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modular conceptual modelling in urban drainage development and application of city drain

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I can't believe it! Reading and writing actually paid off!

Matt Groening

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Zuerst möchte ich mich bei Prof. Wolfgang Rauch (IUT, Universität Innsbruck) bedanken für die Betreuung über die letzten Jahre hinweg. Danke, dass Du neben den theoretischen Aspekten eines universitären Betriebes uns zu praxisorientiertem Denken und Arbeiten angehalten hast.

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Innsbruck, 15th of June 2006

KURZFASSUNG

Im Bereich der Siedlungsentwässerung verschob sich im Laufe der letzten Jahre der Ansatz vom bislang überwiegend verfolgten Prinzip der Emissionsbegrenzung hin zur verstärkten Betrachtung des Zustandes der Gewässer. Besonderes Augenmerk wurde dabei auf die Verbesserung der Qualität des Vorfluters sowie auf die gesamtheitliche Betrachtung von Flusseinzugsgebieten gelegt. Diese stellt ein Kernelement der Wasserrahmenrichtlinie (WRRL) dar.
Ziel ist es, verschiedenste Richtlinien im Bereich der WRRL zu bündeln, diese jedoch nicht zu ersetzen. Die langfristige Zielsetzung ist die Erreichung eines guten Zustandes der Gewässer. Eine Vorgehensweise ist der kombinierte Ansatz bei dem Emissions- und Immissionskriterien gleichermaßen zu betrachten sind. Die integrierte Betrachtung des gesamten Systems – dem Kanalsystem, der Kläranlage und dem Vorfluter – erlangte immer mehr an Bedeutung. Vorrangig im wissenschaftlichen Bereich, aber ebenso in der täglichen Ingenieurpraxis. Die Bedeutung der Modellierung von urbanen Abwassersystemen nahm dabei ebenso zu.

Diese Arbeit beschäftigt sich mit der modularen konzeptionellen Modellierung in der Stadtentwässerung. Dynamische Modelle werden dabei verwendet, um Hydraulik und Schadstofftransport sowie chemische und biologische Prozesse zur beschreiben und zu erfassen. Die Intention ist, das Verhalten der Teilsysteme selbst und deren Beeinflussung untereinander vorherzusagen.

Kern der Arbeit war die Entwicklung und Anwendung von CITY DRAIN, einer open-source Software für die integrierte Modellierung von urbanen Entwässerungssystemen. Die Software wurde in Matlab/Simulink erstellt und ermöglicht eine blockweise Modellierung der verschiedenen Teilsysteme (Einzugsgebiet, Kanalsystem, Rückhaltebauwerke, Vorfluter, etc.). Ziel war es ein Werkzeug zu erstellen, welches einfach handhabbar ist und trotzdem eine flexible Anpassung an verschiedene Situationen und Szenarien erlaubt. Es wurde vermieden unnötige Komplexität in die implementierten Modellen einzubringen, da der Benutzer diese nach seinen speziellen Bedürfnissen erweitern oder adaptieren kann und auch soll. Der Fokus wurde dabei auf konzeptionelle Modelle gelegt, da diese weniger aufwendig in Bezug auf Rechenleistung sind als physikalische Modelle und somit auch für Langzeitsimulationen geeignet sind.

Der Leser wird in die verschiedenen Modellkonzepte und die der numerische Implementierung in CITY DRAIN eingeführt. Die Anwendung von CITY DRAIN wurde anhand der alpinen Fallstudie Vils/Reutte gezeigt. Getroffene modelltechnische Annahmen sowie auch Implikationen bei der Kalibrierung des Modells werden diskutiert. Anwendungsgrenzen der Modelle werden anhand des verwendeten Belebtschlamm-Modells aufgezeigt. Das Kläranlagenmodell benötigt dabei spezielle Anpassung der Modellparameter aufgrund der – für das Fallbeispiel spezifischen – extrem niedrigen Abwassertemperatur im Zulauf der Kläranlage.

Es werden erweiterte Anwendungen von CITY DRAIN, die über die Standardanwendung mit festen Parametersätzen hinausgehen, vorgestellt. Die Anwendungen beinhalten eine Echtzeitsteuerung (Real Time Control – RTC) bzw. eine Modelbasierte Steuerung (Model-based Predictive Control – MBPC). Ziel beider Steuerungsarten ist die Optimierung des integrierten Abwassersystems. Eine detaillierte Darstellung der Arbeiten - welche mit CITY DRAIN realisiert wurden – erfolgt in den entsprechenden Veröffentlichungen welche der Arbeit als Anhänge C/D und E angefügt sind. Es werden Maßnahmen an der Quelle (den Haushalten) und dem Vorfluter (dem Fluss), das heißt an beiden Enden des Abwassersystems, vorgestellt. Beiden gemeinsam ist die Verwendung von Steuerungen (RTC und MBPC), welche entwickelt und anschließend offline getestet wurden.

Die Arbeit befasst sich abschließend mit der Analyse und Bewertung von Simulationsergebnissen, entsprechend dem Bedarf an einer strukturierten Darstellung von Ergebnissen. Eine adäquate Nachbearbeitung und Interpretation von produzierten Daten ist in jedem Fall unabdingbar. Ziel ist, Simulationsergebnisse so zu komprimieren, dass dies eine Beurteilung der Situation erlaubt. Damit wird auch das Ziel verfolgt, die Verwendung konzeptioneller Modelle in der täglichen Ingenieurpraxis weiter zu propagieren.

ABSTRACT

In the last years design procedures of urban drainage systems have shifted from end of pipe design criteria to ambient water quality. Emphasis is put on the improvement of the receiving water quality and the overall management of river basins, which is a core element of the Water Framework Directive (WFD) as well. Enactment of the WFD did not replace, but bundle various guidelines in the field that were issued over the years. Long-term goals defined include reaching a “good status” of water courses. The use of emission and immission criteria was introduced as combined approach. The integrated assessment of the total system - that is the sewer system, treatment plant and the receiving water – became of more concern within science but as well in the daily engineering work. Modelling of urban drainage system as a tool gained importance accordingly.

This thesis deals with the modular conceptual modelling in urban drainage. Dynamic modelling in urban drainage is used to describe and asses processes such as hydraulic and pollutant transport or chemical and biological processes. Intention is to predict the behaviour of subsystems in themselves as well as interactions.

Core of the thesis was the development and application of CITY DRAIN, an open source software for integrated urban drainage modelling. It was developed in the Matlab/Simulink environment, enabling a block wise modelling of the different parts of the urban drainage system (catchment, sewer system, storage devises, receiving water, etc). Aim was to create a tool that provides simplicity in handling and a certain flexibility allowing to cope with different situations and scenarios. It is avoided to introduce unnecessary complexity in the implemented models, where the user is allowed to extend or modify models according to the specific needs. Focus was on conceptual model as they are less demanding in computational time than physical models and allow therefore long term modelling.

The reader is introduced to the modelling concepts and their numerical implemented in the software. The application of CITY DRAIN is shown using the example of the alpine case study Vils/Reutte. Assumptions made within the build up of the model are discussed as well as implications when calibrating an urban drainage model. Limits associated to models are discussed with regard to the activated sludge model used. The WWTP model used required specific modifications due to - in this case study – extreme low temperature of the influent wastewater.

Advanced applications of CITY DRAIN beyond the “straight forward application” with fixed parameters are presented. These include Real Time Control (RTC) and Model-Based Predictive Control (MBPC) technology, both aiming for an optimization of the integrated wastewater. Two applications that were realized within CITY DRAIN are presented in more detail and are attached as Appendix C/D and E. Therein measures were applied to the receiving water and the households, being the opposite ends in the wastewater system. Both applications in common is the use of control features (RTC and MBPC) which were developed and tested offline.

The work concludes with a discussion on the evaluation of simulation results, since there is need for a structured presentation of results. An appropriate post processing and interpretation of produced data is indispensable. Goal is to condense data gained as modelling results, to a feasible level allowing judgement on the situation. This goes hand in hand with the aim to further promote the use of conceptual modelling within the daily engineering work.

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APPENDIX

A	**Achleitner, S.**, DeToffol, S., Engelhard, C. and Rauch, W. (2005). The European Water Framework Directive: Water Quality Classification and Implications to Engineering Planning. *Environmental Management*, Volume 39, No. 4, 517-529.
B	**Achleitner S.**, Möderl M. and Rauch W. (2006). CITY DRAIN © - an open source approach for simulation of integrated urban drainage systems. *Environmental Modelling & Software*, (accepted)
C	**Achleitner, S.**, DeToffol, S., Engelhard, C. and Rauch, W. (2005). Model-based hydropower gate operation for mitigation of CSO impacts by means of river base flow increase. *Water Science and Technology*, Vol 52 No 5 pp 87–94.
D	**Achleitner, S.** and Rauch, W. (2006). Increase of river base flow by hydropower gate operation for mitigation of CSO impacts - Potential and Limitations. *Water Resources Management.* (submitted).
E	**Achleitner, S.**, Möderl, M. and Rauch, W. (2006). Waste design by urine separation – the development of control options for the case study Vils/Reutte. *Urban Water Journal*. (submitted).
F	**Achleitner S.** and Rauch W. (2005). *CITY DRAIN © - an open source integrated simulation of urban drainage systems - User Manual*. Institute of Environmental Engineering, University of Innsbruck, Austria.

1 INTRODUCTION AND OVERVIEW

1.1 INTEGRATED MODELLING IN URBAN DRAINAGE

The assessment of different processes in the field of urban drainage gained more and more importance over the years. Historically the different parts of the urban wastewater and drainage system were covered by different responsibility. Especially the rivers and lakes as receiving water were treated separated from sewer and wastewater treatment systems. In urban drainage, the shift from a narrowed focus on subsystems to an overall assessment of the system took place over the last decade. A milestone towards the integrated assessment in urban drainage was set with the Interurba I conference in 1992 (Lijklema *et al.*, 1993). There the integrated assessment of interactions between sewers, treatment plants and receiving waters (see schematic in Figure 1) has been promoted.

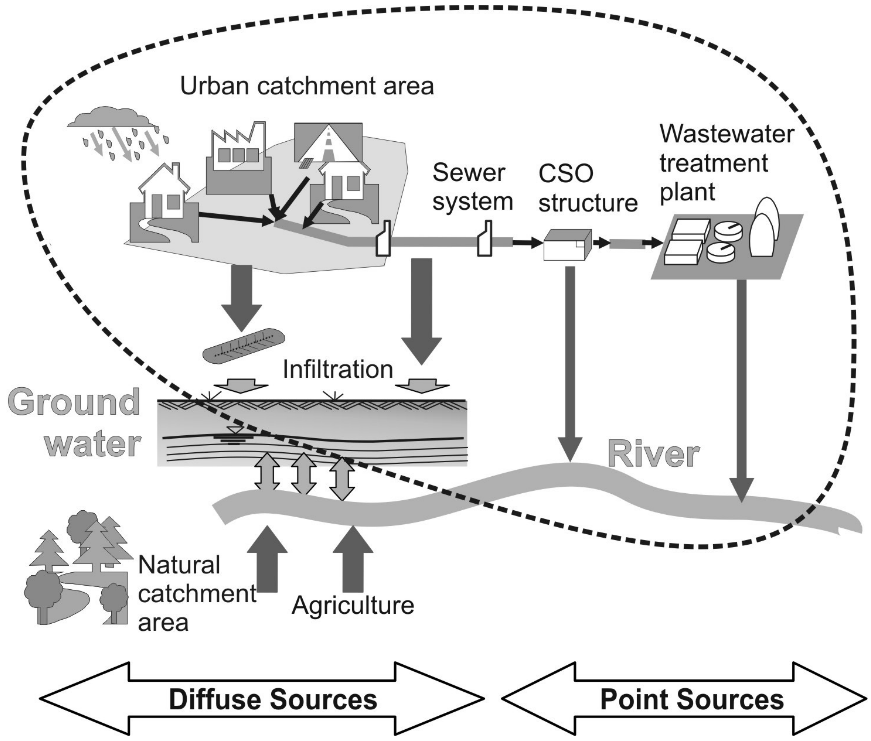


Figure 1: Schematic on system boundaries for an integrated assessment in urban drainage including an ambient water quality based evaluation.

The claim of an integrated assessment including ambient water quality aspects – as it was initiated by scientists – grew and was subsequently introduced to legislation. Thus, dealing with the complete picture of a situation became of more concern – mostly in science but within the daily engineering work as well.

Modelling as a tool to asses and describe processes was used to predict the behavior of each subsystem in itself and in interaction with others. Processes therein can be manifold, ranging from physical processes such as hydraulics and pollutant transport to associated chemical and biological processes.

At the Interurba II conference, almost a decade later in 2001, Harremoës (2002) reported on the status in the field. A number of tools for integrated analysis were developed, but there was unfortunately less implementation than expected. According to Harremoës (2002) this is due to a lack in knowledge, but is attributable as well to conservatism in the business. Focus until then was mainly on sewer and treatment plants, where wastewater sources and recipients were considered to a lesser extent.

Just before Interurba II, the European Water Framework Directive (EU-WFD) (EU/2000/60/EC-en, 2000) came in force, aiming as well at an integrated assessment in the field of water and wastewater management. The directive is to be seen as a document defining a legal frame for the next decades aiming at the preservation of water courses. As the EU-WFD does not include definitive "numbers to work with" existing legal documents and methods are not replaced by it but cumulated to a uniform approach.

This thesis was made against the background of a growing demand on assessing water systems as a whole. Specifically it deals with integrated modelling and assessment of different aspects in urban drainage.

1.2 OVERVIEW ON THE THESIS

Integrated assessment and overall management of river basins evolved over the last years. As legal aspects and current standards in the field are essential, an introductive overview is provided in Chapter 2. The main legal document on EU level is therein the water framework directive (WFD). The European Water Framework Directive and its implications has been covered in (Achleitner *et al.*, 2005b), attached as Appendix A.

<u>Appendix A</u>

> **Achleitner, S.**, DeToffol, S., Engelhard, C. and Rauch, W. (2005).
> The European Water Framework Directive: Water Quality Classification and Implications to Engineering Planning. *Environmental Management*, Volume 39, No. 4, 517-529.

Against the background of the legal frame in urban drainage, tools for the integrated assessment and modelling in urban drainage gain more and more of importance. Typically, it is not necessary to model the whole variety of effects on the receiving water but to focus on the few dominating ones. Only pollutants and processes that have a direct and significant influence on the selected impacts need to be described quantitatively, whereas all other processes can be neglected. Hence, pragmatism is required to avoid unnecessary complexity of integrated models. Within Chapter 3, an overview on modelling concepts is given regarding hydraulics and pollutant transport. Focus is on conceptual models as these types of models are used within this work.

The claim for pragmatism instead of unnecessary complexity is true for software applications as well. Especially in daily engineering work, simplicity in handling and a certain flexibility to adjust for different scenarios is required. Different subsystems should be freely arrangable and connectible to each other for describing an integrated urban drainage system and the fluxes of water and matter.

Core of the thesis is the software CITY DRAIN © following these principals. It is an open source software for integrated modelling of urban drainage systems and was developed by Achleitner and Rauch (2005b) at the Institute of Environmental Engineering at the University of Innsbruck.

