflow limit is below the maximum DWF in winter when infiltration is high. Besides peak flows from the Diekirch catchment itself, the 'Diekirch' CSO is overflowing in case the collector is full during rain events (backwater effects).

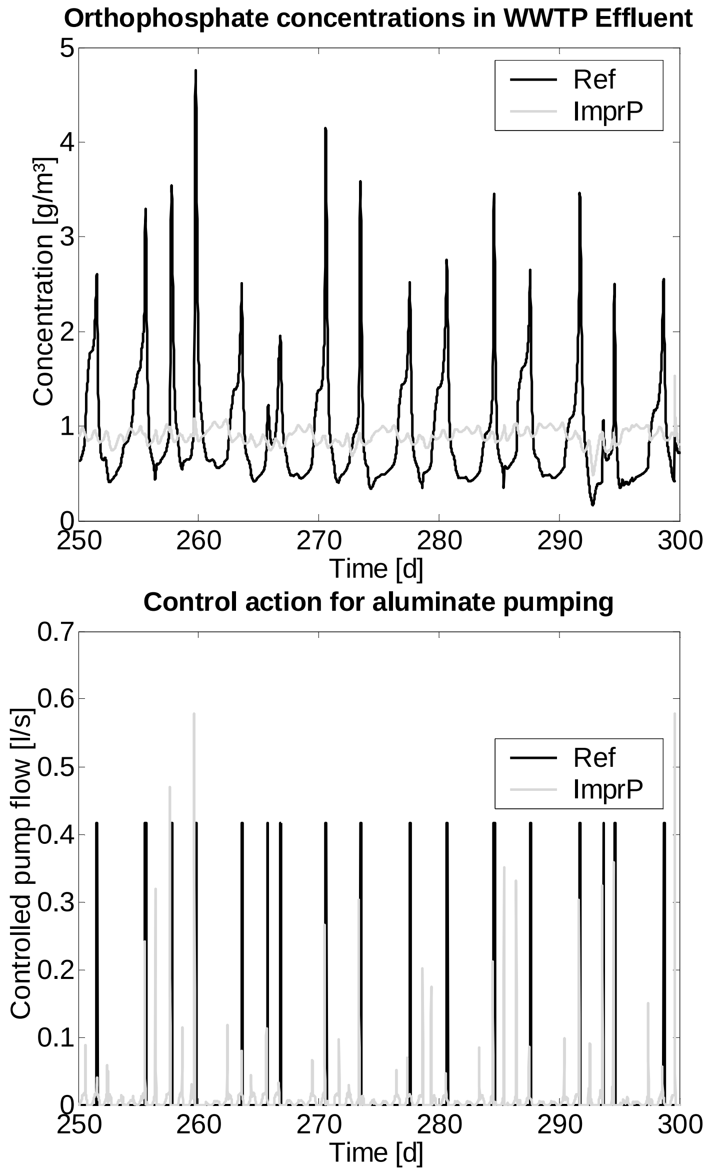


Figure 6.12: WWTP emissions of phosphate concentrations (left) and the controlled pumping rate for aluminate (right).

Table 6.7: Evaluation matrices for immission after the 'Ettelbruck' and the 'Diekirch' CSO: Simulation results with DO threshold 5mg/l, NH_4-N threshold 2 mg/l, PO_4-P threshold 0.4mg/l (see section 6.3.1 for explanation of the matrix).

Immission	DO				NH				PO				COD	
CSO Ettelbruck	Mean	Min	D5	F5	Mean	Max	D5	F5	Mean	Max	D5	F1	Mean	Max
Max	10.6	7.8	47.6	117	1.9	3.2	32.0	61	0.4	0.7	167.8	74	37.8	97.1
Min	6.9	2.9	0.0	0	1.8	3.2	18.0	50	0.4	0.6	138.8	52	35.4	62.7
Ref	9.1	3.3	25.2	59	1.8	3.2	18.5	61	0.4	0.7	141.0	73	37.9	92.4
None										0.87		0.93		0.68
FlatDWF														1.05
FlatNH3														
RedImp														0.76
InfRed														0.95
RetBas										0.94				0.68
Sha	0.76	0.88	1.89	1.98	1.08		1.73	0.82	1.06		1.19	0.71	0.94	
Reae	1.16	2.35	0.00											0.94

Immission	DO				NH				PO				COD	
CSO Diekirch	Mean	Min	D5	F5	Mean	Max	D5	F5	Mean	Max	D5	F1	Mean	Max
Max	11.3	7.5	3.2	3	1.4	2.8	32.0	20	0.39	0.62	106.0	58	35.56	66.7
Min	7.9	3.6	0.0	0	1.2	2.8	18.0	17	0.35	0.56	74.0	36	32.57	56.9
Ref	10.4	4.6	0.2	0	1.2	2.8	18.5	20	0.36	0.61	77.6	56	35.56	62.3
None										0.93		0.91		
FlatDWF														1.07
FlatNH3														
RedImp												0.95		
RedInf														
RetBas														
Sha	0.76	0.79	19.00	3.00	1.14		1.73	0.85	1.09		1.37	0.64	0.92	0.91
Reae	1.09	1.62	0.00											

6.4.2.1 Immission

Immission concentrations after CSO structures are little affected by overflow events as shown in Table 6.7. Similarly to the situation in the river downstream the WWTP, only the scenarios **Reae** and **Sha** show significant positive and negative effects respectively. The best scenario is the **None** case, which shows changes in DO and maximum COD. In Ettelbruck, improvement is possible for maximum COD values in the **RetBas** and the **RedImp** cases, but as already mentioned before, maximum values have to be treated with care as the 'extreme' event might not be representative for other events and the extreme event in the reference scenario might not be the extreme event in the considered scenario. Figure 6.13 shows effects on river COD and DO for the rain event that happened during the second measurement campaign (see Chapter 4). DO concentrations in the river are nearly not affected, and spilled volumes are not important enough compared to river flow. Looking at COD, it shows that in the case of **FlatDWF**, the COD discharge at night is worse than in the reference scenario, so that in case of a rain event at night, control of the basins should receive special attention. The Diekirch CSO overflows for a longer time span due to the backwater effects in the collector, and these can be dampened by the **RetBas** scenario. It should be repeated that the response of the retention basin will be different depending on antecedent rain events and the intensity of the rain event. Figure 6.13 also illustrates that dilution through higher flow in the river happens only after the CSO event has already passed, therefore not helping to reduce the impact of the CSO.

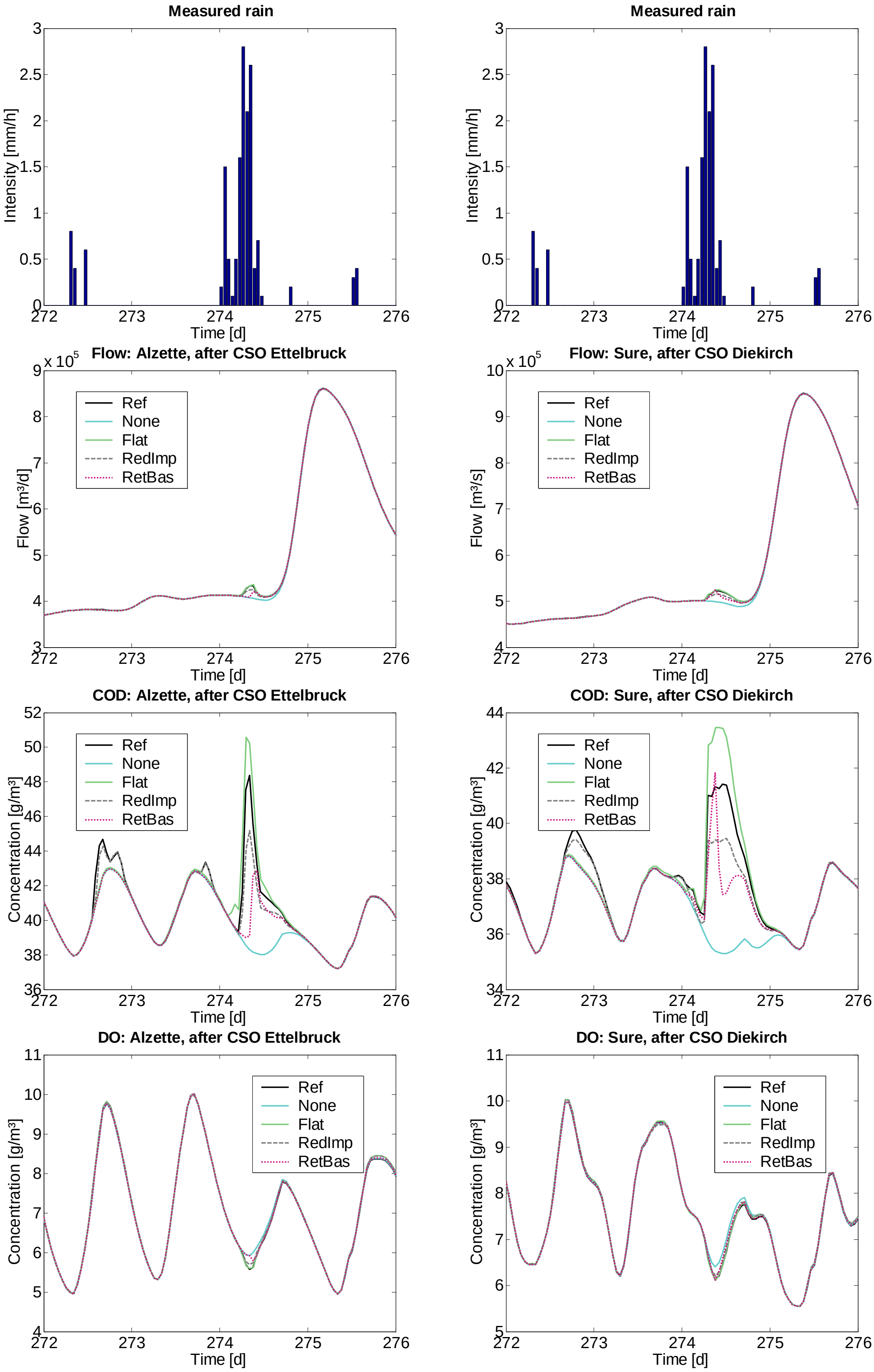


Figure 6.13: Rain (top), river flow (middle-top), COD (middle-bottom) and DO (bottom) concentrations in the rivers after the CSO structures Ettelbruck (left) and Diekirch (right).

Table 6.8: Evaluation matrices for emissions at CSO Ettelbruck and CSO Diekirch (see section 6.3.1 for explanation of the matrix).