CITY DRAIN © was developed in the Matlab/Simulink © environment, enabling a block wise modelling of the different parts of the urban drainage system (catchment, sewer system, storage devises, receiving water, etc). Each block represents a system element (subsystem) with different underlying modelling approaches for hydraulics and mass transport. The principles and models used are shown in Chapter 4. The open structure of the software allows to add own blocks and/or modify blocks (and underlying models) according to the specific needs. A publication on CITY DRAIN from Achleitner *et al.*(2006b) as well as the user manual of CITY DRAIN 1.0 (Achleitner and Rauch, 2005b) is attached as Appendix B and E respectively.

Appendix B

Achleitner S., Möderl M. and Rauch W. (2006).
CITY DRAIN © - an open source approach for simulation of integrated urban drainage systems. *Environmental Modelling & Software*, (accepted)

Appendix F

Achleitner S. and Rauch W. (2005).
CITY DRAIN © - an open source integrated simulation of urban drainage systems - User Manual. Institute of Environmental Engineering, University of Innsbruck, Austria.

The development of the software was made in the framework of the EU-Project CD4WC (Benedetti *et al.*, 2004) and was co-financed by the Tiroler Wissenschaftsfond established by the state of Tyrol, Austria.

Chapter 5 deals with the application of CITY DRAIN using the example of the alpine case study Vils/Reutte. Assumptions made within the build up of the model are discussed as well as implications when calibrating an urban drainage model. Limits associated to models are discussed with regard to the activated sludge model used. The WWTP model used required specific modifications due to - in this case study – extreme low temperature of the influent wastewater.

In chapter 6 a number of applications of CITY DRAIN beyond the “straight forward application” with fixed parameters is shown. The applications range from CITY DRAIN being used for testing multiple scenarios running in batch mode (Engelhard *et al.*, 2006) to applications featuring real time control (RTC) and model-based predictive control (MBPC) technology. Two of such applications realized within CITY DRAIN are presented in more detail and are attached as Appendix C/D and E. Therein measures were applied to the receiving water and the households, being the opposite ends in the wastewater system. Both applications in common is the use of control features (RTC and MBPC) which were developed and tested offline.

Appendix C

Achleitner, S., DeToffol, S., Engelhard, C. and Rauch, W. (2005).
Model-based hydropower gate operation for mitigation of CSO impacts by means of river base flow increase. *Water Science and Technology*, Vol 52 No 5 pp 87–94.

Appendix D

Achleitner, S. and Rauch, W. (2006).
Increase of river base flow by hydropower gate operation for mitigation of CSO impacts - Potential and Limitations. *Water Resources Management.* (submitted).

Appendix E

Achleitner, S., Möderl, M. and Rauch, W. (2006).
Waste design by urine separation – the development of control options for the case study Vils/Reutte. *Urban Water Journal*. (submitted).

The first measure attached as Appendix C/D deals with the increase of river base flow at low flow stretches during rain events (Achleitner *et al.*, 2005a; Achleitner and Rauch, 2006). Low flow stretches downstream of intakes from hydropower station are – as other river parts – subjected to combined sewer overflows. Impacts can, due the low flow capacity of the river, be more severe than it is in other parts of the river. The key idea is to create an increase of river flow by temporal operation of the upstream gates and thereby create sufficient dilution. An operational algorithm has been developed, implemented and tested offline. The measure itself can be referred as an in-stream applied measure using model-based predictive control (MBPC) for operation.

A second application (Appendix E) is situated at "the other end" of the system, dealing with the source side. Waste design – specifically ammonia in the wastewater – was focus of this work. Waste design by urine separation is therein coupled with the storage and controlled release of urine to the drainage system. Performance of the totally eleven control strategies tested was measured by (a) the degree of averaging the ammonia load pollutograph at the WWTP inflow and (b) the increase in overflow quality regarding ammonia loads. Control options applied range from simple to sophisticated ones including real time control (RTC) features.

The work concludes with a discussion on the evaluation of modelling results, since there is need for a structured presentation of results. An appropriate post processing and interpretation of produced data is indispensable. Goal is to condense data gained as modelling results, to a feasible level allowing judgement on the situation.

2 LEGAL FRAMEWORK, STANDARDS AND MOTIVATION FOR INTEGRATED MODELLING

In the following existing legal requirements and boundary conditions shall be given. The main focus is clearly on the EU-water framework directive (EU/2000/60/EC-en, 2000) and its implementation on national level, having Austria as example. The intention is – besides giving an insight in the newly gained legal frame – to show how modelling of urban drainage systems as such fit therein. Especially integrated modelling does not only end in itself, but is an appropriate tool to fulfil state of the art requirements.

Aside legal frameworks on EU and national level, numerous standards are given requiring dynamic modelling as state of the art tool. An excerpt of the most important ones in the field of urban drainage is presented.

2.1 EU WATER FRAMEWORK DIRECTIVE (WFD)

2.1.1 HISTORICAL DEVELOPMENT

Historically the evolution of the WFD was such, that already numerous guidelines in the field of the water supply and waste water treatment were exempted on European level. Still, the implementation of these guidelines and regulations in the members states was partial on going.

Exemplary, without claim to completeness, are mentioned:

- Council Directive 75/440/EEC of 16 June 1975 concerning the quality required of surface water intended for the abstraction of drinking water in the Member States
- Council Directive 76/160/EEC of 8 December 1975 concerning the quality of bathing water.
- Council Directive 76/464/EEC of 4 May 1976 on pollution caused by certain dangerous substances discharged into the aquatic environment of the Community
- Council Directive 80/68/EEC of 17 December 1979 on the protection of groundwater against pollution caused by certain dangerous substances
- Council Directive 91/271/EEC of 21 May 1991 on Urban Waste Water Treatment
- Council Directive 91/676/EEC of 12 December 1991 concerning the protection of waters against pollution caused by nitrates from agricultural sources
- Council Directive 98/83/EC of 3 November 1998 on the quality of water intended for human consumption

The abundance of guidelines shows, that cause-related regulations in the form of directives and regulations on EU-level were considered to be sufficient in the field of the environment.

Due to the numerous single regulations and to the problems of the member states during the implementation it was evident that a comprehensive, homogeneous and new EU-water policy was necessary. Since 1995 the development and discussion of a water directive became increasingly more concrete. Increasing problems with regard to quality and quantity of the water lead to efforts to develop a unified procedure within the EU. Soon it became clear that these "new" problems could just be solved by overall and integrated approaches of the water management system including all water related impacts (Saurer *et al.*, 2000).

In 1996 an extensive public discussion started on the draft for a water framework directive which should cover the whole water management system from now on. This included the objectives for management of ground- and surface water bodies on the basis of ecological needs for a sustainable preservation of a "good status" of all water bodies in Europe. The ultimate objective is the sustainable preservation of all water resources in Europe, measured by the particular ecological needs in a river basin.
Thus, the "Directive 2000/60/EC for establishing a framework for Community action in the field of water policy" (EU/2000/60/EC-en, 2000) represents a new attempt for conservation, improvement and sustainable use of surface- and groundwater bodies in Europe.

2.1.2 AIMS AND FUTURE CHALLENGES

By 2000 the WFD has come into force aiming for an overall and integrated approach for water management systems including all water related impacts. Implementation of the WFD is mainly on a national level, where the member states define their own standards and methods following the main objectives of the WFD (Holzwarth, 2002).

All water bodies have to reach a "good status" by 2015. The good status - represented by reference sites – is by definition a widely undisturbed state where only slight deviations from natural conditions are allowed and EU-quality standards are not to be exceeded. Defined reference sites serve as a goal function for the state of water bodies to be reached. Due to characterization of the good status varying among different ecoregions and sites a further differentiation is needed. Besides the declaration of river basin districts the definition of river type specific regions is required. Subsequently, reference sites and quality objectives, representing the "good status" are defined. Details on the methodology of the definition of reference site and corresponding water bodies can be found in (Achleitner *et al.*, 2005b).

The WFD makes it necessary to change the focus from chemical properties to an overall view on chemical and biological quality. Both are required equally to reach the good status. Figure 2 illustrates the scheme for water quality classification regarding different aspects.

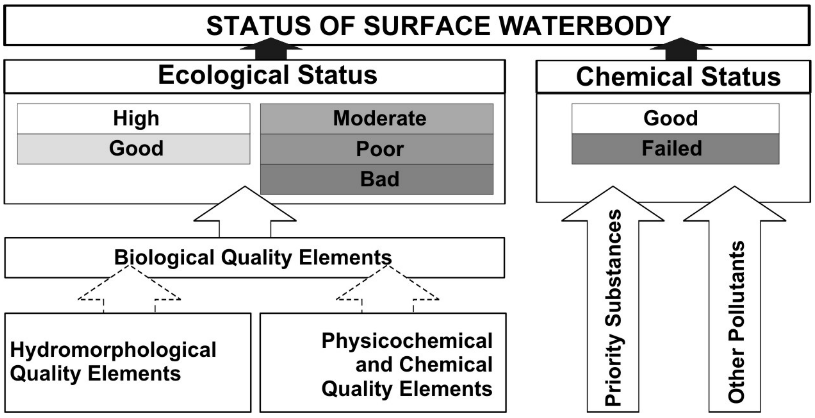


Figure 2: Water quality classification according to WFD (EU/2000/60/EC-en, 2000)

Rating of the ecological status is done using a five class system ranging from “high status” to “bad status”. Biological quality elements used for characterization shall be supported by hydromorphological, chemical and physico-chemical elements. However the WFD does not give an exact definition of the term “good” ecological status. A number of studies and EU-projects such as AQUEM (AQEM, 2002), FAME (Economou *et al.*, 2002) or STAR (Sandin *et al.*, 2000; Sandin *et al.*, 2001) are dealing with the water quality classification.

The second stream, the chemical status, relies on various directives for limiting in–stream concentrations of substances. Thus, rating is simplified to a two class system where the status is either “good” or “Failing to achieve good”. In view of planning chemical alterations in the stream there are in many cases predictable with current models (Bowie *et al.*, 1985; Brown and Barnwell, 1987; Cox, 2003; Dahl and Wilson, 2001; Reichert *et al.*, 2001b).

In contrast to chemical effects, biological alterations are often long-term effects and are thus hardly attributable to a single impact. Accurate prediction of biological alterations is currently limited. Still for future planning purposes in the light of the WFD, it may be essential to have tools available covering cause-effect relations between chemical and biological conditions. The research projects PEAQANN (Lek *et al.*, 2003) or RIVPACS (Clarke *et al.*, 2003) deal with the prediction of biological indicators under natural and/or disturbed conditions. An overview on these works can be found in (Schulz, 2003).

2.1.3 THE COMBINED APPROACH – EMISSION AND IMMISSION

One major innovation is the shift from emission standards towards a combined approach (Barth and Fawell, 2001).
Herein the two approaches - receiving water quality (immission) and pollution limitation (emission standards) – are incorporated. Both are to be considered in an overall approach. Immission-based standards within the WFD do not only incorporate chemical characteristics in the receiving water, but focus is also on the overall ecological condition. Thus, besides chemical and physico-chemical characteristics, biological and hydromorphological conditions are to be considered too. Hydromorphological impacts, their characterisation and the evaluation of mitigation measures in an integrated context can be found in (Engelhard *et al.*, 2005)

or (Rauch *et al.*, 2002c). Lek *et al.*(2006) continued the work by applying more elaborated using more elaborated modelling approaches.

As examples for legal frames on the immission side directives such as the Drinking Water Directive or the Bathing Water Directive will apply. Regarding the emission side the Directive on Integrated Pollution Prevention and Control (IPPC) and Urban Waste Water Treatment Directive (UWWT) apply, among others.

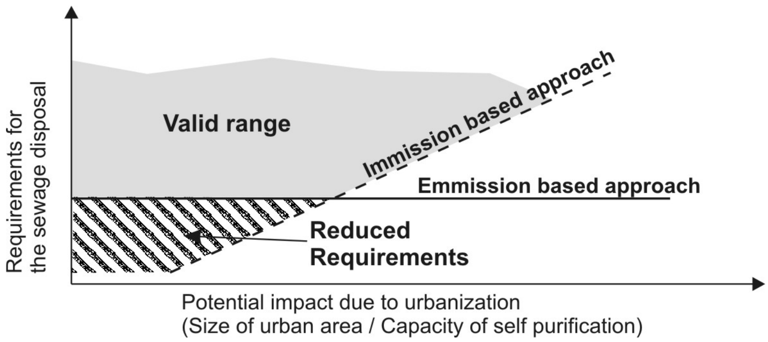


Figure 3: Schematic on requirements due to Emission and Immission-based standards (redrawn from (Krebs, 2003))

Figure 3 schematically illustrates the requirements of a wastewater system as a function of increasing impact to a receiving water. The common denominator of both approaches is a valid range were both emission and water quality standards are served (Blöch, 1999; Holzwarth, 2002; Olson, 1998). The rule of taking the more stringent standard may be questionable when efforts taken due to emission standards do not promise appropriate effects in the receiving water (Achleitner *et al.*, 2005b).

Concluding it is to be said that resulting biological effects in the receiving water are difficult to cover with currently available models, but research in the fields of chemical-biological interactions are in progress. Still, the WFD demanding a wider view does not only mean to include additional criteria, when dealing with urban drainage system. Following the combined approach it is rather that focus is to be put not only on the end of pipes but equally well on the immission side. There, an integrated approach allowing to get a – more – complete picture of situations seems to be an appropriate tool to tackle WFD requirements.

2.1.4 NATIONAL IMPLEMENTATION IN AUSTRIA

According to the EU-WFD, the national laws on federal level were to be harmonized with the EU-WFD. This is for providing a legal basis for the subsequent implementation step stipulated in the WFD.

The Austrian Water Act 1959 (Wasserrechtsgesetz 1959 - WRG, 2003) is the main legal document in field of Water. Its parental act was primarily a “water management act” focused on the distribution and usage of water. In the course of its revision in 1959 the Austrian Water act was extended for sanitary engineering aspects including water pollution control. Over the years more and more ecological aspects were included in the Austrian water act. In Austria an amendment of the Austrian Water Act was made in 2003 (WRG, 2003) to comply with WFD and adapt its contents accordingly.

The subsequent step in the implementation - characterisation of receiving waters and impacts - was taken thereafter. The corresponding report (Marent et al., 2005) was finalized in 2005. For river surface waters, 50 significantly diverse types were reported to the EU. In the frame of a EU-wide intercalibration process the defined reference conditions and classification methods used are to be amended if necessary.

According to the assessment made, 78% of the rivers (8900 km out of 11488 km) are fulfilling the requirements regarding general chemical-physical quality including the saprobiological water quality. Therefore they are categorized as good status. Less positive is the status for hydromorphological parameters. There ~56% failed to reach good status. Out of this percentage, ~21% can be attributed to insufficient quantity at low flow.

2.2 STANDARDS IN THE FIELD OF URBAN DRAINAGE

As outlined, the EU-WFD is to be seen as an overall approach where national laws and regulates are still to be considered. These regulations cover different parts and aspects of the urban drainage system. In contrast to the EU-WFD which has a wider scope, these regulation contain actual limitations on both the emission and the immission side. In the following a number of national regulations are briefly outlined. Focus was on regulations that do not exclusively deal with strict limit values but contain aspect such as reoccurrence intervals of substance concentrations, hence requiring a more sophisticated analysis.

2.2.1 ÖWAV REGELBLATT 11

The ÖWAV Regelblatt 11 (ÖWAV RB11, 2006) represents the amendment of the (ÖNORM EN 752, 2005) respectively the corresponding EU standard. Focus is on the calculation and hydraulic design of combined and separate sewers. It is currently under revision and on its way for publication.

Where the standard aims, besides other goals, for the assessment of flooding events, utilization of appropriate models is recommended. For sake of simplicity, pipe design can be based on the maximum discharge using the Rational Method. The method is limited for small systems without large storage volume and without back pressure associated. When having a more complex system – which is most often the case - hydrological or hydrodynamic models are required to cover the temporal dynamics of the hydraulic behaviour.

2.2.2 ÖWAV REGELBLATT 19

The ÖWAV Regelblatt 19 (ÖWAV-RB19, 2006) deals with the design of overflow structures in combined sewer systems. Different to earlier design guidelines the intention is not to provide rigid design rules for construction. The objective is to deliver a certain portion of storm water – and associated pollutants - to the wastewater treatment plant (WWTP), characterised by the degree of efficiency η. Minimum efficiencies η defined vary with the size of the WWTP connected and the rain intensities.

The efficiency is calculated as

$$\eta = \frac{(VQ_m - VQ_t)\cdot c_m - VQ_e \cdot c_e}{(VQ_m - VQ_t)\cdot c_m}\cdot 100 = \frac{VQ_r \cdot c_m - VQ_e \cdot c_e}{VQ_r \cdot c_m}\cdot 100$$

with

η	…Degree of efficiency [%]
VQ_m	…Annual hydraulic load at the combined sewer [m³/a]
VQ_t	…Annual hydraulic load of dry weather flow [m³/a]
VQ_r	…Annual hydraulic load of storm water flow [m³/a]
VQ_e	…Annual hydraulic load of combined sewer overflow [m³/a]
c_m	…Concentration of combined sewer flow (to CSO structure)[mg/l]
c_e	…Concentration of combined sewer overflow [mg/l]

Therein loads are considered as constant and dry weather is assumed to be fully mixed with storm water flow. For the calculation of the efficiency η the total catchment is taken into account, where the critical flow from an overflow structure is no more a stringent design parameter. Diversion of increased flow to the downstream sub-catchment is permissible as long as the overall efficiency is maintained. A simplified alternative is the calculation of the hydraulic efficiency:

$$\eta_r = \frac{VQ_r - VQ_e}{VQ_r}\cdot 100$$

It may as well be used for pollutants *NH_4-N, N_{TOT}, P_{TOT}, COD and BOD*. For their soluble fraction, the efficiencies are equal to the hydraulic one ($c_m = c_e$). Compared to the particulate fraction it is on the safe side although the true value is unknown.

Evaluation of the efficiency is to be done by means of hydrological long-term simulation based on rain series of several years. Following the simplified approach of the hydraulic efficiency, a hydraulic model without pollutant routing would be sufficient.

How to reach the required degree of efficiency is not defined. Besides the increase of storage volume as a classical option, other methods such as the increase of WWTP inflow, the implementation of sewer control strategies or additional infiltration are thinkable.

Since no local evaluation of emissions is imposed, a number of ambient water quality requirements are defined compensatory to simplifications on the emission side. Defined limits in the receiving water are for the hydraulic loading, acute ammonia(NH_4-N) toxicity, oxygen content, particulate matter, etc. Thus, ÖWAV RB-19 meets the core requirement of a combined approach as defined in the WFD.

2.2.3 UPM (URBAN POLLUTION MANAGEMENT) MANUAL

The Urban Pollution Management Manual (FWR, 1998) developed in Great Britain is a valuable guideline in view of integrated wastewater management. The manual provides background information in a comprehensive way in order to give insight in the used approaches.

When taking acute toxicity as an example, a single limit value or maximum concentration hardly enables a proper description. Whether a substance is harmful to a certain type of organism is not only determined by the magnitude of the impact, but also by the duration and frequency of the impact. Thus, the approach used was to provide maximum permissible limits valid for a certain impact duration and reoccurrence interval (return period).

The findings used in the UPM are based on ecotoxicological studies and laboratory tests. Differentiating for different types of waters (Salmonid and Cyprinid), such “concentration/duration thresholds” are provided for dissolved oxygen and un-ionised ammonia. In Table 1 the thresholds for cyprinid fishery are given.

Table 1: Concentration/duration threshold for DO and NH3-N in waters suitable for cyprinid fishery according to the UPM manual (FWR, 1998)

Return period [months]	Dissolved Oxygen concentrations [mg DO/l][a]		
	1h	6h	24h
1	4.0	5.0	5.5
3	3.5	4.5	5.0
12	3.0	4.0	4.5
	Un-ionised ammonia concentrations [mg NH3-N/l][b]		
	1h	6h	24h
1	0.150	0.075	0.030
3	0.225	0.125	0.050
12	0.250	0.150	0.065

a) applicible when NH3-N < 0.02 mg/l
b) applicible when DO > 5 mg/l, pH > 7 and T > 5°C
a), b) in exceedance of application ranges, correction factors apply

A clear focus is set towards the utilization of modelling tools for the analysis of urban wastewater discharges. This is as well reflected by the way ambient water quality criteria are defined, requiring analysis methods that are based on temporal dynamics.

2.2.4 WWTP EMISSIONS

Emissions from WWTP are regulated in Austria within the *“First Regulation for wastewater emission from municipal combined sewer system”* (1.AEV-K, 1996). Therein requirements for the discharge quality on a daily basis as well as for annual removal efficiencies are defined.

Annual removal capacities are defined as a percentage value compared to the inflow to the WWTP.

BOD_5	95% removal
COD	85% removal
TOC	85% removal
N_{TOT}	70% removal (for days where T>12°C)

With regard to temperature conditions, BOD_5, COD and TOC removal capacity is applied throughout the year, regardless of low temperature. N_{TOT} removal capacity is applied only for days where the temperature in the biological stage exceeds 12°C.

Maximum concentration limits for WWTP effluent quality are defined variable depending on the WWTP class (see Table 2).

Table 2: WWTP classification according to (1.AEV-K, 1996)

WWTP class			
I	50 PE_{60}	-	500 PE_{60}
II	500 PE_{60}	-	5 000 PE_{60}
III	5 000 PE_{60}	-	50 000 PE_{60}
IV		>	50 000 PE_{60}

The design capacity PE_{60} is therein the pollutant load of raw wastewater with a load of 60 g BOD_5 per PE and day.

Table 3: Maximum effluent concentrations according to (1.AEV-K, 1996)

		WWTP class			
		I	II	III	IV
BOD_5	mg/l	2	20	20	15
COD	mg/l	90	75	75	75
TOC	mg/l	30	25	25	25
NH_4N	mg/l	10 [a]	5 [a]	5 [b]	5 [b]
P_{TOT}	mg/l	-	2	1	1

[a]…applies at Temperatures > 12°C

[b]…applies at Temperatures > 8°C

Temperature limits exclude days of low temperature from the evaluation since the nitrification process is hardly maintainable then. For NH_4N, the limits apply to days where the temperature of the wastewater in the biological stage exceed 12°C (classes I and II) and 8 °C (classes III and IV).

Still, these limits are not sharp values but a certain number of limit violations is granted. The allowed number of days of violation increases with the annual measurements. For continu-

ous (daily) measurements, violations at 25 days out of 365 are permissible. When evaluating modelling results, a continuous measurement would apply. Thus, the maximum of 25 permissible violation would apply.
The complement in Germany is the *„Verordnung über Anforderungen an das Einleiten von Abwasser in Gewässer“* (AbwV, 1997) using a similar scheme. Still, the emission limits set differ somewhat. For instance, for treatment plants larger than 5 000 PE_{60}, the limit concentration for unionised ammonia (NH_4-N) is 10 mg/l – which is double the value as defined in Austria. Compensatory, the limit temperature where the limitation applies, is set to 12 °C compared to 8 °C for that size of WWTP in Austria. Similar limit values as in German legislation are found in the urban waste water treatment directive (EU/1991/271/EEC, 1991)

2.3 CONCLUSIONS

The development and implementation of the Water Framework Directive (WFD) has been presented. The corresponding publication covering details on the different implementation steps and their interactions (Achleitner *et al.*, 2005b) is attached as Appendix A.

Appendix A

> **Achleitner, S.**, DeToffol, S., Engelhard, C. and Rauch, W. (2005).
> The European Water Framework Directive: Water Quality Classification and Implications to Engineering Planning. *Environmental Management*, Volume 39, No. 4, 517-529.

In there, the definition of reference sites and water bodies as well as the subsequent water quality classification is discussed. It was shown that the steps strongly depend on each other and that where the size of waterbodies will have a large impact on the water quality.

The WFD demands a wider scope, especially with regard to taking ecological conditions into account. Methods for water quality classification (AQEM, FAME and STAR) concerning biological indicators were reviewed as they are valuable for defining boundaries for the different states of the ecology. For planning purpose, tools for prediction of biological conditions will become more important. Still, tools linking chemical and biological conditions of streams are limited. One research project (PAEQANN) was found linking chemical and biological condition using Artificial Neural Network (ANN) technology.

Further, the “combined approach” defined in the WFD was discussed. A combination of emission and immission standards is to be applied where the common understanding is that the more stringent one applies. Based on an example at the alpine river Drau (Austria) (see Appendix A) the stringent application of the combined approach is questioned. According to the legal implementation in Austria (WRG, 2003), a shift towards a full immission based approach is legally possible. Therefore, it could be shown that the WFD does not necessarily require increased efforts.

As the WFD is not replacing standards but bundles them to a uniform approach, standards in the field of urban drainage are presented, covering different parts of the urban drainage system. Both immission and emission side are addressed. The focus was on regulations that did not only deal with fixed limits but also aspects such as reoccurrence intervals.

The main conclusion drawn from this is that for fulfilment of legal requirements in a planning stage, modelling in one or the other form is required. To cope with the assessment of impacts and effects, especially long term modelling plays – in the regulations presented – a leading role.

3 MODELLING CONCEPTS

Modelling is a vital tool to assess the relevant mechanisms in the urban drainage systems to describe water and pollutants on their way from the source of generation down to the receiving water. A variety of modelling approaches are available to describe water motion as well as the transport and conversion of matter. In this chapter it is intended to review the fundamental equations typically used to describe these mechanisms.

In the following, concepts for describing

- rainfall runoff relations
- hydraulic transport/routing,
- pollutant transport/routing and
- pollutant processes.

are presented.

Core element is the hydraulic transport, that describes the transport of water as well as tracer (solute and conservative) substances. Associated processes (conversion of matter, settling etc.) are usually incorporated in the transport equations for substances. Thus, hydraulic transport is treated separately and the results are utilized as input for pollutant transport equations (including the conversion of matter).

The reviewed methods include complex approaches (e.g. hydrodynamic equations) as well as more simple conceptual approaches. Clear focus in the review is put on conceptual models as this type of models is used in the software CITY DRAIN. The actual models implemented are show in Chapter 4 in the course of presenting the most important blocks of the software.

3.1 FROM RAINFALL TO RAIN-RUNOFF

In urban drainage systems flows resulting from rainfall are of major importance. During intensive rain events, flows due to rainfall exceed other flow quantities by far. Thus, the generation of flow by rainfall and the associated processes are a vital element in urban drainage modelling. Still, not all rainfall that reaches the ground contributes to the surface flow generated. Some portion is lost due to a variety of processes.

The rainfall runoff generation thereby comprises methods that link the rainfall h_N with the effective rainfall h_{Ne} contributing to surface flow.

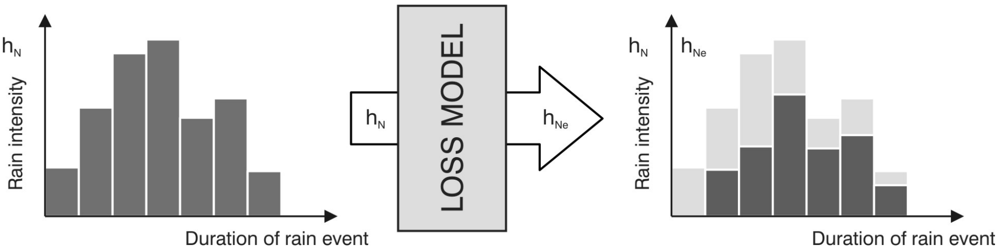


Figure 4: Schematic on the application of rainfall loss model

Rainfall intensity is thereby reduced due to different processes causing a retainment of rainfall. The magnitude of reduction may vary during the event depending on the rainfall intensity and the process applying. The main processes that cause a retainment of rainfall are

- wetting
- depression loss
- evaporation
- infiltration

Wetting of surfaces occurs at the beginning of a rain event resulting in a temporal storage of precipitation. Surface flow occurs when the wetting capacity [mm] is fully utilized. Depression losses occur due to the storage of rainfall in catchment depressions, occurring after the wetting. The stored quantity of both is – at a later point in time – released to the atmosphere via either evaporation or infiltration. Evaporation is compared to rainfall intensities, less relevant during rain events. Still, evaporation during dry periods is most relevant – specially for sealed surfaces - for “removing” stored water. Infiltration is the second process that reduces stored water. The process is less relevant for fortified areas. Different for permeable areas where runoff generation may be reduced and delayed.

In the following an overview on different rainfall runoff models is shown that includes part of the processes. The concepts can as well be found - slightly modified - in (Rauch *et al.*, 2002b).

3.1.1 THRESHOLD METHOD

The threshold method is the most simple method used. Losses are described by an initial loss (threshold value). Both, wetting losses and depression losses, can be associated to this type of loss.

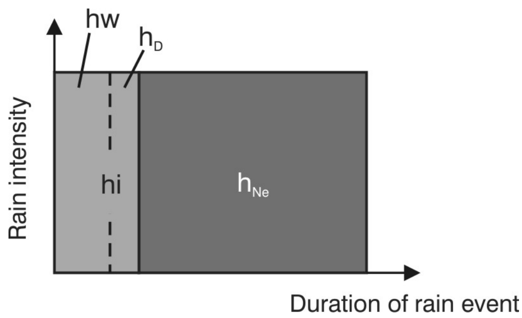


Figure 5: Loss model – Threshold method

Thus, as soon as the cumulated rainfall exceeds the initial loss *hi*, rainfall contributes to a runoff generation. Thus, the effective rainfall h_{Ne} contributing is

$$h_{Ne} = h_N - h_i$$

with

$$h_i = h_W + h_D$$

h_N	...Rainfall height [mm]
h_{Ne}	...Effective rainfall height [mm]
h_i	...Initial loss [mm]
h_W	...Wetting losses [mm]
h_D	...Depression losses [mm]

3.1.2 PERCENTAGE METHOD (PERMANENT LOSSES)

Within the percentage method, not only a constant initial loss *hi*, but a permanent loss *hp* is considered as well. After subtracting an initial loss, a constant percentage of rainfall is withdrawn.