Emission	Water					COD part			NH			PO		
CSO Ettelbruck	Vol	Mean	Max	D	F	Load	Mean	Max	Load (kg)	Mean	Max	Load (kg)	Mean	Max
Max	120069	490.1	406954	101.7	153	55.7	284	3838	1467.1	9.0	41.6	1467	1.26	5.84
Min	20572	84.0	335624	8.2	53	3.2	1	258	76.3	0.0	6.5	76	0.01	2.19
Ref	119967	489.7	406954	71.9	153	55.7	208	3628	1467.1	6.5	41.6	1467	0.93	5.56
FlatDWF				1.41	0.35	0.93	1.36		0.91	1.38	0.76	0.91	1.35	0.76
FlatNH3									0.89	0.89	0.74			
RedImp	0.79	0.79	0.82			0.86		0.84	0.92			0.92		
InfRed	0.58	0.58	0.95	0.11	0.51	0.33	0.08	1.06	0.22	0.06		0.22	0.10	
RetBas	0.17	0.17	0.91		0.82	0.06	0.00	0.07	0.05	0.00	0.16	0.05	0.01	0.39
OvLo														
Emission	Water					COD part			NH			PO		
CSO Diekirch	Vol	Mean	Max	D	F	Load (kg)	Mean	Max	Load (kg)	Mean	Max	Load (kg)	Mean	Max
Max	13160	53.7	244929	1.3	55	3384.8	1.3	2121	54.6	0.0	11.9	28.5	0.012	4.3
Min	5712	23.3	202265	0.4	9	1034.1	0.2	374	17.7	0.0	7.7	10.5	0.003	3.0
Ref	12969	52.9	244181	1.27	52	3183.8	1.3	2026	48.7	0.02	11.90	26.6	0.012	4.3
FlatDWF					1.06	1.06	1.07		1.12	1.06	0.65	1.07		0.69
FlatNH3											0.92			
RedImp	0.62	0.62	0.83	0.67	0.77	0.61	0.67		0.60	0.66		0.60	0.66	
InfRed								0.94			0.94			
RetBas	0.44	0.44	0.95	0.30	0.17	0.32	0.17	0.18	0.36	0.26	0.70	0.39	0.28	0.77
OvLo														

6.4.2.2 Emissions

Emission evaluation matrices of the 2 CSO structures are given in Table 6.8. Looking at numerical values of the **Ref** case and scenario minima/maxima values for the different criteria, a difference in order of magnitude is noticed between the 2 CSOs both in volumes spilled and in pollution discharged. Ettelbruck is indeed an often overflowing CSO, due to the fact that its overflow limit is close to DWF quantities, so that in March for example, when infiltration into the sewer system is still high, the CSO is overflowing even during DWF conditions.

Hence, for the Ettelbruck CSO, **InfRed** already reduces overflows in DWF conditions. **RetBas** has a very good performance in Ettelbruck too, both for water and pollutants, as it collects the spilled DWF even though this is not the primary role of a retention basin. Scenario **FlatDWF** should not reduce the volume and pollution spilled on average. However, through consistent load increase for all pollution components, Diekirch shows that there is a small tendency for rain, i.e. overflows, to happen at night. **FlatNH** reduces ammonium discharge in Ettelbruck due to smaller NH concentrations during high DWF peaks.

Table 6.9 gives results for the sum of all CSO volumes and loads. The **OvLo** scenario reveals that 12% of the total volume of spills occurs before the WWTP. Except for DWF and ammonium flattening scenarios, all measures considerably reduce untreated discharges. This table can be used from an immission point of view of a vulnerable river where <u>no</u> CSOs can be accepted.

Table 6.10 compares the results for the sums of both CSO and WWTP emission loads. Clearly 'best' scenario in terms of emissions is **InfRed**. Hydraulic volume reduction and pollutant concentration increase during DWF can improve the treatment capacity of ammonium. Even stronger improvement of ammonium emissions is achieved by **NitVol** and **ImprN** which are cheap to implement (see section 6.2). Although for **RedImp** water entering into the sewer system is reduced by less than 5% and for **RetBas** the volume of water in the system is the same, total COD and PO emissions are reduced.

Table 6.9: Evaluation matrix for total CSO emissions (see section 6.3.1 for explanation of the matrix).

	Water			COD	NH	PO
Total CSOs	Vol	D	F	Load	Load	Load
Max	420014	147.7	753.0	142.3	3.1	0.90
Min	312136	37.0	627.0	88.1	1.7	0.62
Ref	420014	115.6	753.0	142.3	3.1	0.90
FlatDWF		1.28	0.84		0.95	
FlatNH3					0.95	
RedImp	0.77	0.93	0.86	0.76	0.83	0.75
InfRed	0.81	0.32	0.83	0.66	0.55	0.76
RetBas	0.74		0.89	0.62	0.58	0.69
OvLo	0.88			0.83	0.82	0.85

Table 6.10: Evaluation matrix for total emissions from CSO structures and WWTP (see section 6.3.1 for explanation of the matrix).

	Water	COD	NH	PO
Total Emission	Vol	Load	Load	Load
Max	2857895	260.5	37.4	3.5
Min	1884697	181.1	19.0	2.5
Ref	2853886	260.5	36.2	3.3
FlatDWF			0.82	
FlatNH3			0.93	
RedImp		0.86		0.93
InfRed	0.66	0.70	0.70	0.74
RetBas		0.79		0.94
SluBu				
SluWT			0.77	
NitVol			0.62	
OvLo		0.91		
ImprN			0.52	
ImprP			0.88	0.94

6.5 Scenario Analysis 2: Low Base Pollution in the River Alzette

To test the impact of our urban catchment in case upstream conditions of the river Alzette would have been improved through both implementation of the WFD and emission compliance within the Urban Treatment Directive (CEC (1991)) of WWTPs upstream the 'Bleesbruck' catchment, the same 15 scenarios have been run supposing that the river Alzette (high base pollution) has the same 'good' water quality as the river Sûre already has now. Reference conditions for the Alzette, which, according to the WFD, are to be determined for rivers and lakes (REFCOND (2003)) could not be evaluated within this project. Therefore, the model described in Chapter 5 section 5.4, remains unaltered apart from the upstream boundary input data to the Alzette, now identical to the Sûre's input data. Kinetic parameter values of the simplified river water quality model are put back to default parameter values for all receiving river models. Indeed, in Chapter 5 section 5.4.4 it was explained that the algae growth parameter had to take a high value in order to account for sessile algae. This is not needed anymore once water quality is such that eutrophication is no longer present.

To see the impact on durations and frequencies above thresholds for ammonium, the threshold concentration has been set to 0.6 mg/l, as Figure 6.7 shows that, for such ammonium concentrations, under high temperature and pH=8, toxic ammonia levels can be reached. For phosphorus, the threshold was now fixed to 0.3 mg/l.

6.5.1 Immission-emission

Table 6.11 shows immission results in the Sûre after the WWTP discharge point. The **None** scenario proves that the impact of the urban catchment is considerable, especially for duration and frequency of the variable thresholds. In comparison with Table 6.5, the catchment impact is larger than for Scenario Analysis 1 (SA1), especially for DO, as here the threshold has been kept the same (5mg/l). Table 6.12 contains immission results for the CSO discharge points. The **None** scenario now shows influence of CSOs on the river, which was not the case in SA1.

Mean DO concentrations after the WWTP have sunk from 11.5 in SA1 to 6.6 mg/l in SA2. Although the daily fluctuation in concentrations is not as high anymore than they were for the existing situation, the river is now, due to the lower algae mass and therefore absence of supersaturation, much more vulnerable with regard to DO depleting pollution from the WWTP. Although the duration of DO concentrations below the threshold in the reference scenario has not increased compared to the original reference case, they are now mainly caused by the urban catchment discharges and not by conditions upstream the catchment (see **None**). Indeed, the river's state of eutrophication assured high DO concentrations during the day when the WWTP is discharging its highest load. Now, with DO concentrations being lower, the 5 mg/l threshold is more easily crossed.

From Table 6.11, scenario **OvLo** seems to indicate that although the WWTP has to treat more water and therefore might discharge higher loads and concentrations, the dissolved oxygen

Table 6.11: Evaluation matrix for immission after the WWTP: Simulation results with DO threshold 5mg/l, NH_4-N threshold 0.6 mg/l, PO_4-P threshold 0.3 mg/l (see section 6.3.1 for explanation of the matrix).

Immission WWTP	DO				NH				PO				COD	
	Mean	Min	D5	F5	Mean	Max	D06	F06	Mean	Max	D03	F03	Mean	Max
Max	8.5	6.7	6.1	10	0.32	1.09	10.3	41	0.25	0.45	41.71	33	18.3	48.5
Min	6.5	3.9	0.0	1	0.13	0.27	0.0	2	0.23	0.35	18.96	4	17.3	23.7
Ref	6.6	3.9	4.3	7	0.31	0.96	8.3	40	0.25	0.40	40.04	31	18.2	45.1
None		1.06	0.26	0.14	0.41	0.28	0.00	0.05	0.94	0.87	0.47	0.13		0.53
FlatDWF			0.81		0.87	0.82	0.25	0.43				0.87		
FlatNH3						0.90	0.63	0.68						
RedImp			0.89	0.86			0.87	0.80						0.87
InfRed			0.68	0.57	0.83	1.14	0.71	0.70		1.11	0.83	0.81		
RetBas			0.77	0.71			1.23	0.95		1.08		0.77		
SluBu			0.75	0.71			0.26	0.35			0.93	0.90		
SluWT			0.84	0.86	0.86	0.84		0.90				0.87		
NitVol			0.88		0.77	0.84	0.27	0.40				0.90		
OvLo			0.85	0.86		1.09						0.90		
ImprN			0.58	0.57	0.71	0.85	0.15	0.28				1.06		
ImprP				0.86	0.93	0.93	0.76	0.70			0.89	0.65		
Sha			1.40	1.43			1.06							
Reae	1.28	1.70	0.00	0.29										1.07

Table 6.12: Evaluation matrices for immission after the 'Ettelbruck' and 'Diekirch' CSO structures: Simulation results with DO threshold 5mg/l, NH_4-N threshold 0.6 mg/l, PO_4-P threshold 0.3 mg/l (see section 6.3.1 for explanation of the matrix).