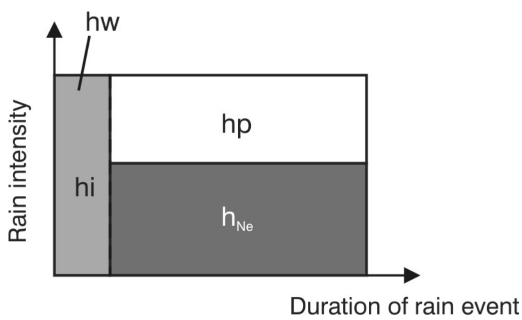


Figure 6: Lossmodel – percentage methode

$$h_{Ne} = h_N - h_i - \sum h_{P,\Delta t} = h_N - h_i - h_P$$

The permanent loss h_P is interpretable as e.g. drift due to wind or evapotranspiration. Regularly, the permanent loss is considered as a percentage of the overflow exceeding *hi*.

$$h_{P,\Delta t} = \varphi_P \cdot \left(h_{N,\Delta t} - h_i\right)$$

h_N	...Rainfall height [mm]
$h_{N,\Delta t}$	...Rainfall height per time step [mm/Δt]
h_{Ne}	...Effective rainfall height [mm]
h_i	...Initial loss [mm]
h_P	...Permanent loss [mm]
$h_{P,\Delta t}$	...Permanent loss per time step [mm/Δt]
φ_P	...Permanent loss factor (percentage) [-]

As an alternative a constant permanent loss can be defined having the unit of a fixed loss rate $h_{P,\Delta t}$. Per time step, a defined rain height is withdrawn.

$$h_{P,\Delta t} = const.$$

For the second option, the constant permanent loss rate $h_{P,\Delta t}$ can be applied during rain and dry periods or during the dry period only. Depending on the numerics behind, the loss rate may be used to empty any virtual reservoirs during dry periods.

3.1.3 LIMIT VALUE METHOD

The limit value method represents an extension of the so far presented methods. Besides considering the wetting effect as initial loss *(hi)* and permanent losses *hp*, the depression losses are included as a separate loss h_D. Still, infiltration is not considered separately. Thus, a method is to be used preferentially for impervious, urban areas.

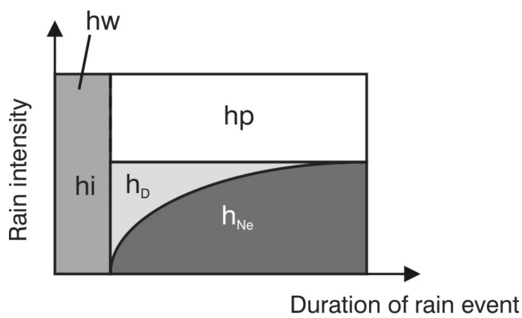


Figure 7: Loss model – limit value method

$$h_{Ne} = h_N - \left(h_i + h_P + h_D\right)$$

The depression loss is introduced as an exponential function.

3.1.4 INFILTRATION

Infiltration processes may be included as well in rainfall runoff modelling, as a relevant part of the area is permeable. A variety of infiltration models are available, describing the change of the infiltration rate over time. The infiltration rate may be taken to be linear related to the soil's permeability as it is the case in the (ATV-DVWK-A138, 2002).

$$r_I = {k_f}/{2}$$

with

r_I	…Infiltration rate [m/s]
k_f	…hydraulic permeability of the soil material [m/s]

A more elaborate approach is the infiltration model by Horton (1940) using an exponential approach (Beven, 2004).

$$r_{I,t} = r_{I,c} + \left(r_{I,0} - r_{I,c}\right) \cdot e^{-K_I \cdot t}$$

Therein

$r_{I,t}$	…Infiltration rate at time t [m/s]
$r_{I,c}$	…Equilibrium infiltration rate [m/s]
$r_{I,o}$	…Initial infiltration rate [m/s]
K_t	…Reduction constant covering the reduced infiltration capacity over time [-]
t	…time since the beginning of infiltration [s]

An overview on a number of other infiltration models can be found in (Clausnitzer *et al.*, 1998).

3.1.5 RUNOFF COEFFICIENT

The runoff coefficient is the most direct acting coefficient within rainfall runoff relations. The coefficient represents the portion of total area that contributes to the effective runoff generated. The effective runoff area is written A_{EFF} as

$$A_{EFF} = \varphi \cdot A_{TOT}$$

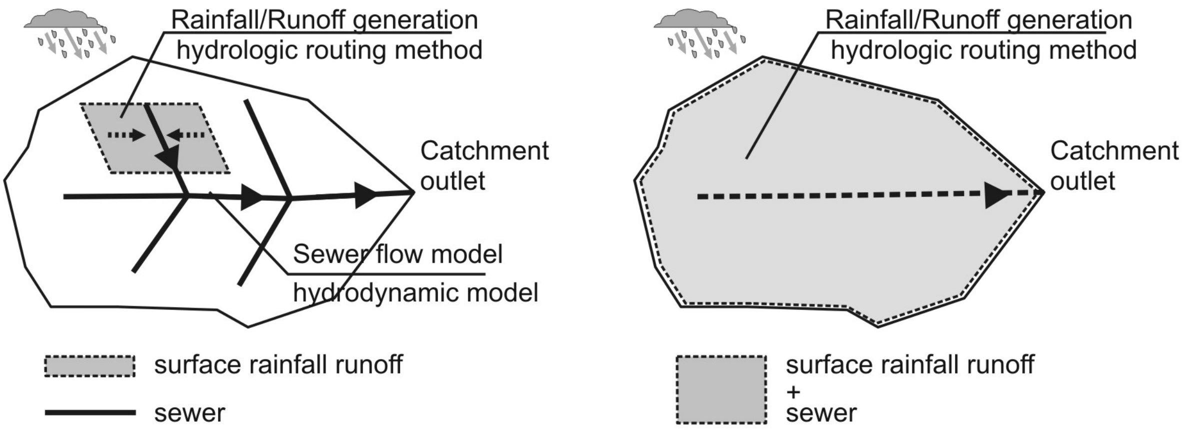
After rain has been subjected to the different types of losses, the effective rainfall h_{Ne} is converted to flow.

$$Q_{Ne} = \frac{h_{Ne} \cdot A_{EFF}}{\Delta t}$$

3.2 FLOW ROUTING MODELS

For the description of flow conditions, two general approaches are used. Thereby flow can either be described by (a) physical-mechanistic model or by (b) a conceptual model. Physical-mechanistic models are based on continuity (mass balance) equations as well as on the preservation of Energy or Momentum. Empirical relations are introduced for calculation of e.g. friction or point losses.
For conceptual (hydrological) model continuity applies as well. Physical relations are described by conceptual relations mostly using simple descriptions of cause effect relations.

Figure 8 illustrates how hydrological and hydrodynamic models are used in urban drainage modelling.



(a) (b)

Figure 8: Schematic on application of (a) hydrological in combination with hydrodynamic models and (b) hydrological models only.

Common to both setups is the transformation of distributed aerial rainfall to effective rainfall contributing to the runoff. Several relations and processes that reduce the quantity of rainfall were already mentioned.

Runoff generation and routing in the catchment can be done either with a combination of hydrological and hydrodynamic model (see Figure 8a) or with a hydrological model alone (see Figure 8b).

For case (a) the hydrological model is used to transform the effective aerial precipitation to flow entering the sewer system. This process is termed flow concentration, where flows are introduced to the connected sewer section via e.g. the manholes. For calculation of the flow regime in the sewer itself, hydrodynamic models are used. Such concepts of linking hydrological and hydrodynamic models are used e.g. by the software packages HYSTEM EXTRAN from ITWH (2002) or Mouse by DHI (2003). A benefit of using such an approach is that the flow regime in the sewer is assessable including backwater effects, overflows etc. A drawback is a larger computation time compared to the full conceptual approach. Still, the correctness of results depends on the inputs from the connected conceptual surface flow models.

Case (b) simplifies the situation, using only a hydrological model that describes the rainfall runoff relation down to the catchment outlet. In case the flow regime at the sewer level is of minor interest, this approach can provide satisfying output information. As a complete part of an urban catchment is approximated by a single model, a number of informations (e.g. detailed sewer data) is not required. Thus, a benefit of the model is a significant reduction of the amount of data required as input. In order to obtain feasible results, sewer storage volume – as it exists in case (a) – is to be mimicked by appropriate conceptual models (De Toffol *et al.*, 2006). This may either be done by applying a conceptual model that already includes a storage term or by introducing a virtual storage volume after the catchment outlet.

In the following, an overview on common hydraulic and hydrological models is given. Stress is clearly put on the hydrological models as this type of model is used in the software CITY DRAIN. Still, for completeness, the St. Venant Equation is given to illustrate differences.

3.2.1 PHYSICAL FLOW MODELS

The St. Venant Equation represents an approximation of the Navier-Stokes Equation. For 1-dimensional flow, the St. Venant equations comprise the continuity and energy equation (Haestad and Durrans, 2003).

$$\frac{1}{b}\cdot\frac{\partial Q}{\partial x}+\frac{\partial h}{\partial t}=0$$

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\cdot\left(\frac{Q^2}{A}\right)+g\cdot A\cdot\frac{\partial h}{\partial x}+g\cdot A\cdot\left(I_E-I_S\right)=0$$

Unknowns in this equation are the flow *Q(x,t)* and the corresponding cross section *A(x,t).* Depending on the degree of simplification, the full dynamic equation can be approximated as:

$$g\cdot\left(I_E-I_S\right)=0 \qquad \frac{\partial Q}{\partial x}+b\cdot\frac{\partial h}{\partial t}=0 \qquad \text{…kinematic wave}$$

$$g\cdot\frac{\partial h}{\partial x}+g\cdot\left(I_E-I_S\right)=0 \qquad \frac{\partial Q}{\partial x}+b\cdot\frac{\partial h}{\partial t}=0 \qquad \text{…diffusive wave}$$

$$\frac{\partial v}{\partial t}+v\cdot\frac{\partial v}{\partial x}+g\cdot\frac{\partial h}{\partial x}+g\cdot\left(I_E-I_S\right)=0 \qquad \frac{\partial Q}{\partial x}+b\cdot\frac{\partial h}{\partial t}=0 \qquad \text{…dynamic wave}$$

For the kinematic wave approximation, friction and energy slope are equal (I_E=I_S). Therefore only translation of a travelling wave is modelled. With the diffusive wave approximation, translation and damping effects are modelled. Since it is an equation of second order, two boundary conditions (up- and downstream) are required. Backwater effects and wave attenuation can be calculated.

3.2.2 CONCEPTUAL (HYDROLOGICAL) MODELS

As shown earlier, the utilization of conceptual flow models is multiple. This ranges from flow generation of a single sub-area to modelling on a catchment level. Other applications include river flow routing or transport sewers. In the following, an overview on the most common conceptual models for hydraulic routing is given.

3.2.2.1 Time Area Method (Isochrones Method)

The Time Area Method is commonly used for catchments considering the (time) stepwise translation of rainfall towards the catchment's outlet. For each point of a catchment water requires a specific time to reach the outlet. Thus the catchment is divided by isochrones, defining lines of points having equal flow time until reaching the outlet. Having isochrones of time steps Δt, enables one to implement the method in a discrete time computation (Rauch *et al.*, 2002b).

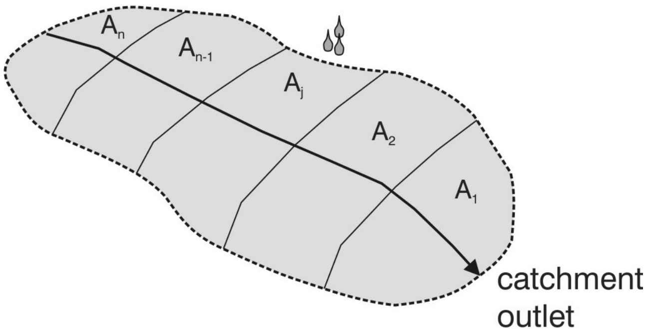


Figure 9: Schematic on sub-area distribution within the time area method

For a given catchment with n subareas, it takes the water $n.\Delta t$ to travel through the catchment. Within each time step, the volume is shifted one area downwards, and summed up with the rain fallen within the next time step. Thus, for the flow from the catchment Q_i at time i, rainfall from the last n time steps is contributing, considering the respective subarea.

$$Q_i = A_1 \cdot r_i + A_2 \cdot r_{i-1} + A_3 \cdot r_{i-2} + ... + A_j \cdot r_{i-(j-1)} + ...A_{n-1} \cdot r_{i-(n-2)} + A_n \cdot r_{i-(n-1)}$$

$$Q_i = \sum_{j=1}^{n} A_j \cdot r_{i-(j-1)}$$

Preferable for numerical computation, is the use of matrix operations. The above equation can be written as a scalar product of $\vec{r}$ and $\vec{A}$, being a vector of rainfalls r_i and sub areas A_j. The vector of areas is constant throughout the computation, storing $A_1 \ldots A_n$ in a column vector. The vector of rainfall changes with time steps i, storing rainfalls from $r_i \, .. r_{i-(n-1)}$.

$$Q_i = \begin{bmatrix} r_i & r_{i-1} & \cdots & r_{i-(j-1)} \end{bmatrix} \times \begin{bmatrix} A_1 \\ A_2 \\ \vdots \\ A_n \end{bmatrix} = \vec{r_i} \times \overrightarrow{A_j}$$

A simplified version of the time area method is the one in which the subareas are assumed equal size. A subarea would then be

$$Q_i = \begin{bmatrix} r_i & r_{i-1} & \cdots & r_{i-(j-1)} \end{bmatrix} \times \begin{bmatrix} A_{TOT}/n \\ A_{TOT}/n \\ \vdots \\ A_{TOT}/n \end{bmatrix} = \begin{bmatrix} r_i & r_{i-1} & \cdots & r_{i-(j-1)} \end{bmatrix} \times \begin{bmatrix} 1/n \\ 1/n \\ \vdots \\ 1/n \end{bmatrix} \cdot A_{TOT}$$

Thus the rainfall $r_{i,OUT}$ contributing to the outflow Q_i would be

$$r_{,OUTi} = \begin{bmatrix} r_i & r_{i-1} & \cdots & r_{i-(j-1)} \end{bmatrix} \times \begin{bmatrix} 1/n \\ 1/n \\ \vdots \\ 1/n \end{bmatrix} = \frac{Qi}{A_{TOT}}$$

being the outflow Q_i based on the catchments total Area A_{TOT}. Summation of r leads to

$$r_{i,OUT} = \sum_{j=1}^{n} \frac{1}{n} \cdot r_{i-(j-1)}$$

An alternative application of the methodology is to use it for flow routing in general. Thereby the rainfall intensity introduced to the subcatchments at each time step is interpreted as flow quantity. Time steps are again assumed to be constant. The resulting flow at the catchment outlet would then be

$$Q_i = Q_{U,i-(n-1)} + \sum_{j=1}^{n} Q_{i-(j-1)}$$

where flows $Q_{i-(j-1)}$ are flows introduced along the pathways. They can be used to mimic flows produced within the catchment such as dry weather flow quantities. Flows introduced up-stream – e.g. from an upstream catchment - have to overcome the flow path through all sub-areas, thus they are considered additive as $Q_{U,i-(n-1)}$.

3.2.2.2 Linear hydraulic retention

The simplest type of hydraulic retention is the linear retention basin. The type of storage is such that the retention effects are linearly related to the stored volume. A physical analogon is a vessel whose outflow is in linear relation to the volume stored (e.g. bottom outlet). Additionally continuity applies to the system.