Immission CSO Ettelbruck	**DO**				**NH**				**PO**				**COD**	
	Mean	Min	D5	F5	Mean	Max	D06	F06	Mean	Max	D03	F03	Mean	Max
Max	8.97	7.50	5.50	16	0.16	0.67	0.0	1	0.2	0.5	25.2	23	17.7	89.0
Min	6.79	3.56	0.00	0	0.14	0.32	0.0	1	0.2	0.4	21.5	4	17.3	25.0
Ref	6.88	3.56	4.54	14	0.16	0.61	0.0	1	0.2	0.5	24.8	22	17.7	86.5
None		1.22	0.82	0.93	0.93	0.52				0.77	0.87	0.18		0.29
FlatDWF						1.10				1.07				
FlatNH3														
RedImp						0.79				0.91		0.82		0.76
InfRed						0.91						0.91		
RetBas					0.95	0.81				0.84	0.91	0.50		0.47
Sha			1.21	1.14										
Reae	1.30	2.10	0.00	0.00										

Immission CSO Diekirch	**DO**				**NH**				**PO**				**COD**	
	Mean	Min	D5	F5	Mean	Max	D06	F06	Mean	Max	D03	F03	Mean	Max
Max	9.2	7.7	0.17	1	0.16	0.54	0.00	2	0.24	0.42	26.42	17	18.2	56.6
Min	7.1	4.1	0.00	0	0.15	0.30	0.00	2	0.24	0.35	20.79	4	17.8	24.6
Ref	7.2	4.1	0.17	1	0.16	0.51	0.00	2	0.24	0.41	25.71	17	18.2	50.4
None		1.18	0.00	0.00	0.94	0.60				0.87	0.81	0.24		0.49
FlatDWF						1.06						0.94		
FlatNH3														
RedImp						0.76					0.94	0.82		0.89
InfRed											0.94	0.94		1.10
RetBas											0.87	0.76		
Sha						1.06								1.12
Reae	1.27	1.90				0.93								1.07

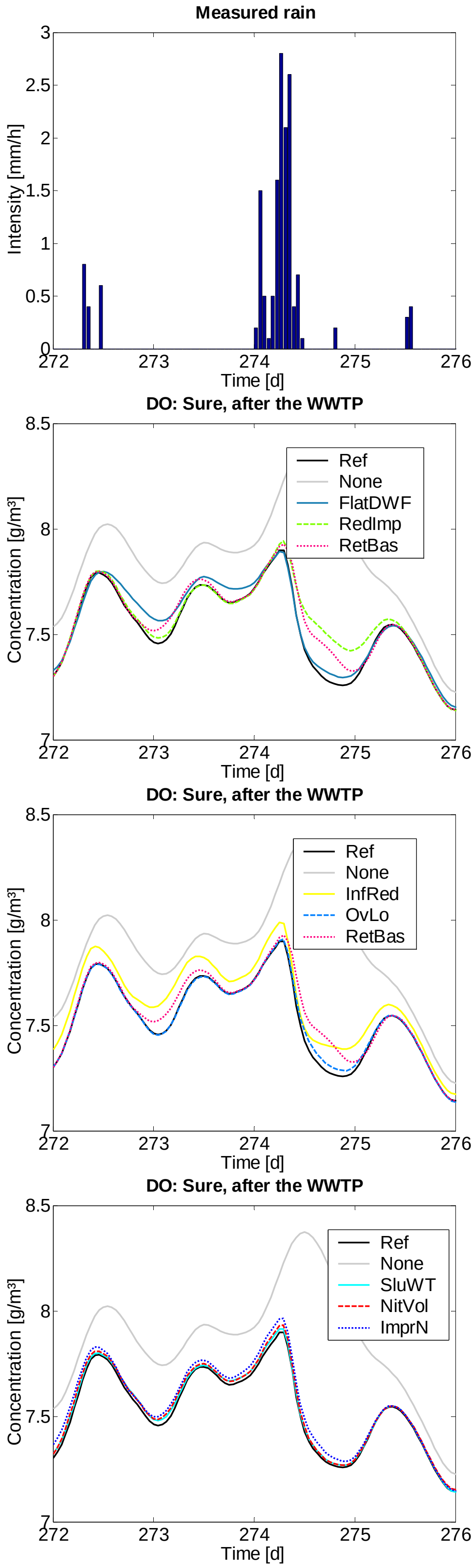


Figure 6.14: DO concentrations in the Sûre after the WWTP.

is more affected by the discharges of the CSO prior to the WWTP than the WWTP itself. Total emissions from Table 6.10 indeed report 10% of total COD reduction. **RetBas** reflects a similar result in the sense that reducing untreated discharges from Diekirch shortens the time during which concentrations stay below 5 mg/l for DO. However the nitrification capacity decreases due to the prolonged higher flow to WWTP. Best performing scenarios in terms of ammonium are **InfRed**, **ImprN** and **NitVol** similar to the evaluation in SA1. Scenarios **Sha** and **Reae** show similar but reduced effects than for the original set up of the model.

6.5.2 Event-based analysis

To look at what happens inside the river, it makes sense to look at concentrations in function of time, as was already done in SA1 (see Figures 6.11, 6.13 and 6.14).

Figure 6.14 illustrates the effect of the different measures on DO concentrations after the WWTP in the river. Overall, flows in the river are so high compared to the WWTP effluent that the contribution of the catchment is little. However it can be observed that there is a consistent difference in DO for all scenarios with regard to DO for the **None** case, something that could not be observed in SA1 (see Figure 6.9). This impact must mainly be due to WWTP emissions, especially from COD related pollutants. **InfRed** shows improvement for DO all the time, through consistent reduction of COD discharges. **RedImp** and **RetBas** show their benefit during the rain event of day 274.

For NH immission, Figure 6.15 reveals that the high ammonium discharges from the WWTP now have considerable effect on the ammonium concentrations in the river. From Table 6.11, the **None** case allows for 60% improvement on the mean concentration. Also seen from the Figure is that concentrations have even more effect in summer than in winter, due to low flow in the river. The threshold of 0.6 mg/l is crossed for rain events and even in late summer for DWF. It is clear that such an impact requires WWTP upgrade. When zooming in on rain events, **RedInf** shows slightly higher peak concentrations than the **Ref** scenario during rain events (explained in section 6.4.1.1) so that this scenario would certainly need improved control, for example, to adapt for maximal immission-based effluent concentrations.

To see whether on a shorter time scale, DO concentrations are affected by CSOs, Figure 6.16 uses the rain event as was used in SA1 and shown in Figure 6.13. Changes are very small, but again larger than in SA1. They reveal the same trends as the matrices did: Both **RedImp** and **RetBas** reduce COD discharges and reduce the impact on DO. However, apart from some improvement in the durations above the threshold for ammonium and phosphate due to measures in the catchment, the high river flow compared to the overflow quantities seem to be able to deal with the incoming point pollution.

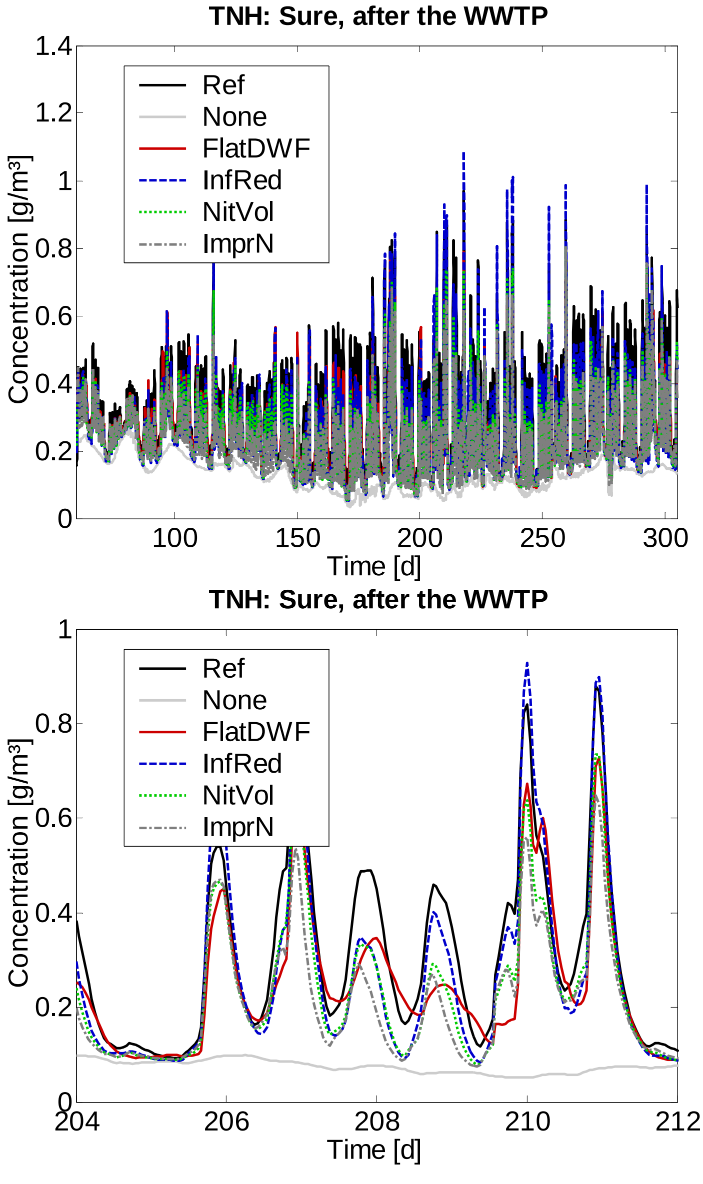


Figure 6.15: TNH concentrations in the Sûre after the WWTP

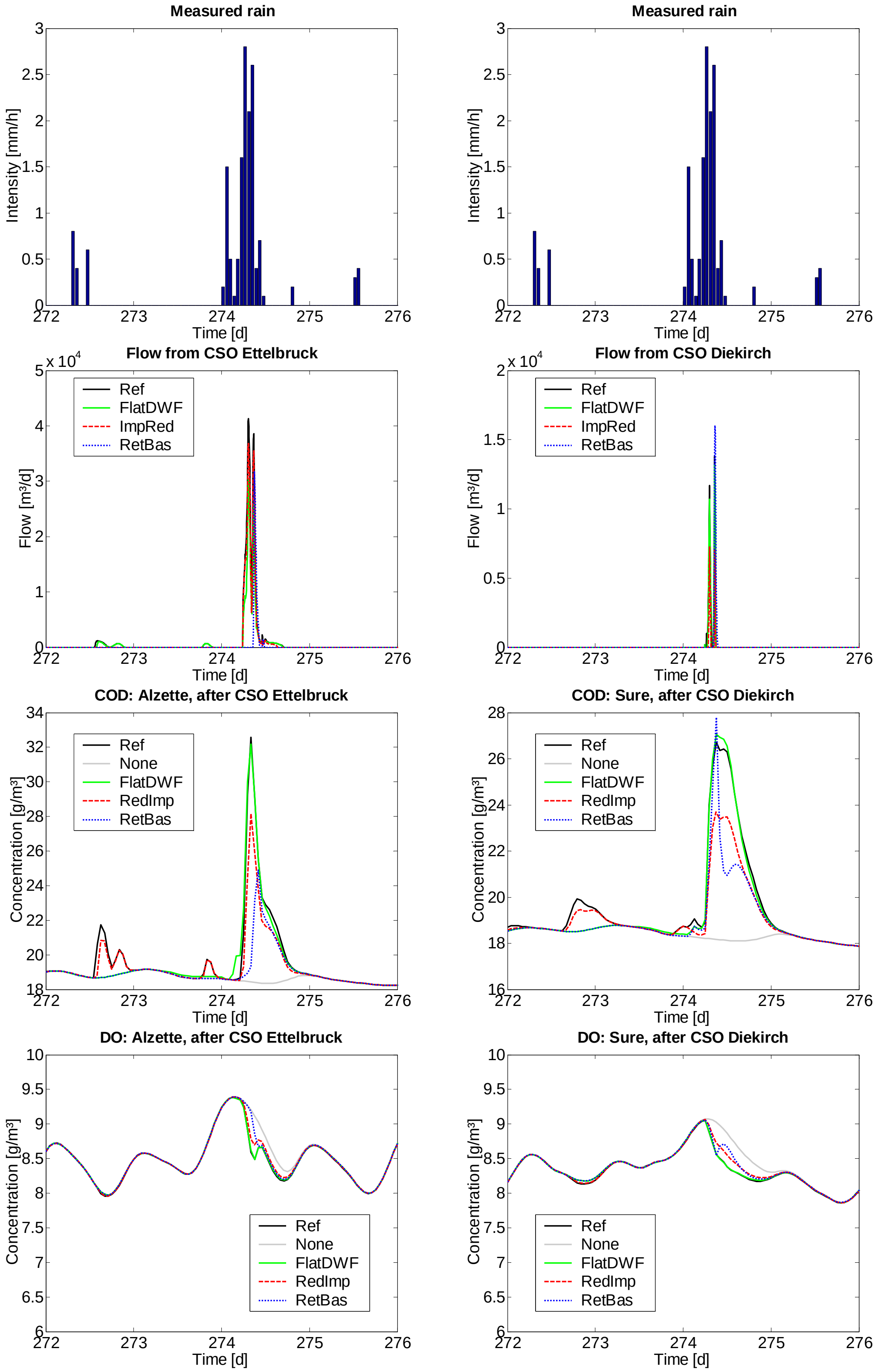


Figure 6.16: Rain (top), CSO overflow (middle-top), COD (middle-bottom) and DO (bottom) concentrations in rivers after the CSO structures in Ettelbruck (left) and Diekirch (right).