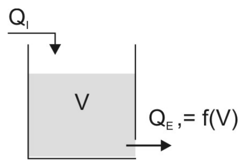


Figure 10: Single linear storage

Outflow variation is given by

$$Q_E(t) = \frac{1}{Ks} \cdot V(t)$$

$Q_E(t)$	…Outflow from the storage at time t [m^3/s]
Ks	…Storage Constant [s]
$V(t)$	…Storage Volume at time t [m^3]

Including the continuity equation ($dV(t)/dt = Q_I(t) - Q_E(t)$) the temporal derivation denotes:

$$\frac{dQ_E(t)}{dt} = \frac{1}{Ks} \cdot \frac{dV(t)}{dt} = \frac{1}{Ks} \cdot \left(Q_I(t) - Q_E(t)\right)$$

A limitation of the model is that it is not possible to consider translations between inflow and outflow hydrograph. Peaks at the inflow occur simultaneously in the outflow hydrograph.

In the same manner an outflow – volume relation of higher order can be described. In that case the outflow $Q_E(t)$ is not to be related proportional to the volume $V(t)$, but related to a more general form $V^X(t)$.

3.2.2.3 Cascading linear hydraulic retention

A more elaborated type of describing linear storage with only a single parameter can be done by taking a cascade of linear storage units (see (Rauch *et al.*, 2002b)).

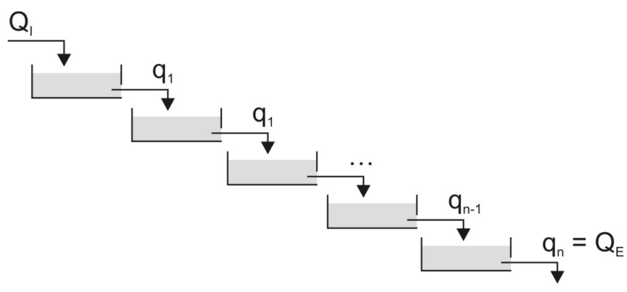


Figure 11: Cascading linear storage

with n being the number of cascade units. The flow from a storage unit i can be written as

$$\frac{dqi}{dt} = \frac{1}{K_{S,i}} \cdot \left(q_{i-1} - q_i\right)$$

analogue to the single linear storage. The total storage constant $K_{S,TOT}$ is the sum of the single constants of each unit:

$$\sum K_{S,i} = K_{S,TOT}$$

In case of applying the same constant $K_{S,I}$ to each of the n units, $K_{S,I}$ is given as

$$K_{S,i} = \frac{1}{n} \cdot K_{S,TOT}$$

The analytical solution of a cascade of linear storages is given as

$$q_n(t) = \frac{1}{K_S\left(n-1\right)!} \cdot \left(\frac{t}{K_S}\right)^{n-1} e^{-t/Ks}$$

for a unit hydrograph event. Hydrographs are obtainable as a superposition of unit hydrograph events.

3.2.2.4 Muskingum – Method of Flood Routing

The Muskingum Method has been developed for flood routing through channels. It has first been applied to flood control work on the Muskingum river, therefore it has been called *Muskingum Method* (Roberson *et al.*, 1995). For a wave passing a reach of a channel, the storage is described as a function of Inflow *(Qi)* and Outflow *(Qe)* as follows:

$$V = K \cdot Q_E + K \cdot X \cdot (Q_I - Q_E)$$

K …Constant [T]
X …Dimensionless weighting factor that relates to the amount of wedge storage

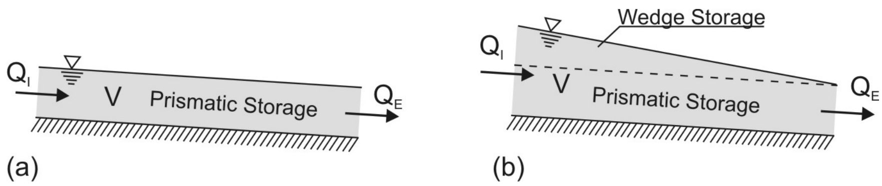


Figure 12: Flow in channels (a) Steady-uniform flow and (b) Flood-wave flow

The first term of the right hand side represents the prismatic storage where the second represents the wedge storage. K is equivalent to the time required for a unit discharge wave travelling through the reach ($K \approx \Delta t$). Thus, the term for prism storage can be written as

$$K \cdot Q_E = \Delta t \cdot Q_E = \Delta t \cdot V \cdot A_0 = L \cdot A_0$$

Within the continuity equation

$$\Delta V / \Delta t = Q_I - Q_E$$

The flows Q_I and Q_E as well as storage V vary over time. They are approximated by their values at times (i and $i+1$):

$$Q_I = \frac{Q_{I,i} + Q_{I,i+1}}{2}, \quad Q_E = \frac{Q_{E,i} + Q_{E,i+1}}{2} \text{ and } \Delta V = V_{i+1} - V_i$$

Thus, continuity can be rewritten as

$$Q_{I,i} + Q_{I,i+1} + \frac{2V_i}{\Delta t} - Q_{E,i} = Q_{E,i+1} + \frac{2V_{i+1}}{\Delta t}$$

The storage equation is formulated for time steps i and $i+1$ as well

$$V_i = K \cdot \left[X \cdot Q_{I,i} + (1-X) \cdot Q_{E,i})\right]$$

$$V_{i+1} = K \cdot \left[X \cdot Q_{I,i+1} + (1-X) \cdot Q_{E,i+1})\right]$$

Substituting the storage Volumes V_i and V_{i+1} in the continuity equation leads to:

$$Q_{E,i+1} = C_0 \cdot Q_{I,i+1} + C_1 \cdot Q_{I,i} + C_2 \cdot Q_{E,i}$$

and three coefficients C_0, C_1 and C_2 specific for the Muskingum model.

$$C_0 = \frac{0{,}5 \cdot \Delta t - K \cdot X}{K \cdot (1-X) + 0{,}5 \cdot \Delta t}, \quad C_1 = \frac{0{,}5 \cdot \Delta t + K \cdot X}{K \cdot (1-X) + 0{,}5 \cdot \Delta t} \text{ and } C_2 = \frac{K \cdot (1-X) - 0{,}5 \cdot \Delta t}{K \cdot (1-X) + 0{,}5 \cdot \Delta t}$$

For numerical stability the following boundary conditions are to be fulfilled:

$K \geq \Delta t$ …for being able to reproduce the flowing wave within the timely grid Δt

$C_0, C_1, C_2 \geq 0$ … all terms are required to – positively – contribute to a generated outflow

This leads to the final relation to be fulfilled for combinations of *K, X* with constant time steps *Δt*:

$$\frac{1}{2\cdot(1-X)} \le \frac{K}{\Delta t} \le \frac{1}{2\cdot X}$$

Further, the dimensionless weighing factor *X* has to fulfil that

$$\frac{1}{2\cdot(1-X)} \le \frac{1}{2\cdot X}\ .$$

Consequently, valid numerical values for *X* are in the range between 0 and 0.5. Further, the two extremes for *X* define special cases. The lower limit (*X=0*) represents the linear hydraulic storage where the upper limit (*X=0.5*) equals the translation without peak damping.

3.2.2.5 Muskingum – Method of Flood Routing with subreaches

Applying the Muskingum Method for flood routing with multiple subreaches, the same equations are generally used. Adaptations are needed with regard to nomenclature that needs to include the numbering of subreaches *j (1...n)* (see Figure 13).

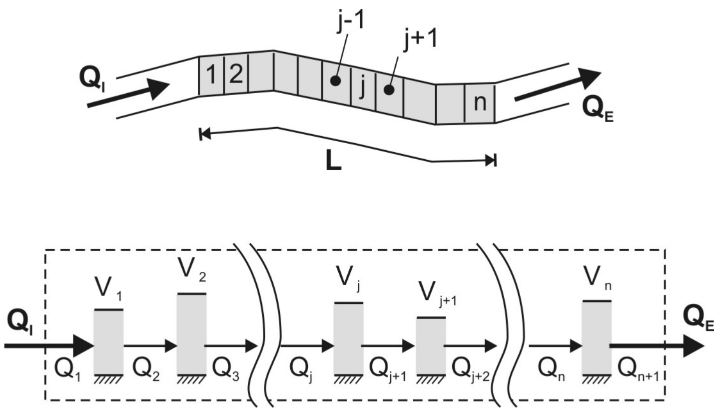


Figure 13: Schematic on nomenclature for multiple subreaches

The Muskingum parameter *K* applies to the total reach. For simplicity a reach is always split for *n* equal subreaches, each having an associated Muskingum parameter *K'*:

$$K' = K/n$$

The flow from the subreach *j* can be written as

$$Q_{i+1}^{j+1} = C_0 \cdot Q_{i+1}^{j} + C_1 \cdot Q_i^{j} + C_2 \cdot Q_i^{j+1}$$

using

$$C_0 = \frac{0{,}5 \cdot \Delta t - K' \cdot X}{K' \cdot (1-X) + 0{,}5 \cdot \Delta t} \;;\; C_1 = \frac{0{,}5 \cdot \Delta t + K' \cdot X}{K' \cdot (1-X) + 0{,}5 \cdot \Delta t} \;;\; C_2 = \frac{K' \cdot (1-X) - 0{,}5 \cdot \Delta t}{K' \cdot (1-X) + 0{,}5 \cdot \Delta t}$$

Current Volume stored for a subreach *j* is calculated as:

$$V_{i+1}^{j} = K' \cdot \left[X \cdot Q_{i+1}^{j} + (1-X) \cdot Q_{i+1}^{j+1})\right]$$

and the total volume in the reach is

$$V_{i+1} = \sum_{j=1}^{n} V_{i+1}^{j}$$

3.3 POLLUTANT TRANSPORT AND PROCESS MODELLING

The transport of pollutants is in principle based on the conservation of mass. Mass balance applies within defined system boundaries involving storage-, transport- and conversion (reaction) processes.

The storage of matter is represented by the temporal variation of mass in the system

$$\frac{dM}{dt} = \frac{d(V \cdot C)}{dt} = C \cdot \frac{d(V)}{dt} + V \cdot \frac{d(C)}{dt}$$

with

M	…Pollutant mass [g]
C	…Pollutant concentration [g/m3]
V	…Volume [m^3]

For conservative matter, mass fluxes from ($Q_I \cdot C_I$) and to the system ($Q_E \cdot C_E$) maintain the equilibrium. Reaction terms for non-conservative matter are added to the equation:

$$V \cdot \frac{d(C)}{dt} = -C \cdot \frac{d(V)}{dt} + Q_I \cdot C_I - Q_E \cdot C_E + r \cdot V$$

r	…reaction rate [g/(m^3.s)]

The reaction rate is defined positive (*r > 0*) in case of production and negative (*r<0*) in case of consumption. Transport modelling may be based on either physical or conceptual models.

3.3.1 ADVECTION-DISPERSION TRANSPORT

Transport of matter in a moving fluid is based on the transport mechanisms advection, diffusion and shear dispersion. Advection is the transport along with the fluid itself. Diffusion is the mixing of contaminants that is driven by contaminant concentrations. Shear dispersion is the mixing due to velocity gradients. Both diffusion and shear dispersion are - in the field of groundwater and surface water modelling - summarized as dispersion using one net dispersion coefficient. For the one-dimensional case the dispersion relation is denoted as:

$$Jx = Dx \cdot \frac{\partial C}{\partial x}$$

with

Jx	mass flux of contaminants per unit area [g/(m^2.s)]
Dx	Diffusion coefficient [m^2/s]

The temporal change of concentration is thereby

$$\frac{dC}{dt} = D_x \cdot \frac{\partial^2 C}{dx^2}$$

The full advection-dispersion equation reads as:

$$\frac{\partial C}{\partial t} = -v \cdot \frac{\partial C}{\partial x} + D \cdot \frac{\partial^2 C}{\partial x^2} + r$$

3.3.2 IDEAL REACTORS

Usually, the advection dispersion equation is simplified for different practical aspects by the conceptual model of an ideal reactor. The most common ideal reactor types are

- Full mixed batch reactor
- Continuous flow stirred tank reactor (CSTR)
- Plug flow reactor (PFR)
- Plug flow reactor with dispersion (PFRD)

In the following the reactors are briefly presented with regard to their simplifications and main equations. Besides, a discrete formulation of the ideal mixing process is shown.

3.3.2.1 Full mixed batch reactor

The batch reactor is characterized by having neither inflow nor outflow. This implies that the volume (*V*) of the reactor is kept constant. Applying the simplifications that no flows $Q_I = Q_E = 0$ occur and no change in volume ($dV/dt = 0$) is given. The general mass balance equation reduces to:

$$\frac{d(C)}{dt} = r \qquad \rightarrow \qquad \frac{\Delta C}{\Delta t} = r$$

Only a change in substance concentration, due to internal reaction, occurs in the reactor.

3.3.2.2 Continuous flow stirred tank reactor (CSTR)

The continuous flow stirred tank reactor (CSTR) allows, different to the batch reactor, flow through the system (see Figure 14).

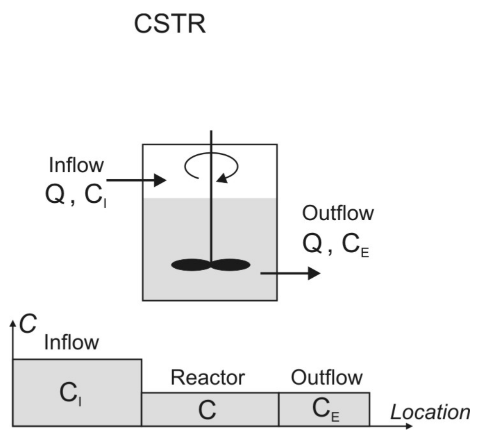


Figure 14: Schematic of a continuous stirred tank reactor (CSTR)

Limitation is that the volume of the reactor is constant, thus the water flux to and from the reactor are equal ($Q_I = Q_E$; and $dV/dt = 0$). Due to the ideal mixing in the reactor the concentrations in the effluent and the reactor are considered to be the same ($C = C_E$).

$$V \cdot \frac{d(C)}{dt} = Q \cdot (C_I - C) + r \cdot V$$

3.3.2.3 Plug flow reactor (PFR)

The plug flow reactor considers, different to the batch reactor and the CSTR, no longitudinal mixing. As internal process in longitudinal direction, advection (transport by the flowing water) is applied. This enables the development of concentration gradients along the flow axis in the reactor.

The plug flow reactor can be interpreted as a series of batch reactors, being transported (moved) according to the flow velocity in the plug flow reactor. There is no flow, subsequently no mixing, between the batch reactors. Within each reactor internal process may be applied, causing degradation of increase of substances over time ($dC/dt = r$).