6.6 Discussion

Scenario analyses have confirmed that in this case study, investments for implementation of the WFD need to be done in a first instance upstream of the Bleesbruck catchment, i.e. requiring emission-immission based upgrade of the treatment facilities of the city of Luxembourg and others. These are currently being rebuilt, as well as the 'Bleesbruck' WWTP will have to be adapted. Taking into account the present background pollution, implementation of *improved control algorithms* for nitrogen and phosphorus removal at the treatment plant present good results at relatively low costs and can bring about positive changes with regard to peak reduction or even elimination. Consequently it can reduce the risk of ammonia fish intoxication and, as phosphorus is the limiting nutrient for algae growth, decrease algae mass in the vicinity of the WWTP.

The second scenario analysis has shown that once goals for WFD implementation are reached in the Alzette, the catchment and especially the existing WWTP, even with the high dilution from the Sûre, have more impact upon river water quality. Especially in terms of ammonium, river concentrations are more than doubled and reflect the daily effluent pattern of the WWTP effluent. Regarding DO concentrations, the river has become more vulnerable to CSOs and WWTP emissions due to reduced algae presence and absence of supersaturation. From the simulations it could be seen that COD peak emissions have an impact on DO concentrations.

A clear distinction can be made between scenarios that are realistic to be implemented now and scenarios that present advantages and deserve serious consideration in future planning processes. Certainly not all scenarios are attractive for the here considered case study, but from the interpretation of the results, several of their advantages and disadvantages can be extrapolated.

The implementation of *shading* along river banks proved not to be an appropriate solution for this river system, as the downstream oxygen demand is too high and will provoke low DO concentrations at night. This obviously has to be seen within the here analysed context, as the planting of trees can be a good measure for other, ecological reasons. Local *reaeration* in the river however produced good results. In a more detailed study of the river Alzette, exact locations for application and aeration control schemes could be determined. Nevertheless such end-of-pipe water management option should be regarded as a temporary option, and ideally be combined in an integrated way with direct reduction of pollution at the source, i.e. in order not to only reduce symptoms but take away the causes of eutrophication.

Flattening of flow or pollution peaks shows small improvements, but they are very costly scenarios and certainly not suitable in this case. However, the construction of local retention basins can be interesting to weigh performance and costs of such tanks at newly planned housing or industry zones, especially if they are considered for having their own local treatment system instead of investment into pipes for connections to existing networks. Also, such measures should not only control the flow but be combined with water saving appliances, both to save water resources and to increase wastewater concentrations. For example, pollution separation is suited for decentralised treatment of urine in rural areas, which the Bleesbruck catchment is not.

The *infiltration reduction* shows considerable improvement for all components through the consistent reduction of effluent loads. It minimises incoming water during DWF and increases treatment efficiency. It represents a sustainable option for implementation (it can be seen as a source control option) and is special as by reducing incoming 'clean' water, a considerable amount of pollution will not enter the receiving system. For old centralised systems, the measure presents a good alternative to the construction of new infrastructures (land costs). It should indeed be considered as a measure that is constantly implemented over time, i.e. as a maintenance of the sewer network. The downside is possibly higher WWTP effluent concentrations during wet weather. They can, at least momentarily, create high concentrations that could become toxic for fish. Hence, emission concentrations need to be immission related (i.e. how much water is there to dilute discharges).

Apart from measures at the WWTP explicitly reducing ammonium emissions, measures in the sewer system, although costly, can consistently reduce loads discharged from the urban wastewater system. *Impervious surface reduction* lowers incoming water peaks during rain reducing untreated discharges and disturbance of treatment. Hence, keeping the impervious surfaces to a minimum during planning processes is beneficial with regard to the system's immission concentrations. *Storage tanks* will send the otherwise discharged water to the WWTP for treatment. Without however assessing the receiving water quality at discharge locations, the question of whether early untreated discharge or increased loading of the WWTP is the optimum solution remains to be solved for each individual case study. In the here presented case study, WWTP emission loads do not show significant increase and especially for COD, the WWTP Bleesbruck still complies with emission standards. This is only valid in the limits of the model, as clarifiers are modelled as the ideal settlers. Results show that the duration of DO immission concentrations below the threshold become shorter when avoiding CSOs and sending this wastewater through the WWTP (overloading).

Hence, depending on the identified problems in the river, different measures can be chosen. From the here investigated case study it can be derived that: in case nitrogen is the limiting nutrient for eutrophication, treatment of ammonium at the WWTP is crucial; if phosphorus is the limiting factor, both the treatment of the latter at the WWTP needs to be optimised, and the CSO events reduced depending on concentrations in the sewage. In case of DO depletion danger in the river, next to COD removal at the WWTP, measures in the catchment like minimising impervious surfaces and construction of storage tanks become relevant.

A suggested approach to analyse a case study where an integrated model is available can be the following: For WFD implementation a long term immission-based analysis using scenario simulations is proposed. Condensation of results in a matrix provides an excellent overview on the general situation. Simultaneous consideration of emissions will allow for understanding of interactions and impacts. An event-based analysis will help to visualise what was hidden behind the matrix. For cases where NO overflows are allowed for example, or where a minimum of phosphorus is to be released (and peaks might be unimportant), total emission results from WWTP and CSOs can be very useful.

6.7 Conclusions

Using a dynamic modelling approach, various scenarios have been tested. Results were evaluated according to means, maxima, minima, frequency and duration. The complexity lays within the many locations and criteria the assessment can be done with. Duration and frequency of DO depletion below some threshold are suitable and important in rivers with fish species that are especially vulnerable to oxygen content or intoxication risks. A main observation made during the evaluation of results is that all of the criteria are to be considered together. A reduced mean value does not tell anything about the variation of the considered variable, and an increased frequency does not include any information on the duration above/below thresholds. Therefore the matrix format is very appropriate.

Some of the found results were predictable, nevertheless others were unexpected. The learning process through the analysis, both in terms of the case study as well as on mechanisms within the integrated urban wastewater system is large. Indeed, inspection of results at various locations not only illustrate the various effects that subsystems have on each other, but help to understand connections in the changes, hence the processes in the models.

One should certainly bare in mind the limits of this model. It was constructed for a scenario analysis focussing on biochemical water quality. With this model, conclusions can not be drawn for hydraulic impacts of CSOs on river morphology nor can it provide answers to consequences for living organism population around the effluent. If these issues are considered critical for a case study, different, more appropriate models need to be used.

Moreover, the outcomes are to be seen as qualitative and not quantitative results, as they are to be evaluated with respect to the reference state.

The selected case study was for sure not an ideal case for an immission based evaluation of an urban catchment, as pollutant base concentrations in the river are high from upstream and the river is hydraulically much more important than what comes from the catchment. However, even if DO concentrations are less affected in that case, bad treatment of ammonium at the WWTP induces concentration changes that can be observed in the river. Also, did the second scenario analysis reveal that, in case of WFD compliance of the Alzette, emissions of the current catchment gain importance. It could be shown that depending on the problems in the river, a suitable measure can be found either within the catchment, the network, the WWTP or the river.

Chapter 7

Conclusions

This final Chapter gives some general conclusions by summarising the achievements of this thesis, identifying the prospects for improvement within the here discussed case study and integrated urban wastewater system modelling in general. The Chapter ends on some general thoughts for future directions within urban wastewater management.

7.1 Achievements

The aim of the here presented work was the construction of an integrated model for a sewer-WWTP-river system in order to analyse via computer simulations the impact of various system configurations on the water quality of the receiving water. The adopted framework (see Figure 7.1) and the presentation of results were designed to go within the context of the EU Water Framework Directive implementation. Inspired from the Driver-Pressure-State-Impact-Response (DPSIR) framework, the approach was used as underlying structure to define the individual elements that constitute the system's analysis.

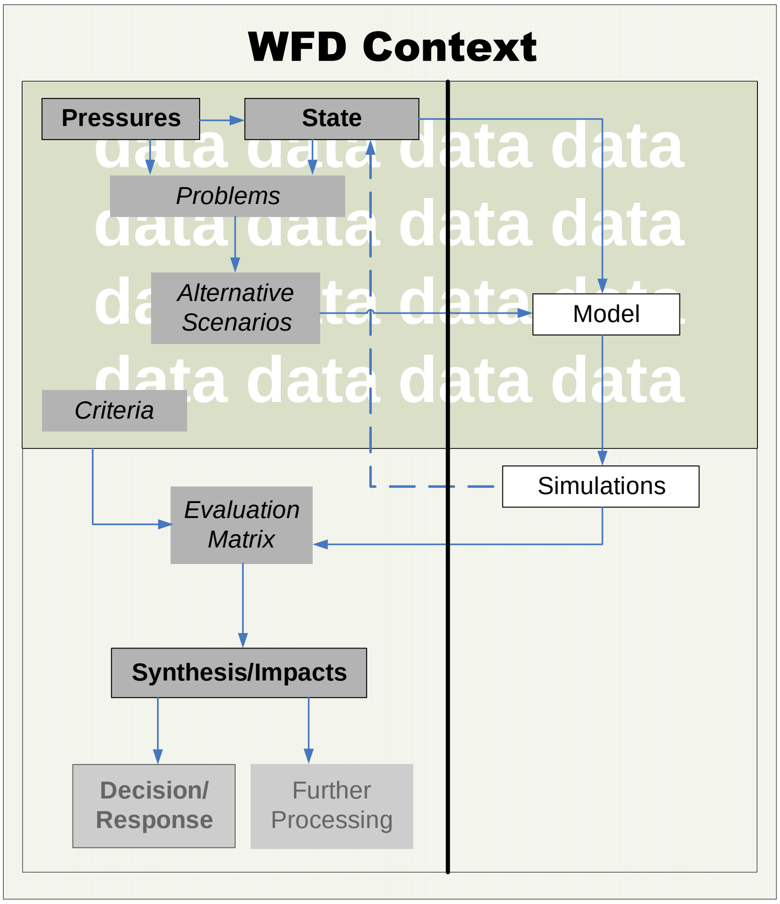


Figure 7.1: Overall scheme of the impact analysis approach adopted in this thesis.

For the here presented thesis, the following can be identified as main steps for the achievement of goals:

1. extension of the WEST® model base with the KOSIM model for urban runoff and sewer transport modelling;

2. collection of available data on the Bleesbruck case study and the execution of 2 integrated measurement campaigns;

3. construction and calibration of the Bleesbruck integrated model;

4. analysis of scenarios using the integrated model simulations.

Several measures for optimisation of the 'Bleesbruck' system performance could be tested and evaluated with respect to chosen emission and immission criteria. The measures covered source control, infrastructure rehabilitation and construction, control strategies and measures within the receiving water. Results of the scenario analysis show the usefulness of modelling within the implementation process of the WFD. For the 'Bleesbruck' case study, it highlighted that priorities for recovery of river water quality are investments upstream of the urban catchment and improvement of nitrification and denitrification through better control at the WWTP. The simulated hypothetical situation of the receiving river being compliant to the WFD requirements showed the higher sensitivity of the river to treated and non-treated urban wastewater discharges, hence the importance of immission-based evaluation in that case.

The integrated IUWS model

In order to have the necessary modules for the assemblage of all the components constituting an integrated urban wastewater system model, the conceptual KOSIM (ITWH (2000)) model was included into the WEST® (MOSTforWATER N.V., Kortrijk, Belgium) modelbase. Next to the already available biochemical conversion process models for the WWTP and the river, the latter now contains elements for simulation of urban runoff and sewer transport. This gives WEST® a harmonised modelbase for the IUWS in terms of a consistent and uniform level of complexity and makes it a user-friendly, modular tool for the study of effects and interactions among subsystems of the IUWS.

The new modelbase was used on an integrated *real* case study, where conditions (such as lack of appropriate data, etc.) had to be taken into account and could not be avoided as in a designed, hypothetical case study. Two integrated measurement campaigns were planned and conducted in order to fill information gaps and to have dynamic data for calibration of the river model. In- or exclusion of certain elements into the model, related either to unavailability of necessary information or apparent irrelevance to the aimed objectives, was explained. The process of construction and calibration was presented extensively and it illustrated that long-term calibration is important due to the seasonal variability of the system.

In contrast to the imprecise water flow predictions caused by too large rain variability in the Bonnevoie catchment, to which KOSIM-WEST was applied as well (see Chapter 3), it seemed that for the Bleesbruck case study, this variability could be averaged out by the larger size of the urban catchment.

Within the WFD implementation context, modelling of the IUWS proved to be a good approach for more detailed analysis of an urban catchment where basin-wide models cannot give precise answers on how to operate or plan urban wastewater management that takes into account river water quality. The ability for zooming in on rain events gives the modeller the opportunity to understand what is happening on short time-scales and to verify if simulated results seem plausible.

Scenario Analysis

Through interpretation of simulation results, various behaviours of the system could be explained and evaluating and interpreting the simulation results has shown to be an excellent model validation in itself. The work illustrates that the tools applied here can serve educational purposes in order to learn more about the system under study.

An underestimated difficulty encountered within the study was the determination of an evaluation method for comparison of the scenarios and the choice of suitable criteria. The scenario simulations generated long-term data on many variables, available within and at every interface of the subsystems. This overwhelming amount of information showed a new face of what can again be called 'complexity' and illustrated how easily one can get lost in interpreting the outcomes. The proposed evaluation matrix is estimated a good way to summarise some of the necessary results for this impact analysis and together with the colour scheme applied to the matrix cells, a good visual appreciation of the simulation outcomes was found.

Overall the model is expected to contribute to reported real integrated case studies, so that the investigation and implementation performed here can be used in a comparative study of the outcomes of model predictions from other studies. It allows to demonstrate the usefulness of an integrated approach and to bring forward discussion on when and where an integrated model is appropriate.

Within the context of Luxembourg, this study should function as a driver to make authorities aware of the potential of integrated models and scenario analysis, ranging from aid to decision, as an illustrative tool to test different options in water management and as focus point of interdisciplinary research and training. The implementation is done on a real case study in the country and this work certainly highlights the interdisciplinarity and the collaboration between different stakeholders, in this case, research groups, syndicates on-site, the Water Management Authority and other authorities, engineering offices, ... and hence it should be an example of the WFD implementation process in Luxembourg.

7.2 Prospects for IUWS Modelling

Bleesbruck case study

Certainly the modelling of the Bleesbruck system is an ongoing process and further refinement is necessary, first of all to include upgrades in the system and second to further investigate the appropriateness and validity of the model. From here, a new targeted measurement campaign in the sewer network could be a good initiative especially to calibrate wet weather quantity and quality. Such calibration could not only improve the results of model simulations, but also open the discussion on whether or not processes like surface accumulation and wash-off are to be included in the integrated model because of their influence on the results.

In that context, uncertainty and sensitivity assessment of model parameters will certainly be required using perturbation methods and Monte Carlo simulations. They can also guide the planning of future monitoring campaigns.

An interesting follow-up of the scenario analysis could be to use the model for elaboration of control strategies. Such control strategies should be immission related, either by direct online specific sensors in the river or through correlation of emissions to certain characteristics like for example river flow, season, antecedent dry weather period,... Such 'rules' could be identified through statistical data analysis and clustering of simulated data according to situations fulfilling certain criteria, like time of the day, filling degree in the storage tanks, nitrification at the WWTP, etc.

IUWS modelling

To model the IUWS, a definite answer on the most appropriate models to be used cannot be given. This regards the level of detail to be included in the model as well as the question on whether the heterogeneity and intrinsic randomness of processes would not better be represented by a stochastic model component or probabilistic approaches. Hence, in order to gain credibility that outcomes of such modelling exercise can deliver information on solutions in a decision process, further examples of integrated modelling are needed. For sure, also follow-ups and implementation of proposed solutions need to be monitored. Only through a more abundant application and investigation of models and their outcomes can questions regarding validity and appropriateness of a model be solved.

A next crucial issue concerns the sensitivity analysis of calibration parameters and the uncertainty assessment of model outcomes, already pointed to in the previous section. Such sensitivities and uncertainties were not evaluated within the project, and very often they remain an unsolved matter. Especially for any integrated model, the heaviness of the model makes it very time-consuming to perform uncertainty analysis. Hopefully once the model is set-up and with increasing computer capacities, such assessments become more attractive to be applied.

Modelling and monitoring need to be regarded as an alternating process. The abundant needs of quality data for integrated model calibration ask for more monitoring practice. Apparati need to become more adapted to the specific requirements (Allan *et al.* (2006)) and through stronger demands (e.g. legislative pressures, ...), monitoring tools should also become more affordable. Increased availability of databases, both spatially and temporarily distributed, and the always improving computer technology will facilitate testing models as well as related uncertainties and propagation of errors within them. One promising way forward could be remote-sensing of algae blooms or total suspended solids (Qi (2007)), especially for river-basin wide modelling.

Although the goal of this work was situated within the urban wastewater influence on natural systems, diffuse pollution is another major problem to be tackled within immission oriented assessment of river basins and the implementation of the WFD. Indeed, due to the constant upgrade of sewer systems and WWTPs over the last 20 years, diffuse pollution becomes more and more important and does of course belong into the integrative character of the WFD (Holvoet (2006), Bach *et al.* (2006)).

From the beginning of this project, the objectives were related to chemical parameter assessments in the river. However, the dynamics of chemicals are obviously directly linked to the ecological status of the river and certainly much research is still needed in order to relate ecological indicators in receiving waters to chemical indicators that environmental engineers or decision support systems can work with. In that sense the development of ecological dose-response models to simulate relationships between chemical and ecological status are a way forward in the WFD implementation process (Rekolainen *et al.* (2003)).

7.3 Future Directions

Certainly the large number of degrees of freedom allowing to pursue water-quality driven objectives promotes tailor-made solutions involving newest technologies. Indeed, although technologies will have to evolve and be developed, this has to be done while bringing them into an optimal management context. In that sense, modelling as a tool will help to illustrate and assess various system configurations, exposed to different boundary conditions that can be of geographic, economical or social nature. However, in order for modelling to become an every day tool, the models and model-based tools need to be at the best level of scientific knowledge and at the same time user-friendly. In other words, they are attractive by being reliable, by providing state-of-the-art solutions on top of being time and cost-effective.

Centralised systems are particularly suited for evaluation and optimisation within an integrated context and new approaches and techniques, like impervious surface reduction, water saving and many others can easily be tested (Harremoës (2002)). The modular nature of decentralised systems allows sized solutions but also requires good planning tools. Good design and planning at the beginning of a project can often reduce unforeseen required modifications after implementation and save considerable amounts of money. Therefore, further development in terms of available models and user-friendliness of WEST® and similar softwares, also for testing decentralised options, is a way forward in WFD compliant planning and management of wastewater.

Too often professionals are stuck in their own field of expertise and therefore do often not step back to look at the whole picture. The importance of interdisciplinarity in environmental sciences needs to be promoted both at university level as well as within the stakeholder community in the field, including natural and social sciences, engineering and operators.

Overall, models, in an engineering sense, can be regarded upon as excellent means to connect research with application in the field and could help to fill the gap between advanced findings and needs within water management, and bring together theoreticians and practitioners. Not only do models provide solutions but they can, on the one hand, serve as illustrative and explanatory tool to provide more expert knowledge to stakeholders working in the 'real' world, and can, on the other hand, provide experience related feedback to the research community. Such valuable exchange could support implementation of innovative solutions like new technologies and sanitary concepts or decentralisation of infrastructures, and promote new targeted research programmes. Over the last two centuries, a huge know-how has been gained in developed countries on rain- and wastewater evacuation, treatment and reuse. Modelling within the context of all this expertise can serve as cost-effective tool and training method to investigate solutions for developing countries. Very often, other, financially or politically related issues hamper such studies, but targeted and well-argumented cooperation projects in collaboration with authorities, industries and research are future challenges for water management world-wide.