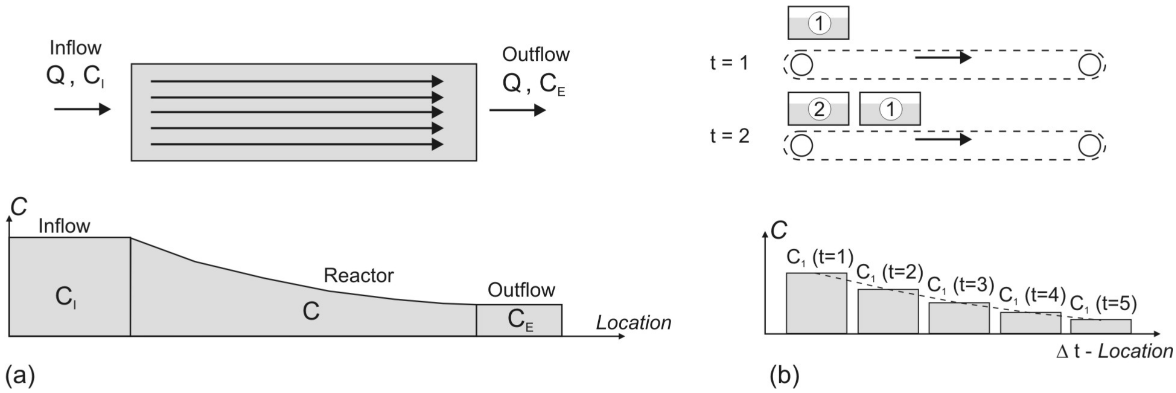


Figure 15: Plug Flow reactor (a) analytical and (b) numerical interpretation

A numerical interpretation would be a row of reactors, where the full content of a chamber is transferred into the subsequent one within discrete time steps. The volume of each chamber is a matter of required accuracy for the longitudinal distribution of concentration. For a defined total volume (V_T) of a modelled plug flow reactor and a fixed number of modelled batch reactors (n), the (equal) volume (Vi) for each batch reactor would be $Vi = V_T / n$.

Having a defined (and constant) flow Q, the time steps Δt at which the volume Vi is transferred further downstream can be calculated as $\Delta t = Vi / Q$.

3.3.2.4 Plug flow reactor with dispersion (PFRD)

The CSTR and the PFR represent extremes that usually do not occur as such in reality. The plug flow reactor with dispersion represents a case in between the two extremes. The continuity equation is given as:

$$\frac{\partial C}{\partial t} = -v \cdot \frac{\partial C}{\partial x} + D \cdot \frac{\partial^2 C}{\partial x^2} + r$$

For a conservative tracer with reaction rate $r=0$, the equation can be solved explicitly, which is not presented in here.

3.3.2.5 Discrete scheme for ideal mixing

For derivation of a discrete formulation of substance mixing, reactions are neglected ($r = 0$). Reaction kinetics are to be dealt with in an extra calculation step. The mass balance is reformulated for a discrete time step Δt having t_i as current point of time and t_{i-1} as previous point of time. Changes of flow, volume or concentration during a time step are considered to be linear. Thus for balancing, the timely derivatives can be written as

$$\frac{d(V)}{dt} = \frac{\Delta V}{\Delta t} = \frac{V_i - V_{i-1}}{\Delta t} \quad \text{and} \quad \frac{d(C)}{dt} = \frac{\Delta C}{\Delta t} = \frac{C_i - C_{i-1}}{\Delta t}$$

Concentrations, volumes and flows are taken into account as mean values, resulting from the states at beginning and end of the time step (t_{i-1} and t_i). The differential equation

$$C \cdot \frac{d(V)}{dt} + V \cdot \frac{d(C)}{dt} = Q_I \cdot C_I - Q_E \cdot C_E$$

is discretised as:

$$\frac{C_{V,i} + C_{V,i-1}}{2} \cdot \frac{V_i - V_{i-1}}{\Delta t} + \frac{V_i + V_{i-1}}{2} \cdot \frac{C_{V,i} - C_{V,i-1}}{\Delta t} = \frac{Q_{I,i} + Q_{I,i-1}}{2} \cdot \frac{C_{I,i} + C_{I,i-1}}{2} - \frac{Q_{E,i} + Q_{E,i-1}}{2} \cdot \frac{C_{E,i} + C_{E,i-1}}{2}$$

Similar to a CSTR full mixing of the chamber content and the inflow is assumed prior to the flow leaving the tank. Therefore $C_{V,i} = C_{E,i}$ and $C_{V,i-1} = C_{E,i-1}$ resulting in

$$C_{E,i} = C_{V,i} = \frac{\frac{\Delta t}{2} \cdot (Q_{I,i} + Q_{I,i-1}) \cdot (C_{I,i} + C_{I,i-1}) + C_{V,i} \cdot \left[2 \cdot V_{i-1} - \frac{\Delta t}{2} \cdot (Q_{E,i} + Q_{E,i-1}) \right]}{\left[2 \cdot V_i + \frac{\Delta t}{2} \cdot (Q_{E,i} + Q_{E,i-1}) \right]}$$

For implementation in CITY DRAIN an even simpler discrete scheme was used. By definition, flows and concentrations exchanged during a time are considered as mean concentrations. over the respective time step *Δt*. Thus, terms on the right hand side of the mass balance equation are not needed to be discretised any further.

$$C \cdot \frac{d(V)}{dt} + V \cdot \frac{d(C)}{dt} = Q_I \cdot C_I - Q_E \cdot C_E$$

They already represent the mean flux between t_{i-1} and t_i . Thus, the starting point for the discrete scheme is:

$$\frac{C_{V,i} + C_{V,i-1}}{2} \cdot \frac{V_i - V_{i-1}}{\Delta t} + \frac{V_i + V_{i-1}}{2} \cdot \frac{C_{V,i} - C_{V,i-1}}{\Delta t} = Q_I \cdot C_I - Q_E \cdot C_E$$

The outflow concentration being equal to the chamber concentration ($C_E = (C_{V,i} + C_{V,i-1})/2$) leads to

$$C_{V,i} = \frac{Q_{I,i} \cdot C_{I,i} - C_{V,i-1} \cdot \left[\frac{Q_{E,i}}{2} - \frac{V_{i-1}}{\Delta t} \right]}{\left[\frac{Q_{E,i}}{2} + \frac{V_i}{\Delta t} \right]} \qquad \text{and} \qquad C_{E,i} = \frac{C_{V,i} - C_{V,i-1}}{2}$$

Numerical stability is guaranteed since the denominator is positive in all cases. The numerator is required to be positive as well in order to avoid negative concentrations.

3.4 PROCESSES

In the following, the principals in modelling of bio-chemical processes such as ASM (Activated Sludge Model) type processes shall be shown. The focus is not on process detail in activated sludge modelling but on the modelling concept, in principle using a Petersen matrix format (Henze *et al.*, 2000). Therein model components, the processes and the associated process rates are organized in a structured way.
Considering the mass balance, it is seen that changes in mass due to processes are included as an additional term (*r.V*) in the equation.

$$\frac{dM}{dt} = C \cdot \frac{d(V)}{dt} + V \cdot \frac{d(C)}{dt} = Q_I \cdot C_I - Q_E \cdot C_E + \boxed{r \cdot V}$$

The reaction rate *r* is defined positive ($r > 0$) in case of production and negative ($r<0$) in case of consumption.

The model is built up in matrix format where processes are organized in rows and model components in columns. Additionally, process rates ρ are formulated for each process used. The full ASM matrices can be found in (Henze *et al.*, 2000). In the following, the model concept is shown for the heterotrophic bacterial growth and decay. The relevant substances considered are:

X_h …hetherotrophic biomass
S_S …soluable subtrate
S_O …dissolved oxygen

Table 4 shows the Petersen matrix for the considered processes and components.

Table 4: Process kinetics and stoichiometric for heterotrophic bacterial growth and decay

Component / process		1	2	3	
		X_H	S_s	S_O	process rate ρ [$ML^{-3} T^{-1}$]
1	Heterotrophic bacterial growth	1	-1/Yh	-(1-Yh)/Yh	$\bar{\mu}_h \cdot X_h \cdot \frac{S_S}{K_S + S_S} \cdot \frac{S_O}{K_o + S_O}$
2	Heterotrophic bacterial decay	-1		-1	$b \cdot X_h$
		Heterotrophic biomass	soluble substrate	Dissolved oxygen (as negative COD)	

Within the defined processes, substances are produced or utilized. As mass balance applies within each process, no “new” matter is to be produced. The quantitative interrelation between substances as they change is defined via stoichiometric coefficients. For instance the quantity of heterotrophic biomass grown is related to the substrate S_S used via a heterotrophic yield factor (*Yh*).

$$X_h = -S_S \cdot Y_h$$

The respective stoichiometric coefficient $a_{2,1}$ denotes as

$$a_{1,2} = -\frac{1}{Y_h}$$

The indices used correspond to the numbering of processes and components respectively. In the stoichiometric parameters, production and consumption is indicated as positive or negative parameters. The mass balances apply for each process, where the sum of stoichiometric coefficients in a row must be zero.

The reaction rate for a substance *i* is obtained as the sum of productions and consumptions per time step. For *j* associated processes, the reaction rate for component *i* is:

$$r_i = \sum_j a_{j,i} \cdot \rho_j$$

For the heterotrophic biomass, the reaction rate would therefore be

$$r_{Xh} = \bar{\mu}_h \cdot X_h \cdot \frac{S_S}{K_S + S_S} \cdot \frac{S_O}{K_o + S_O} + b \cdot X_h$$

The process rates ρ_j are determined by the kinetic parameters and the current concentration of the substance available.

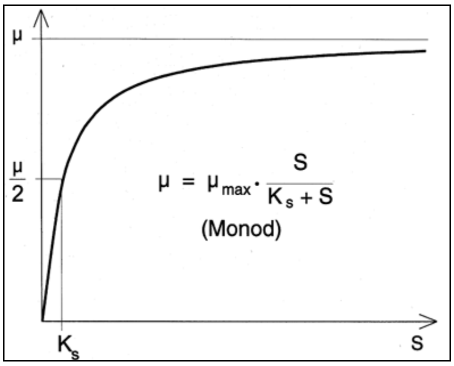


Figure 16: Growth rate according to Monod kinetics

For instance, the growth rate constant (μ) is a function of the substrate concentration *S* and two parameters μ_{max} and K_S.

$$\mu = \overline{\mu}_{\max} \cdot \frac{S}{K_S + S}$$

The first constant is μ_{max}, being the maximum growth rate constant. This is the rate to which the growth rate is limited to, regardless how much substrate is available. K_s is the half-saturation coefficient corresponding to the concentration of *S* when $\mu = \mu_{max}/2$.

The concept described is used by a number of models such as the activated sludge models ASM1, 2, 2d and 3 (Henze *et al.*, 2000) or the River Water Quality Model No. 1 by Reichert *et al.* (2001a).

3.5 CONCLUSIONS

In this chapter the basic modelling concepts in urban drainage were presented. Thereby the most relevant mechanism were covered, describing the routing of water and pollutants on their way from the source of generation to the receiving water. The fundamental equations and concepts typically used to describe

- rainfall runoff process,
- hydraulic transport routing,
- pollutant transport/routing and
- pollutant processes

were presented.

Both, physical and conceptual approaches were covered for hydraulic and pollutant transport. The focus was put on the conceptual models as these are used in the software CITY DRAIN. Limitations of both physical and conceptual models were discussed. In hydraulic modelling, physical models such as the St. Venant Equation enable to assess processes at a very detailed level. Using for instance the full St. Venant Equation enables to deal with back pressure effects. As this is required for detailed assessment of pipe hydraulics, these equations are the most commonly used ones for assessment of sewer systems. A disadvantage of the St. Venant equations is the high computational effort that limits its application for predominantly short simulation periods. Conceptual model being significantly less demanding in computational effort. They are applicable as long as not detailed hydraulic assessment of sewer hydraulics is required. When aiming to assess for instance surcharge and overflow of the sewer system itself, application of physical models is required. Thus, depending on the aim of the study, different models may be suitable. This is true for hydraulics, as well as for pollutant routing and pollutant processes.

As example for pollutant processes, the general concept of ASM type models based on the Petersen Matrix was presented. The focus was not on showing the Activated Sludge Model (ASM) type models in detail, but presenting the modelling concept itself. The presented concept is used as well for other biological-chemical processes such as the River Water Quality Model (RWQM) No. 1.

4 CITY DRAIN – MODELLING APPROACHES

4.1 THE MATLAB/SIMULINK ENVIRONMENT

To cope with aims of the EU-WFD, software tools are required that are capable of modelling urban drainage systems (including the receiving water) in an integrated manner. Rainfall as the elementary input source is of irregular occurrence in intensity and duration, which leads to the need of long-term simulations for being capable of evaluating a system's performance.

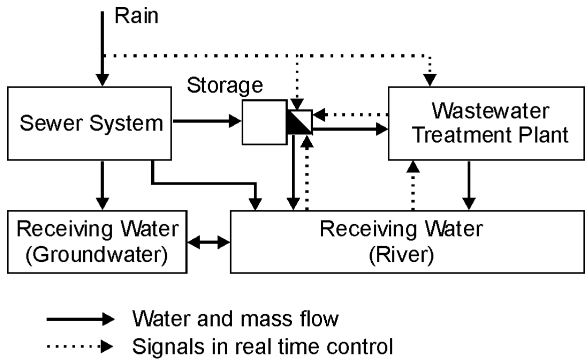


Figure 17: Schematic on the main elements and information flow in an integrated model (redrawn from Rauch et al. (2002a))

Realisation of a block wise representation of parts of the urban drainage system as in Rauch *et al.* (2002a) is based on the state-space approach as shown by (Schreider *et al.*, 2001). The principles of state-space modelling have been formulated by (Kalman, 1960) and it is since then widely used. The Matlab/Simulink© environment is using the same principles embedded in a graphical user interface to arrange blocks (The Mathworks, 2003). In principle input (*u*) is fed to a state-space model that is variable in time. A dynamic output is generated, based on both, the dynamic input (*u*) and the model's state (*x*).

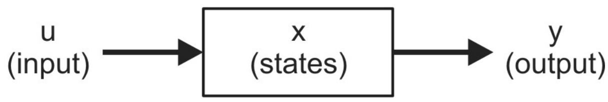


Figure 18; State-space model

The state (*x*) of the system is defined as the values of state variables at any instant point of time. When modelling urban drainage systems this may be the current volume *V* or the current substance concentration *C*. The utilization of states is the core element, where the change of the states is defined via mathematical equations. This can be either differential equations or – as used in CITY DRAIN – discrete formulations of the differential equations. As an additional information, the state-space model accepts parameters which are constant in time. A linear state-space model is described by the following equations

$$\frac{dx}{dt} = Ax + Bu$$
$$y = Cx + Du$$

The set of differential equations describing the output as function of states and input are to be solved for each time step. Different numerical methods for solving the differential equations are available. The time steps for which the equations are solved relate to the accuracy of the solution. Depending on the software, the time steps can be implemented as variable or – as in CITY DRAIN – as fixed time steps. Usually when implementing a dynamic system, the time management is to be organised by the programmer. Within the Simulink environment, a defined scheme is already provided that takes care of the time management. Thus, discretised equations are to be formulated for a single time step and to be embedded in the provided scheme.

The scheme is realized in so called s-functions that are available for different software languages (Matlab, C++ or MEX functions). Within the graphical environment each block addresses a certain s-function. Figure 19 shows the temporal workflow of initialization, output and update procedures.