Symbols

a	Pipe cross-section [m^2]
A	Catchment area [ha]
α	Weir constant [m$^{\frac{3}{2}}$ s^{-1}]
β	Weir exponent [-]
c	Storage loss rate for depressions [mm^{-1}]
c_d	Weir discharge coefficient [-]
c_E	Evaporation correction factor [-]
c_P	Pipe cross-section coefficient [-]
C_x	Concentration of component x [mg l^{-1}]
d	Pipe diameter [m]
d_T	Tank depth [m]
D	Depression loss [mm]
D_{max}	Maximum depression loss [mm]
e	Potential evaporation [mm day^{-1}]
$\overline{E}_y$	Mean yearly evaporation [mm]
ϵ	Filling degree of depressions at time t [-]
f	Soil infiltration capacity [mm day^{-1}]
f_0	Maximum infiltration capacity [mm day^{-1}]
f_∞	Minimum infiltration capacity [mm day^{-1}]
f_s	Sedimentation factor in tanks [-]
φ	Impervious to pervious surface fraction [-]
ϕ	Accumulation rate of particulates on surface [s^{-1}]
g	Gravity constant [m s^{-2}]
i	Rain intensity at time t [mm day^{-1}]
i_s	Sewer infiltration percentage [mm day^{-1}]
k	Linear reservoir constant [day^{-1}]
k_e	wash-off coefficient [mm^{-1}]
k_s	Pipe roughness [m]

k_+	Regeneration constant for infiltration capacity [day^{-1}]
k_-	Regression constant for infiltration capacity [day^{-1}]
κ	Deposition rate of particulates in sewers[-]
l_T	Tank length [m]
L	Pipe length [m]
n	Number of tanks in a reservoir cascade [-]
ν	Kinematic viscosity [m^2 s^{-1}]
p	Population density [inh km^{-2}]
P	Catchment population [inh]
Q	Flow rate at time t [m^3 s^{-1}]
Q_{max}	Maximum discharge of a pipe [m^3 s^{-1}]
$\overline{Q}_{DWF}$	Mean DWF [m^3 s^{-1}]
$\overline{Q}_{PE}$	Mean daily water consumption per person [l inh^{-1} day^{-1}]
R	Effective rainfall after wetting and depression losses [mm day^{-1}]
s	Hydraulic gradient of the pipe [-]
t_c	Concentration time in the catchment [s]
V	Volume [m^3]
w_T	Tank width [m]
w_w	Weir width [m]
W	Wetting loss [mm]
W_{max}	Total wetting loss [mm]
Ψ	Runoff coefficient [-]

Acronyms

AS	Activated Sludge
BOD	Biological Oxygen Demand
CIS	Common Implementation Strategy
COD	Chemical Oxygen Demand
CSO	Combined Sewer Overflow
DAE	Differential and Algebraic Equation
DPSIR	Driver-Pressure-State-Impact-Response
DO	Dissolved Oxygen
DWF	Dry Weather Flow
FP5	5^{th} Framework Programme of the European Commission
IUWS	Integrated Urban Wastewater System
MSL	Model Specification Language
ODE	Ordinary Differential Equation
PE	Population Equivalent
RTC	Real Time Control
TN	Total Nitrogen
TNH	Total ammonium
TOC	Total Organic Carbon
TP	Total Phosphorus
TSS	Total Suspended Solids
WEST	Worldwide Engine for Simulation, Training and Automation
WFD	Water Framework Directive
WWF	Wet Weather Flow
WWTP	Wastewater Treatment Plant

Appendix A

Default parameters in KOSIM-WEST

This Appendix contains parameter values for rain, evaporation, impervious and pervious surfaces as used in the Bleesbruck case study.

Table A.1: Default catchment parameter values in KOSIM-WEST.

Climate values		
Parameter	**Default**	**Units**
$\overline{E}_y$	647	mm
$\overline{I}_y$	800	mm
Impervious surfaces (ATV 128)		
Ψ_0	0.25	
Ψ_e	1	
W_{max}	0.5	mm
D_{max}	1.8	mm
ACCU($\mathrm{COD_{part}}$)	4.4	kg/ha/d
Pervious surfaces (lawn / clay)		
Ψ_0	0	
Ψ_e	0.3	
W_{max}	2	mm
D_{max}	3	mm
k_-	43.2	day^{-1}
k_+	0.144	day^{-1}
f_0	0.3	mm min^{-1}
f_∞	0.03	mm min^{-1}
Wet weather concentrations		
$\mathrm{k_e(COD_{part})}$	0.18	mm^{-1}
$\mathrm{C_R(COD_{sol})}$	10	mg/l
$\mathrm{C_R(NH_4)}$	3	mg/l
$\mathrm{C_R(PO_4)}$	2	mg/l

Table A.2: Default DWF parameter values in KOSIM-WEST (partly taken from ATV-DVWK (2000)).

Dry weather flow parameters		
Parameter	**Default**	**Units**
$\overline{Q}_{PE}$	150	l/d/PE
C(COD$_{part}$)+C(COD$_{sol}$)	120	g/d/PE
C(TN)	11	g/d/PE
C(TP)	1.8	g/d/PE
C(NH_4)	9	g/d/PE
C(PO_4)	1.2	g/d/PE
Tourism	165-274	day
Tourism Water Factor	1	-
Tourism Pollution Factor	1	-
W-E Water Factor	1	-
W-E Pollution Factor	1	-
Infiltration	0.1	l/s/ha

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Summary

The presented work lies within the context of the EU Water Framework Directive (WFD) adopted in 2000. It sets a number of deadlines to be met by Member States to reach good quantitative and qualitative status of water resources and introduces the concept of integrated river basin management. More particularly, it shifts the focus from a purely source emission approach to a combined approach with 'control of pollution at source through the setting of emission limit values and of environmental quality standards' (Article 40, WFD). It is widely accepted that modelling will play a major role within the WFD implementation, e.g. to fill information gaps around a river basin or to design monitoring and management plans.

This dissertation focuses on the integrated urban wastewater system (IUWS), i.e. the system consisting of urban runoff, sewer system, treatment plant and receiving river, which is, next to diffuse pollution, an important source of pollution to receiving waters. Within the immission-based approach of the WFD, the water quality of the river evaluates the performance of the IUWS in terms of hydraulic and pollution impacts. As a consequence, when exposed to the same urban catchment, a small creek would require different IUWS management practices than a larger river. Some of the possibilities and new directions for IUWS management are exposed in Chapter 2 and make clear that the large number of degrees of freedom to implement management schemes do not make it straightforward to find the 'right' solution for a considered urban catchment. It is also concluded that model-based scenario analysis represents one appropriate tool to test for the 'best' solution(s) according to given criteria.

The challenge to model the IUWS lies above all within the structural complexity of the system itself. Besides the system's large spatial extent, the complexity is also the result of its non-linear dynamics that are the result of a complex interplay of a wide diversity of processes. Hence, setting the level of detail of the representation of such a system in order to attain the set goals is a global challenge of such modelling exercise, and, in particular, for the here presented work.

The developed approach, based on the Driver-Pressure-State-Impact-Response framework, is also explained. The different steps to be accomplished are the collection of data, the identification of deficits and pressures, the construction and calibration of the model and the definition of scenarios that are expected to improve the system's performance. Once simulations are run, results can be evaluated according to well-defined criteria.

A major challenge for IUWS modelling is the connection of subsystem models. Through the fact that the subsystems sewer, WWTP and river are usually dealt with by different stakeholders and that they are subject to different problems (e.g. hydraulics in the sewer, biochemical processes in the WWTP, ecological indicators in the river), modelling softwares for subsystems most often differ and are difficult to link, making data transfer a necessity and a problem. The here used modelling software, called WEST®(MOSTforWATER, N.V., Kortrijk, Belgium), contains models for the simulation of the WWTP and river and Chapter 3 presents the newly implemented models for urban runoff and sewer transport. The idea behind this extensive additional implementation is to have all necessary modules to build the integrated model for sewer, WWTP and river available in one software. This arrangement will facilitate data transfer between models. The general concept of the implemented models has been taken from the KOSIM software (ITWH, 2000), which is widely used in Germany and Luxembourg.

The Chapter explains that the models are of appropriate complexity for application within an integrated context: not too complex to keep the data required for calibration reasonable and to keep the simulation times to a minimum, however detailed enough to contain all the necessary processes affecting the variables of interest. In addition, models for accumulation and wash-off of particulate matter on the surface and backwater effects were included as they were considered necessary for the required quality of simulation results. The developed model to approximate backflow is presented in more detail, and calibration of the model using hydrodynamic simulation results is illustrated on the integrated case study in Chapter 5. KOSIM-WEST simulations are compared with the results obtained with the original KOSIM software in order to identify their respective fields of application. To test and gain experience with the new models in WEST, they have been applied to a small catchment in Luxembourg City where data, both for hydraulics and water quality, were monitored in a storm water tank. Major outcomes of the evaluation are that first the local availability of incoming rain data determines the quality of the results to a large extent and that good dry weather flow calibration using online quality data is important before looking at wet weather results. Towards the end, Chapter 3 briefly recalls the modelling approaches applied to the WWTP and river systems and gives principles for the connector models which are used to link variables among these submodels.

Chapter 4 contains the characteristics of the case study 'Bleesbruck' (Luxembourg), from population density to industry information in the catchment, to wastewater treatment plant layout and description and data on the 3 rivers of the considered urban catchment. Through two targeted measurement campaigns, water quality at the WWTP and the river were monitored in order to serve the subsequent model construction and calibration. The planning and set up of the campaigns are explained. Using the gained system information, deficits and pressures are identified as they are important to define alternate system configurations that can be used in the subsequent scenario analysis. It is concluded that one of the catchments' rivers brings high background pollution from upstream the Bleesbruck catchment, that the WWTP has poor nitrification capacity and that the sewer system is overflowing regularly.

Both existing and measurement campaign data were used to build and calibrate an integrated model of the case study in order to perform a scenario analysis using various system configurations. It is not a trivial task to build such an integrated model, first of all due to the complexity of the integrated urban wastewater system and therefore the related model itself, and secondly due to the difficulty for the user to choose the appropriate submodels for the integrated model out of a multitude of possible options. The choice depends on the level of data availability and the objectives of the study in question.

For each of the 3 submodels, after all available data were gathered and analysed for quality, the model was constructed and calibrated. In the case no or few data were present for calibration, parameter values were either fixed at default values taken from literature, or estimations were done where possible. The adopted methodology and approaches to build the integrated model in order to achieve our goals are presented in Chapter 5.

For the development of the urban drainage model, several steps were followed. First, hydraulic calibration of the hydrologic model was performed on the basis of hydrodynamic simulation results of the main collector obtained in InfoWorks CS (Wallingford Software, UK). Second, water quantity and quality were calibrated for 8 months with online measurements at the WWTP influent. However, overflows at individual catchments could not be adjusted, as, apart from visual inspections and experience of the operator, no data was available regarding the activity of the overflow structures.

Using an already existing SIMBA model as a basis, the WWTP model was implemented in WEST and has been calibrated and validated using 2 weekly measurement campaigns. Subsequently, the model was recalibrated over an 8 month period. This long-term calibration was a necessary step to allow using the model for the purpose of this long-term assessment of the system, i.e. to account for seasonal differences.

The main objective of the river model calibration is to get good water quality predictions as water quality will be the relevant criterion during scenario analysis. The used model is a simplified version of the IWA River Water Quality Model No. 1 (RWQM). pH has been omitted as monitoring showed it could be considered as constant and consumers are left out due to unavailability of data to state anything about their influence. The river model was calibrated using the data from the two measurement campaigns. The main components of importance in the study are nutrients and dissolved oxygen and this is where the calibration focused upon. Although the three submodels differ with respect to the different processes that take place, it was ensured that all of the submodels were calibrated over the longest time period so as to make them consistent in that sense. The simulation results are extensively presented and the chosen calibration parameter sets are discussed.

With the calibrated integrated model and the information on the case study deficits, 15 scenarios were developed and described in the first part of Chapter 6. The scenarios include among others source control measures like load peak flattening or reduction of water masses through impervious surface reduction, construction measures like sewer retention tanks or WWTP nitrification volume increase, system operation modification like improved phosphorus control or measures taken directly in the river like aeration and shading. For each of them, an indicative cost analysis is performed to help stakeholders to choose optimal scenario(s) later on. The evaluation criteria were defined for emissions as well as immission concentrations, using mean, minima, maxima, duration and frequencies above/below thresholds. Variables of concern that were considered are chemical oxygen demand, dissolved oxygen, ammonia and nitrates, and orthophosphates. The aim of the developed evaluation approach was to design a concept for easy interpretation of simulation results. Through the abundance of data, both in terms of the frequency over time and the variety of locations, the overview on essential, objective driven outcomes is quickly lost.

The here developed evaluation matrix contains all the information for scenarios and criteria and gives a clear overview. Together with an analysis of selected events, the evaluation approach will show which variable is mostly affected by which scenario.

In this specific case, the scenario analysis illustrates that the impact of the different scenarios on the already eutrophied and polluted river in the Bleesbruck case study is small, due to the already high background pollution present. It could be concluded that investments for implementation of the WFD in this river basin need to be done in a first instance upstream of the Bleesbruck catchment, i.e. requiring emission-immission based upgrades of the treatment facilities of cities upstream. Taking into account the present background pollution, implementation of improved control algorithms for nitrogen and phosphorus removal at the treatment plant presents good results at relatively low costs and can bring about positive changes with regard to reduction or even elimination of concentration peaks. Consequently, it can reduce the risk of ammonia fish intoxication and, as phosphorus is the limiting nutrient for algae growth, decrease algae mass in the vicinity of the WWTP.

In a second scenarios analysis, the same system configurations were tested, however, assuming that all receiving rivers are already WFD-compliant according to biochemical criteria. Results show that in such situation the receiving waters are much more vulnerable to urban pollution originating from the Bleesbruck catchment, as for example the capacity to cope with DO depleting discharges is gone with the state of supersaturation that existed with the presence of algae.

The conclusions highlight the usefulness of modelling both within WFD and IUWS management and presents prospects for further research in this area. The Driver-Pressure-State-Impact-Response (DPSIR) framework served as a basic structure to define the individual steps that constituted the system's analysis and it is repeated that the developed methodology from data collection to model construction to scenario analysis can be applied to other case studies. Within the WFD implementation context, modelling of the IUWS proved to be an essential ingredient supporting a more detailed analysis of an urban catchment where basin-wide models cannot give precise answers on how to operate or plan urban wastewater management that takes into account river water quality. The ability for zooming in on specific events gives the modeller the opportunity to understand what is happening on short time-scales and to verify whether simulated results are plausible.

Overall such model is expected to increase the number of reported real integrated case studies, so that it will be possible in the future to compare the investigation and implementation performed here with the outcomes of model predictions from other studies. It will further allow to demonstrate the usefulness of an integrated approach and to bring forward the discussion on when and where an integrated model is appropriate.

Samenvatting

Dit werk kan gesitueerd worden in de context van de Europese Kaderrichtlijn Water (KRW), ingevoerd in 2000. Het legt de deelstaten een aantal deadlines op om tot een 'goede' kwantitatieve en kwalitatieve toestand van de wateren te komen, en introduceert het concept van geïntegreerd rivierbekkenbeheer. Meer specifiek wordt de nadruk gelegd op een gecombineerde aanpak met 'brongerichte controle door het stellen van emissiegrenswaarden en milieukwaliteitsstandaarden', in plaats van een brongerichte aanpak alleen. Wereldwijd wordt aangenomen dat modellen een belangrijke rol zullen spelen binnen de KRW-implementatie, bv. door het leveren van ontbrekende informatie omtrent een rivierbekken of door het ontwerpen van monitoring- en beheerstrategieën.

Deze thesis richt zich op het geïntegreerd stedelijk afvalwatersysteem (SAWS), i.e. het systeem bestaande uit de afspoeling van het stedelijke gebied, het rioleringssysteem, de waterzuiveringsinstallatie en de ontvangende rivier, dat naast diffuse vervuiling een belangrijke bron vormt van verontreiniging van de ontvangende wateren. Binnen de context van de immissiegebaseerde aanpak zal de resulterende waterkwaliteit van de rivier een evaluatie vormen van de werking van het SAWS in termen van hydraulica en vervuilingimpact. Binnen eenzelfde stedelijke gebied zou een smalle beek bijgevolg een ander SAWS-beheer vereisen dan indien een grote rivier doorheen het gebied zou vloeien. Een aantal mogelijkheden en nieuwe tendensen voor SAWS-beheer worden beschreven in hoofdstuk 2. Dit overzicht maakt duidelijk dat een groot aantal vrijheidsgraden ter beschikking staat voor het implementeren van beheerschema's, waardoor het moeilijk wordt om de 'geschikte' oplossing te vinden voor het beschouwde stedelijke gebied. Modelgebaseerde scenarioanalyse kan dus een hulpmiddel zijn bij het vinden van de 'beste' oplossing(en) op basis van gegeven criteria.

De uitdaging bij het modelleren van SAWS ligt voornamelijk in de structurele complexiteit van het systeem. Enerzijds is er de grote ruimtelijke omvang, maar daarnaast is er ook de complexe wisselwerking tussen diverse processen die aanleiding geven tot niet-lineaire dynamica. Het vinden van het juiste niveau van detaillering van een model van dergelijk systeem om de gestelde doelstellingen van de oefening te bereiken, is een algemene uitdaging van elke modelleringoefening en ook van het voorgestelde werk.

De in dit werk ontwikkelde aanpak, gebaseerd op het DPSIR-kader ("Driver-Pressure-State-Impact-Response"), wordt ook voorgesteld. De verschillende stappen die moeten ondernomen worden zijn het verzamelen van data, het identificeren van tekortkomingen en moeilijkheden in het rivierbekken, het ontwikkelen en kalibreren van het model en het beschrijven van scenario's die verondersteld worden de werking van het systeem te verbeteren. Eenmaal het systeem in silico gesimuleerd kan worden, kunnen de resultaten gevalueerd worden volgens goed gedefinieerde criteria.

En van de grote uitdagingen in het modelleren van SAWS is het koppelen van de deelsysteemmodellen. Daar de deelsystemen riolering, afvalwaterzuiveringsinstallatie (AWZI) en rivier meestal behandeld worden door verschillende beheerders en zij met verschillende problemen worden geconfronteerd (bv. hydraulica in de riolering, biochemische processen in de AWZI, ecologische indicatoren in de rivier), is de modelleringsoftware voor deze deelsystemen vaak verschillend en moeilijk te koppelen, wat leidt tot problemen bij dataoverdracht. Het in dit doctoraatsonderzoek gebruikte softwarepakket is WEST® (MOSTforWATER, N.V., Kortrijk, Belgi). Het bevat reeds modellen voor het simuleren van de AWZI en de rivier. Hoofdstuk 3 beschrijft de implementatie van de nieuwe modellen voor afspoeling van stedelijk gebied en rioleringstransport. Deze implementatie had tot doel het geïntegreerde model - bestaande uit alle noodzakelijke modules voor riolering, AWZI en rivier - te kunnen opbouwen in n softwarepakket. Dit vereenvoudigt de dataoverdracht tussen de modellen. Het algemene concept van de nieuw gemplementeerde modellen is gebaseerd op de KOSIM software (ITWH, 2000), dat veel gebruikt wordt in Duitsland en Luxemburg.

Verder wordt in hoofdstuk 3 aangetoond dat de modellen voldoende complex zijn voor toepassing binnen een gentegreerde context: niet te complex zodat de vereiste data voor kalibratie en ook de simulatieduur tot een minimum kunnen beperkt blijven, maar toch gedetailleerd genoeg om alle noodzakelijke processen te bevatten die de belangrijke variabelen kunnen benvloeden. Daarenboven werden modellen voor accumulatie en afspoeling van partikels op de oppervlakte en modellen voor terugstroomeffecten in de riolen opgenomen, daar zij noodzakelijk geacht werden voor het bekomen van de vereiste kwaliteit van de resultaten. Het ontwikkelde model om de terugstroming in riolen na te bootsen wordt in meer detail besproken. De kalibratie van het model dat steunt op simulatieresultaten bekomen met een hydrodynamisch model, wordt toegepast op de gentegreerde gevallenstudie en is besproken in hoofdstuk 5.

KOSIM-WEST simulaties worden vergeleken met de resultaten bekomen met de oorspronkelijke KOSIM software om aldus de mogelijke toepassingsdomeinen ervan te identificeren. Als test en om ervaring op te doen over de nieuwe modellen in WEST®, werden ze toegepast op een klein bekken in de stad Luxemburg. Hiervoor werden gemeten data zowel voor hydraulica als voor waterkwaliteit in een stormwatertank gebruikt. De belangrijkste resultaten zijn vooreerst dat de lokale variabiliteit in regenintensiteitsgegevens de resultaten benvloedt, en dat de kwaliteit van de kalibratie van de droogweerafvoer op basis van on-line kwaliteitsdata voldoende moet zijn vooraleer de kalibratie op basis van natweerafvoer-resultaten begonnen wordt. Op het einde van hoofdstuk 3 worden ook de modellen voor AWZI en rivier voorgesteld, en worden de principes van de koppelingsmodellen gegeven die gebruikt werden om de variabelen van de verschillende deelmodellen met elkaar te koppelen.

Hoofdstuk 4 bevat informatie over de gevallenstudie 'Bleesbruck' (Luxemburg), gaande van bevolkingsaantal over informatie betreffende de industrie in het gebied tot de lay-out van de afvalwaterzuiveringsinstallatie en de 3 ontvangende rivieren van dit SAWS. Door gerichte meetcampagnes werd de waterkwaliteit van het AWZI-effluent en de rivier gemeten, en kon deze meetdata vervolgens gebruikt worden in het modelontwerp en de kalibratie. De planning en opzet van de campagnes worden beschreven. Gebruik makende van de verworven systeeminformatie werden tekortkomingen en moeilijkheden gedentificeerd. Zij zijn belangrijk om relevante systeemconfiguraties te definiren die kunnen gevalueerd worden in de daaropvolgende scenarioanalyse. In deze studie naar tekortkomingen en moeilijkheden werd duidelijk dat één van de ontvangende rivieren zorgt voor een sterke achtergrondvervuiling stroomopwaarts van het Bleesbruck gebied, dat de AWZI slechts een beperkte nitrificatiecapaciteit bezit, en dat overstorten in het rioleringssysteem regelmatig voorkomen.

Zowel bestaande als gemeten data werden gebruikt om een gentegreerd model van de gevallenstudie op te bouwen en te kalibreren, en er vervolgens een scenarioanalyse mee uit te voeren gebruik makende van verschillende systeemconfiguraties. Het is geen eenvoudige opdracht om een dergelijk geïntegreerd model op te bouwen; vooreerst omdat het geïntegreerd stedelijke afvalwatersysteem en dus ook het model complex is, en verder omdat het voor de gebruiker niet gemakkelijk is om de geschikte deelmodellen voor het gentegreerde model te kiezen uit een waaier aan mogelijkheden. De keuze zal afhangen van de databeschikbaarheid en de doelstellingen van de studie.