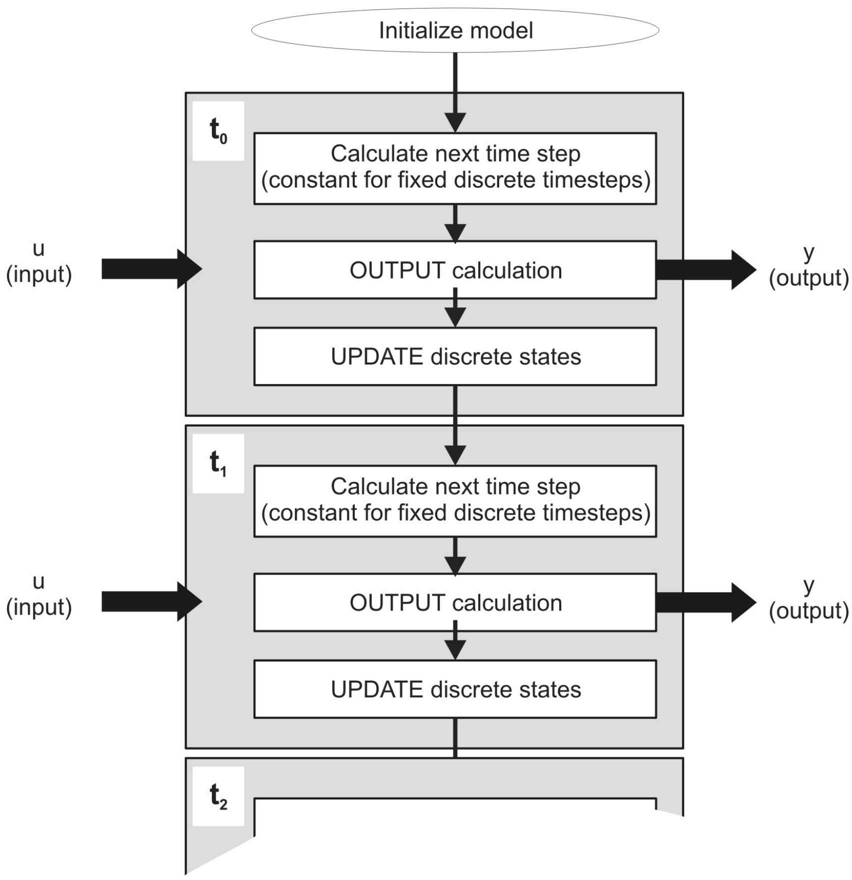


Figure 19: Simulink workflow for Initialization, Update and Output

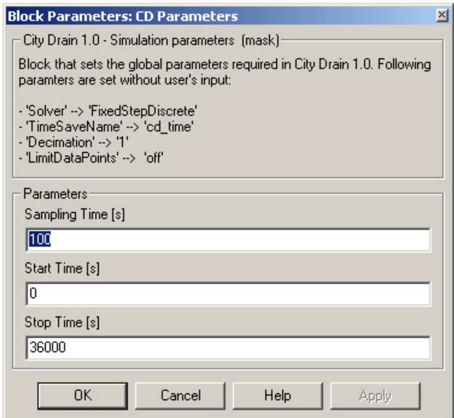
When the model is started, the defined state variables are to be set within the initialization step. This initial setting of the state variable is used within the first output calculations at $t = 0$. Thus, one can virtually attribute the initial states to the time step $t = -1$. Subsequently, output and update procedures are redone for each time step.

4.2 CITY DRAIN – THE LIBRARY AND CONVENTIONS

CITY DRAIN and the implemented library blocks are designed to work within a discrete time scheme. Constant and discrete time steps are used within a simulation where simulation time and size of time steps are to be chosen by the user. Models implemented for hydraulics and mass transport are formulated for discrete time steps Δt.

Core element of every CITY DRAIN simulation is the block **"CD - Simulation Parameters"**

CD Parameters



This block ensures that simulation parameters in the Matlab/Simulink © are set correctly. User input is required for the

- sampling time Δt [s],
- start time t_0 and
- stop time t_E.

of the simulation. The sampling time defined is utilized within all CITY DRAIN blocks provided, thus it is being globally used. Hidden settings (without required user input) are made for

- 'Solver' .... 'FixedStepDiscrete'
- 'TimeSaveName'.... 'cd_time'
- 'Decimation' .... '1'
- 'LimitDataPoints' ... 'off'

Within version 2 of CITY DRAIN the "central data management" embedded in the Simulation parameters is extended for dealing with substance. Associated concentrations in a stream are associated to names and are therefore identifiable. Having substances identified not only by number but also by name makes it easier to implement them within processes. Conventions used in City Drain II are kept the same as before:

Q …Flow [m^3/s]
V …Volume [m]
L …Length [m]
t, Δt …Time [s]
C …Concentrations [g/m^3]
M …Mass [g]

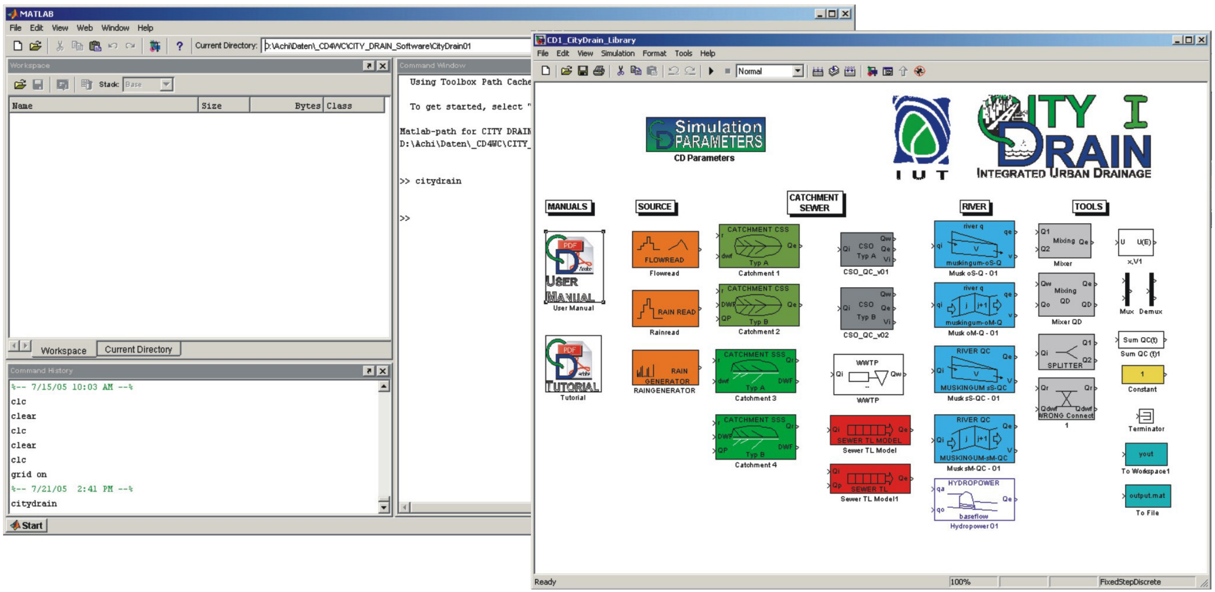


Figure 20: Screenshot, CITY DRAIN I © Library

Figure 20 shows a screenshot of the CITY DRAIN I library realized in Matlab/Simulink. The underlying models that are implemented in the different compartments are explained in more detail in the following sections respectively in the manual attached to this work (Achleitner and Rauch, 2005b). Building ones' own model is simply done by drag and drop of blocks from the library. An example showing step by step how to build a model is included in the tutorial part of the manual. Data assessment and calibration is discussed in Chapter 5.

4.3 IMPLEMENTED BLOCKS AND MODELLING CONCEPTS

This section includes details on specific blocks of the CITY DRAIN library covering

- Specification of the block (how to use)
- The models behind and their derivation

Current models include the fluid phase exclusively using tracer substances for pollutant routing. Extension for describing processes in the soluble phase may be included. This is the case for the WWTP currently being under development. As CITY DRAIN is open source other process may be included as well. Especially version 2 of CITY DRAIN supports the definition and assessment of substance flows.

4.3.1 SOURCE BLOCK - RAINREAD

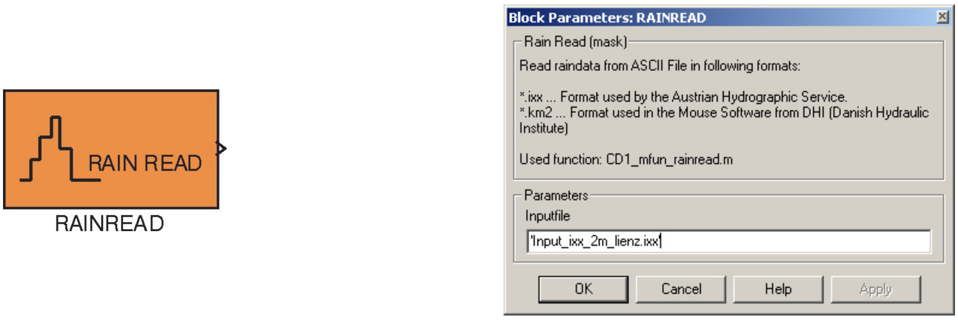


Figure 21: (a) Block rain read and (b) the blocks parameter mask

Table 5: Syntax of supported rain formats; left: mse-format, right: ixx-format

mse-format							*ixx-format*	
YY	MM	DD	hh	mm	ss	$r_R[10^{-3}mm/s]$	DD.MM.YYYY.hh.mm.ss	$r_R[mm/\Delta t]$
81	1	1	22	40	0	0.0000	01.01.1991 00:00:00	0.1
81	1	1	22	50	0	0.0000	01.01.1991 00:05:00	0.1
81	1	1	23	0	0	0.1670	01.01.1991 00:10:00	0.1
81	1	2	4	30	0	0.1670	01.01.1991 00:15:00	0.1
81	1	2	4	40	0	0.0000	01.01.1991 00:20:00	0.1

For rain data, two formats, *mse* and *ixx* , are supported.
The syntax of both, using date and time formats, is shown in Table 5. The *mse* and *ixx* formats are used by the national weather service MeteoSwiss and the Austrian Hydrographic Service respectively. For both formats dates are read and transferred into consecutive numerical values in seconds. Output of rain data is cumulated volume in millimetres per time step (e.g. mm/5min). The *mse*-format stores rain data only for rain events, neglecting dry periods. Missing gaps are therefore filled when data is read from the file. Where the *ixx* format already provides rain data in [mm/ΔT], data read from *mse* format is to be converted from a different unit [10^{-3} mm/s].

$$r[mm/\Delta t] = R[10^{-3}mm/s] \cdot 0.001 \cdot \Delta t[s]$$

In case time steps of the raw data ΔT do not match the simulation time steps Δt, interpolation/cumulation of rainfall is made.

4.3.2 SOURCE BLOCK - FLOWREAD

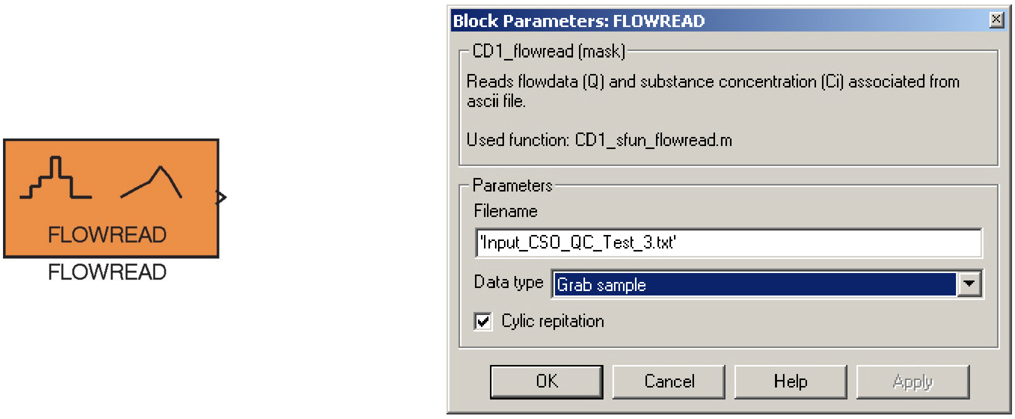


Figure 22: (a) Block flow read and (b) the blocks parameter mask

The simplest formats are used for flow data containing either a time series of flow rates or flow rates and associated substance concentrations. Measured series for flow and/or associated concentrations may be read using the block "flowread". Table 6 provides an example input time series where the number of concentrations is flexible.

Table 6: Syntax for flow and concentration series

t [s]	q [m^3/s]	C_1 [g/m^3]	C_2 [g/m^3]	.....C_x [g/m^3]
0	0.40	0.16	0.06	
900	0.45	0.20	0.02	
1800	0.50	0.25	0.70	
2700	0.60	0.36	0.15	
3600	1.00	1.00	0.30	
4500	2.00	4.00	0.04	
5400	0.50	0.25	0.06	
6300	0.30	0.09	0.18	
7200	0.40	0.16	0.22	

Data is differently treated as being interpreted as either grab samples or composite samples (see Figure 23).

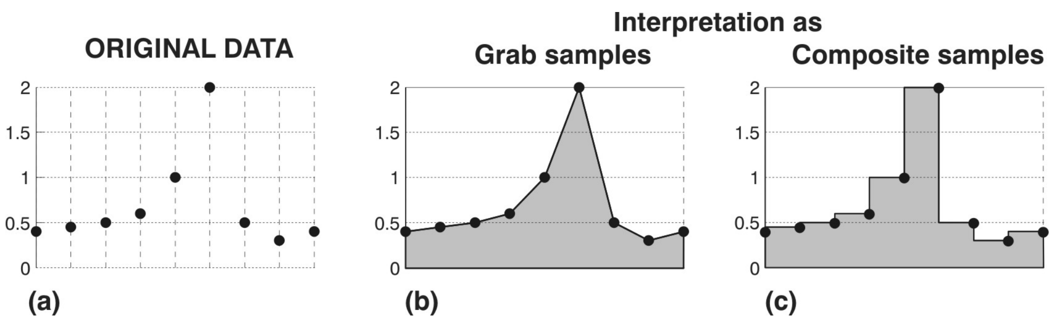


Figure 23: Interpretation of (a) stored data as either (b) grab samples or (c) composite samples.

This differentiation is of special importance, since data may not necessarily be provided in the same temporal resolution as applied for modelling. For interpolated data points, the type of dataset (grab or composite samples) is of importance and would lead to wrong results if interpreted wrongly.

For grab sample data, the distribution of flow and concentrations is assumed to be linear over ΔT between data points. In the case of composite samples the data read represents mean flows/concentration over the past time step ΔT.

The output generated from this block is always of type “composite sample” regardless what type of raw data was used. The values represent the mean flow/mean concentration over the last time step Δt used in the model (not the raw data).