Nadat alle beschikbare data was verzameld en op kwaliteit onderzocht, kon ieder van de drie deelmodellen opgezet en gekalibreerd worden. In geval geen of weinig data ter beschikking was voor kalibratie, werden voor de parameterwaarden ofwel standaardwaarden uit de literatuur genomen ofwel waarden geschat op basis van de metingen. De methodologie en aanpak om onze doelstellingen te bereiken, is beschreven in hoofdstuk 5. Voor het stedelijke drainagemodel werden verschillende stappen ondernomen. Vooreerst werd de hydraulische kalibratie van het hydrologische model uitgevoerd met behulp van de hydrodynamische simulatieresultaten van de belangrijkste collector bekomen met InfoWorks CS (Wallingford Software, UK). Ten tweede werden waterkwantiteit -en kwaliteit gekalibreerd met on-line metingen verzameld aan de ingang van de AWZI over een periode van 8 maanden. Het model kon echter niet aangepast worden om ook overstorten aan de individuele bekkens te simuleren daar, naast visuele inspecties en ervaring van de operator, geen data ter beschikking was met betrekking tot de overstortactiviteit.

Met het bestaande SIMBA model als basis, werd het AWZI model in WEST® geïmplementeerd, opnieuw gekalibreerd en gevalideerd gebruik makende van twee wekelijkse data sets. Vervolgens werd het model gekalibreerd op basis van data verzameld gedurende 8 maanden. Deze lange termijn kalibratie was een noodzakelijke stap om het model te kunnen gebruiken voor lange termijn voorspellingen van het systeem, i.e. het in rekening brengen van seizoensgebonden variaties.

Het belangrijkste doel van de kalibratie van het riviermodel is goede voorspellingen te bekomen van de waterkwaliteit daar waterkwaliteit het relevante criterium is voor de scenarioanalyse. Het gebruikte model is een vereenvoudigde versie van het IWA rivierwaterkwaliteitsmodel (RWQM). Een gedetailleerde beschrijving van de pH werd weggelaten uit het model daar deze als constant kon beschouwd worden. Ook de verbruiker-organismen werden uit het model weggelaten daar geen data beschikbaar waren om iets over hun invloed op het systeemgedrag op te nemen in het model. Het riviermodel werd gekalibreerd met behulp van data afkomstig van de twee meetcampagnes. De belangrijkste componenten die werden beschouwd, zijn de nutrinten en opgeloste zuurstof. Hoewel de drie deelmodellen verschillen vertonen met betrekking tot de verschillende processen die erin plaatsgrijpen, werd er toch voor gezorgd dat alle deelmodellen gekalibreerd werden over de langst mogelijke tijdsperiode zodat ze hieromtrent consistent bleven. De simulatieresultaten worden in detail getoond en besproken, en de gekozen waarden voor de kalibratieparameters worden bediscussieerd.

Met behulp van het gekalibreerd gentegreerd model en de informatie over de tekortkomingen en problemen van de gevallenstudie, werden 15 scenario's ontwikkeld die beschreven worden in het eerste deel van hoofdstuk 6. Ze omvatten onder andere brongerichte maatregelen zoals piekafvlakking of vermindering in aangevoerde waterhoeveelheden door een afname in de ondoorlaatbare oppervlakten; ontwerpmaatregelen zoals rioolretentie of toename in nitrificatievolume van AWZI; aanpassingen in werking van de installaties zoals verbeterde fosforcontrole of maatregelen die onmiddellijk genomen kunnen worden in de rivier zoals artificile beluchting en beschaduwen door aanplanting van de oevers met bomen. Voor ieder van hen werd een kostenanalyse uitgevoerd die beheerders later moet helpen bij het kiezen van het optimale scenario('s). Evaluatiecriteria werden gedefinieerd voor emissie- n immissieconcentraties, gebruik makende van gemiddelden, minima, maxima, duur en frequenties boven/beneden grenswaarden. Chemische zuurstofvraag, opgeloste zuurstof, ammonium en nitraat, en orthofosfaat zijn de belangrijkste variabelen.

Het doel van de ontwikkeling van een evaluatiemethode was het vinden van een manier om de simulatieresultaten eenvoudig te interpreteren. Door de overvloed aan data door de hoge frequentie in tijd en de verschillende plaatsen waarop data worden geproduceerd, kan men het overzicht over de belangrijkste resultaten gemakkelijk verliezen. De hier ontwikkelde evaluatiematrix bevat alle informatie over scenario's en criteria, en geeft een duidelijk overzicht. Samen met een analyse van specifieke gebeurtenissen, laat dit toe te bepalen welke variabele het meest benvloed wordt door welk scenario.

Specifiek voor deze gevallenstudie toont dit aan dat de impact van aanpassingen in het systeem op de al geutrofieerde en vervuilde rivier klein is door een reeds te hoge achtergrondvervuiling in het gebied. Stroomopwaarts van het Bleesbruckgebied dient men te investeren in de implementatie van de KRW, i.e. het vereisen van een emissie-immissiegebaseerde verbetering van de behandelingsinstallaties in de stroomopwaarts gelegen steden. Rekening houdende met de aanwezige achtergrondvervuiling geeft de implementatie van verbeterde controlealgoritmen voor stikstof- en fosforverwijdering in de AWZI goede resultaten, aan een redelijk lage kostprijs. Verder kan het ook positieve veranderingen teweegbrengen zoals piekvermindering of zelfs -eliminatie. Het kan bijgevolg het risico van ammoniumintoxicatie bij vissen doen dalen, en de algenpopulatie verminderen in de nabijheid van de AWZI, daar fosfor het limiterende element is voor algengroei. In een tweede scenarioanalyse werden dezelfde systeemconfiguraties getest, maar nu werd verondersteld dat alle rivieren reeds voldoen aan de KRW met betrekking tot biochemische criteria. De resultaten tonen aan dat de ontvangende wateren onder deze verbeterde condities veel kwetsbaarder geworden zijn voor inkomende stedelijke vervuiling. Het kunnen omgaan met lozingen die leiden tot zuurstoftekort bijvoorbeeld, zal samengaan met een toestand van oververzadiging die bestaat in aanwezigheid van algen.

De conclusies belichten de bruikbaarheid van modellen zowel binnen de implementatie van de KRW als bij het SAWS-beheer, en perspectieven voor verder onderzoek binnen dit domein worden ook aangegeven. Het DPSIR-kader wordt hier nogmaals naar voor gebracht als basisstructuur waarin de individuele stappen die deel uitmaken van de systeemanalyse worden samengebracht. Er wordt herhaald dat de ontwikkelde methodologie gaande van dataverzameling over modelontwerp tot scenarioanalyse kan toegepast worden op andere gevallenstudies. Binnen de context van de KRW-implementatie toonde het modelleren van SAWS aan dat het geschikt was voor een meer gedetailleerde analyse van een stedelijk gebied. Bekkenmodellen kunnen echter geen exacte antwoorden geven betreffende het plannen van stedelijk afvalwatermanagement dat de rivierkwaliteit in rekening brengt. De mogelijkheid tot het dieper ingaan op bepaalde gebeurtenissen, geeft de modeleerder de kans om te begrijpen wat er gebeurt op korte termijn en te verifiren of de simulatieresultaten plausibel zijn.

Tot slot wordt verwacht dat het gebruik van dergelijke modellen bijdraagt tot meer gerapporteerde, echte geïntegreerde gevallenstudies zodat het hier uitgevoerde onderzoek en de implementatie van beheersmaatregelen in de toekomst zal kunnen vergeleken worden met de modelresultaten van andere studies. Het zal dan ook toelaten om de bruikbaarheid te tonen van een geïntegreerde aanpak, en om de discussie op gang te brengen over wanneer en waar het gebruik van een dergelijk geïntegreerd model geschikt is.

Curriculum Vitae

CURRICULUM VITAE

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EDUCATION

Sept. 2003 – today:	Ghent University, Belgium **PhD**: "Modelling of the Integrated Urban Wastewater System."
Sept. 2001 – Sept. 2002:	Université Libre de Bruxelles, Belgium Diplôme d'Etudes Approfondies (**D.E.A.**) in Physics "Spatial self-organisation in vegetation"
Sept. 1997 - June 2001:	University College London, UK **M.Sc.** in Physics, *First Class Honours* "Electron transfer probabilities in macromolecules"
Sept. 1989 - June 1997:	Athénée de Luxembourg, Luxembourg Diplôme de Fin d'Etudes Secondaires, *Mention "Bien"* Section : Mathématiques – Sciences Physiques - Latin

WORK EXPERIENCE

April 2007 – today:	Project Engineer at Paul Wurth S.A., Department of "Civil Construction & Project Management"
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Nov. 2002 – Sept. 2003:	High school teacher in physics : 'Chargée de cours', Athenée de Luxembourg.
Jan. – June 2000:	MHR Catering Service, London.

RESEARCH STAYS ABROAD

24 Feb. 2007 – 10 March 2007:	modelEAU, Université Laval, Québec, Canada
Oct. 2003 – Dec. 2003	BIOMATH, Ghent University, Belgium

CONFERENCE PROCEEDINGS

Solvi, A.-M., L. Benedetti, S. Gillé, P. M. Schosseler, A. Weidenhaupt and P. A. Vanrolleghem (2004). Modélisation d'un bassin versant urbain avec sa rivière en vue de la Directive Cadre Eau. In: *Proc. 1^{ères} Journées Doctorales en Hydrologie Urbaine*, 25-26 October 2004, Lyon, France.

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WORKSHOPS & CONFERENCES

"Hochschul-Simulations-Gruppe" (HSG) Workshops, Subgroup: "Integrated Modelling"
27-28 October 2003, Magdeburg (D)
14-16 May 2004, Luxembourg (L)
18-20 November 2004, Essen (D)
14-16 April 2005, Graz (A)
27-29 October 2005, Berlin (D)
27-29 April 2006, Darmstadt (D)
23-25 November 2006, Linz (A)

Premier Séminaire Scientifique Européen de L'OTHU, 12 November 2003, Lyon, France.

CityNet Conference, 15-18 March, Ghent, Belgium.

6th International Conference on Urban Drainage & Modelling, 15-17 September 2004, Dresden, Germany.

Journées Doctorales en Hydrologie Urbaine, 25-26 Oct. 2004, Lyon, France. *Oral presentation*

10th International Conference on Urban Drainage, 21-26 August 2005, Copenhagen, Denmark. *Oral presentation.*

CD4WC project meeting, 6-7 March 2006, Oostende, Belgium

WWWEST06 - User Conference, 6-7 May 2006, Kortrijk, Belgium. *Oral presentation.*

CD4WC dissemination workshop, 6-7 July 2006, Dresden, Germany. *Oral presentation.*

International Conference in Hydroinformatics 2006, 4-7 September 2006, Nice, France. *Oral presentation.*

World Water Congress and Exhibition 2006, 10-15 September 2006, Beijing, China. *Oral presentation.*

LANGUAGES

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