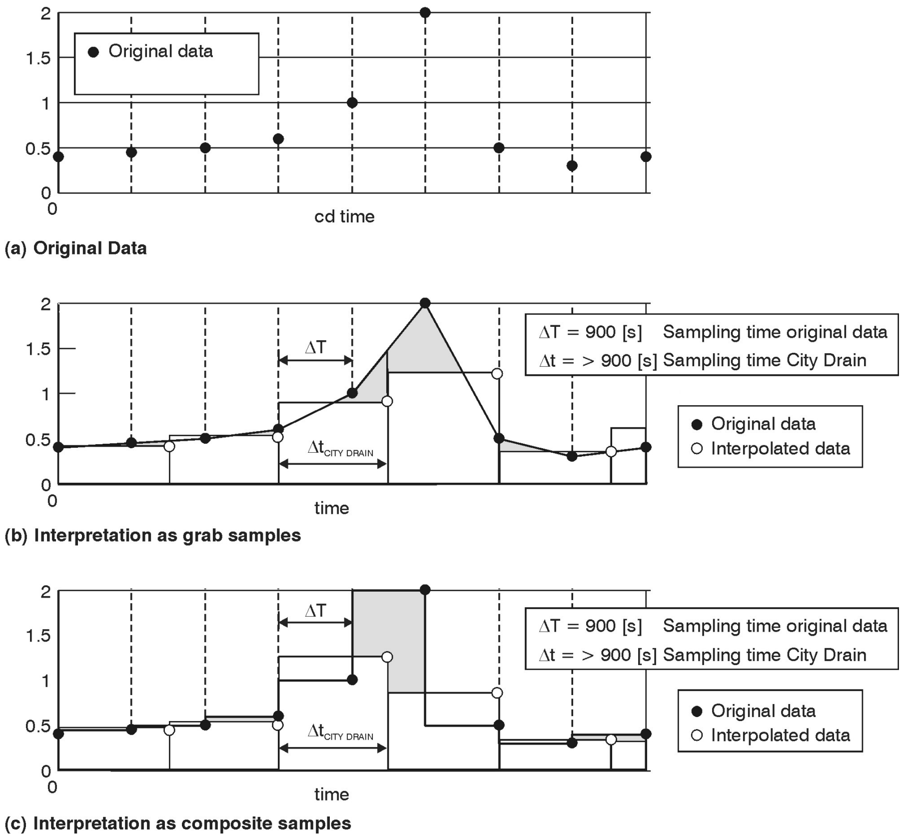


Figure 24: Schematic of interpolation procedure in case ΔT (raw data) and Δt (simulation) are not equal

The interpolation algorithm applied is based on the principal of conservation of mass. Volume (V) and mass flux (M) over each time step are integrated and are maintained when transferred to sampling steps used in the simulation (Δt). The distribution in between measurement points (of the raw data) is – in absence of deeper knowledge – assumed to be linear for both, the flux (Q) and concentrations (C). A linear relation for interpolated points can be written as

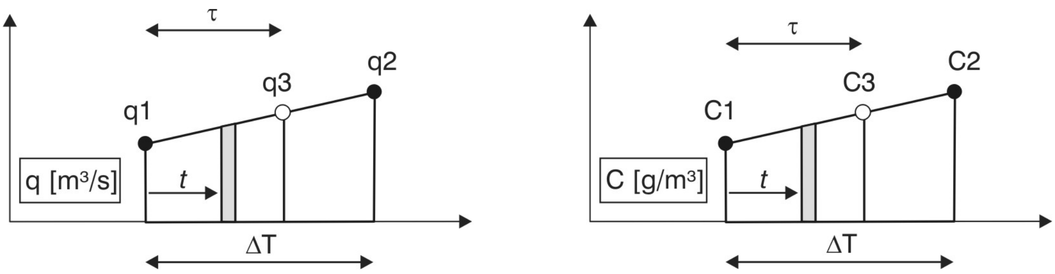


Figure 25: Linear distribution of q and C between measured points

$$q(t) = q_1 + \frac{q_2 - q_1}{\Delta T} \cdot t \quad \text{and} \quad C(t) = C_1 + \frac{C_2 - C_1}{\Delta T} \cdot t$$

The volume discharged in the interval ΔT can be written as

$$V = \int_0^{\Delta T} q(t) \cdot dt = \frac{q_1 + q_2}{2} \cdot \Delta T$$

where the pollutant load discharged is

$$M_{TOT} = \int_0^{\Delta T} q(t) \cdot C(t) \cdot dt = \frac{q_1 \cdot (2 \cdot C_1 + C_2) + q_2 \cdot (C_1 + 2 \cdot C_2)}{6} \cdot \Delta T$$

The more complex solution of the integral is due to *M(t)* being a quadratic equation. The mean concentration over time is obtained as quotient of mass and volume.

$$\overline{C} = \frac{M_{TOT}}{V} = \frac{q_1 \cdot (2 \cdot C_1 + C_2) + q_2 \cdot (C_1 + 2 \cdot C_2)}{3 \cdot (q_1 + q_2)}$$

where the calculation of the mean concentration as the arithmetic mean of C_1 and C_2 would lead to a result not being mass conservative. For the special case of $q_1 = q_2$ and $C_1 + C_2$ as is the case for composite samples the equations reduce to

$$V = q_1 \cdot \Delta T \quad \text{and} \quad M_{TOT} = q_1 \cdot C_1 \cdot \Delta T$$

Obviously the mean concentration reduces to $\overline{C} = C_1 = C_2$. For an interpolated point in time *(t_3)* located between the two measurements, it is necessary to calculate the cumulated volumes *V*:

$$V_{1-3}(\tau) = \int_0^{\tau} q(t) \cdot dt = \frac{\tau^2 \cdot (q_2 - q_1)}{2 \cdot \Delta T} + q_1 \cdot \tau$$

$$V_{3-2}(\tau)=\int_{\tau}^{\Delta T} q(t)\cdot dt=\frac{q_1+q_2}{2}\cdot\Delta T-\frac{\tau^2\cdot(q_2-q_1)}{2\cdot\Delta T}-q_1\cdot\tau$$

the τ is the time between the first measurement and the interpolated point in time. Corresponding loads *M* are

$$M_{1-3}(\tau)=\int_0^{\tau} q(t)\cdot C(t)\cdot dt$$

$$M_{1-3}(\tau)=\frac{\tau\cdot\left(2\cdot\tau^2\cdot(q_1-q_2)\cdot(C_1-C_2)+3\cdot\Delta T\cdot\tau\cdot(C_1\cdot q_2-q_1\cdot(2\cdot C_1-C_2))+6\cdot q_1\cdot C_1\cdot\Delta T^2\right)}{6\cdot\Delta T^2}$$

$$M_{2-1}(\tau)=\int_{\tau}^{\Delta T} q(t)\cdot C(t)\cdot dt$$

$$M_{2-1}(\tau)=\frac{q_1\cdot(2\cdot C_1+C_2)+q_2\cdot(C_1+2\cdot C_2)}{6}\cdot\Delta T\ldots$$

$$\ldots-\frac{\tau\cdot\left(2\cdot\tau^2\cdot(q_1-q_2)\cdot(C_1-C_2)+3\cdot\Delta T\cdot\tau\cdot(C_1\cdot q_2-q_1\cdot(2\cdot C_1-C_2))+6\cdot q_1\cdot C_1\cdot\Delta T^2\right)}{6\cdot\Delta T^2}$$

The mean concentration over both time periods, is again obtained as:

$$\overline{C_{1-3}}=\frac{M_{1-3}(\tau)}{V_{1-3}(\tau)} \quad \text{and} \quad \overline{C_{3-2}}=\frac{M_{3-2}(\tau)}{V_{3-2}(\tau)}$$

4.3.3 IMPLEMENTED LOSS MODEL

The implemented loss model is used within the catchment blocks only. A simple method to account for the initial loss *hi [mm]* is the application of a basin-methodology. The volume of the basin (respectively the height) represents the volume of water retained due to initial losses *hi*. The volume of rain exceeding the basin's volume is considered to be the effective precipitation, contributing to the catchment surface flow.

Permanent losses *hp [mm/Δt]* such as evapotranspiration can be either considered acting all the time or during dry weather only. Either case, the volume per time step to be evaporated is limited by the initial loss specified.

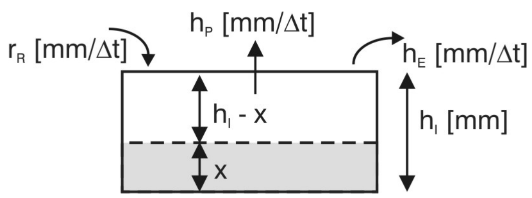


Figure 26: Schematic of loss model with initial loss (hi) and permanent loss (hp)

Considering r_i as the volume of rain per time step, *xi* as the current volume retained and i denoting for the time step, the mass balance for the effective runoff is:

$$h_{E,i} = r_{R,i} - \left(h_{I,i} - x_i\right) - h_{P,i} \geq 0$$

This formula stated considers evaporation during rainfall as well as during dry periods. When considering evaporation during dry weather, only the evaporation term on the right hand side is set to be zero. The current volume stored changes from one time step to another by:

$$x_i = x_{i-1} + r_{R,i} - h_{P,i} \qquad \text{for} \qquad 0 \leq x_i \leq h_{Ii}$$

Again, if evaporation is wanted only for dry periods, differentiation of dry and wet periods is required, considering *hp* for *r=0* or neglecting *hp* for *r>0*. Accounting for the given areas in a catchment being permeable or impervious is done using the runoff coefficient φ.

$$\varphi = A_E / A_{TOT}$$

The effective area A_E account herein mainly for the contribution of urbanized, impermeable areas. Applying the runoff coefficient to h_E leads to the effective precipitation height *he* contributing to the surface flow.

$$he = \varphi \cdot h_E = \varphi \cdot \left(r_{R,i} - \left(h_{I,i} - x_i\right) - h_{P,i}\right)$$

4.3.4 Muskingum model - simplified discrete scheme

The in the following presented simplified discrete scheme for the Muskingum method is used in following CITY DRAIN blocks:

- Catchment models
- Sewer model
- River model

Usage of the blocks is shown in the user manual attached as Appendix F. The catchment block is shown in some more detail here as loss models and flow routing are combined. In contrast to the Muskingum flow routing, derivation is based on a simplified discrete scheme. Flows and associated concentrations are considered as mean concentrations.

Rainfall measurements are likely given as rain volumes, accumulated during a certain period of time (usually in the range of 5 – 10 min intervals). The rain r_i [mm] is the volume of rain fallen between t_{i-1} and t_i time. Consequently the derived flow does not describe the actual flow at time t_i but the mean flow between t_{i-1} and t_i anyway. Therefore the Muskingum equation (Roberson *et al.*, 1995) is newly derived using a simplified numerical scheme (Achleitner *et al.*, 2006b; Motiee *et al.*, 1997).

$$V = \frac{V_i + V_{i-1}}{2} = K \cdot Q_{E,i} + K \cdot X \cdot (Q_{I,i} - Q_{E,i})$$

$$\frac{\Delta V}{\Delta t} = \frac{V_i - V_{i-1}}{\Delta t} = Q_{I,i} - Q_{E,i}$$

Eliminating the stored volume V_i leads to

$$2 \cdot K\left[X \cdot Q_{I,i} + (1-X) \cdot Q_{E,i}\right] - V_{i-1} = \left(Q_{I,i} - Q_{E,i}\right) \cdot \Delta t + V_{i-1}$$

Consequently the mean outflow $Q_{E,i}$ is

$$Q_{E,i} = C_X \cdot Q_{I,i} + C_Y \cdot V_{i-1}$$

with

$$C_X = \frac{\frac{\Delta t}{2} - K \cdot X}{\frac{\Delta t}{2} + K \cdot (1-X)} \quad \text{and} \quad C_Y = \frac{1}{\frac{\Delta t}{2} + K \cdot (1-X)}$$

In contrast to the original discrete scheme used, only two instead of three parameters are required. The volume stored at time t_i is

$$V_i = \left(Q_{I,i} - Q_{E,i}\right) \cdot \Delta t + V_{i-1}$$

For numerical stability, different requirements are to be fulfilled. For assessing the wave travelling through the reach, the sampling time Δt is to be smaller than the flow time K.

$$\Delta t \le K \,;\; 1 \le \frac{K}{\Delta t} \qquad \text{…Requirement (1)}$$

Further, the coefficients C_X and C_Y are required to be positive for a positive contribution of the inflow and the stored volume to the outflow.

$$C_X \ge 0; C_Y \ge 0 \qquad \text{…Requirement (2)}$$

Derived from these requirements the ratio of $K/\Delta t$ is to be in the range of

$$1 \le \frac{K}{\Delta t} \le \frac{1}{2X}$$

for obtaining numerical stability and avoiding negative values in the coefficients (see Figure 27).

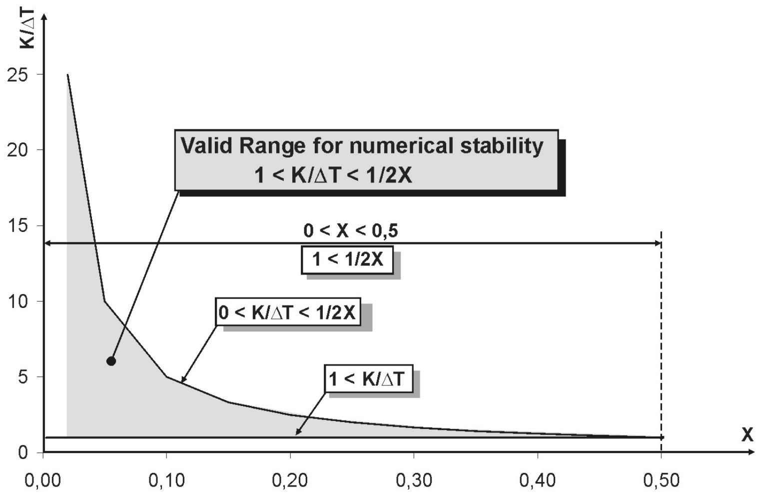


Figure 27: Valid range for numerical stability in the simplified discrete Muskingum scheme.

Consequently the range for *X* – values is defined as well from this equation, where *X* is in the range of *0 < X < 0.5*. Only for this range the above stated relation of

$$1 \le .... \le \frac{1}{2X}$$

holds true. For dealing with *n* subreaches, the formulas are rearranged and the nomenclature is taken as shown earlier in Figure 13. The overall wave travelling time *K* is substituted by *K'* being the travelling time for each subreach ($K' = K/n$). The outflow of the reach *j* is:

$$Q_i^{j+1} = C_X \cdot Q_i^j + C_Y \cdot V_{i-1}^j$$

The storage volume at time t_i in reach *j* is

$$V_i^j = \left(Q_i^j - Q_i^{j+1}\right) \cdot \Delta t + V_{i-1}^j$$

So far, the model only considers flow entering the most upstream compartment. This is feasible, as long as the method is used for only transport pipes without any lateral flow. For applications such as river or catchment routing, flow entering along the pathways is necessary. This is the case for dry weather flow that is generated all over the catchment but also for rainfall that is spatially distributed. A general scheme including compartment-wise inflows is illustrated in Figure 28.

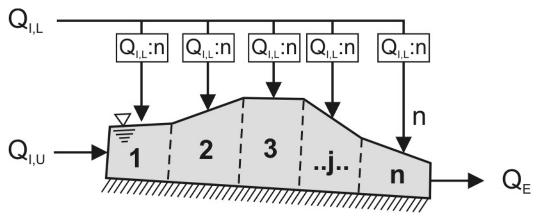


Figure 28: Muskingum routing with multiple sub-reaches having upstream and compartment-wise inflows.

The total flow introduced "locally" $Q_{I,L}$ is – according to the area and subreaches - evenly distributed over *n* sub-reaches ($Q_{I,L}$ / *n*). The formulas do not have to be changed as the total local inflow comprises both, the upstream flow and the locally generated flow.

$$Q_i^{j+1} = Q_i^j \cdot C_X + V_{i-1}^j \cdot C_Y = \left(Q_i^j + \frac{Q_{I,L}}{n} \right) \cdot C_X + V_{i-1}^j \cdot C_Y$$

4.3.5 CATCHMENT BLOCKS

Two catchment models are available for modelling either a combined or a separate sewer system (CSS or SSS). The associated dynamic inputs can be distinguished for inputs that (a) originate from the catchment and (b) inputs which originate from upstream and are to be routed through the catchment. For flow routing the simplified Muskingum scheme is used as explained earlier. The blocks and their underlying sub-models are shown in Figure 29 and Figure 30.

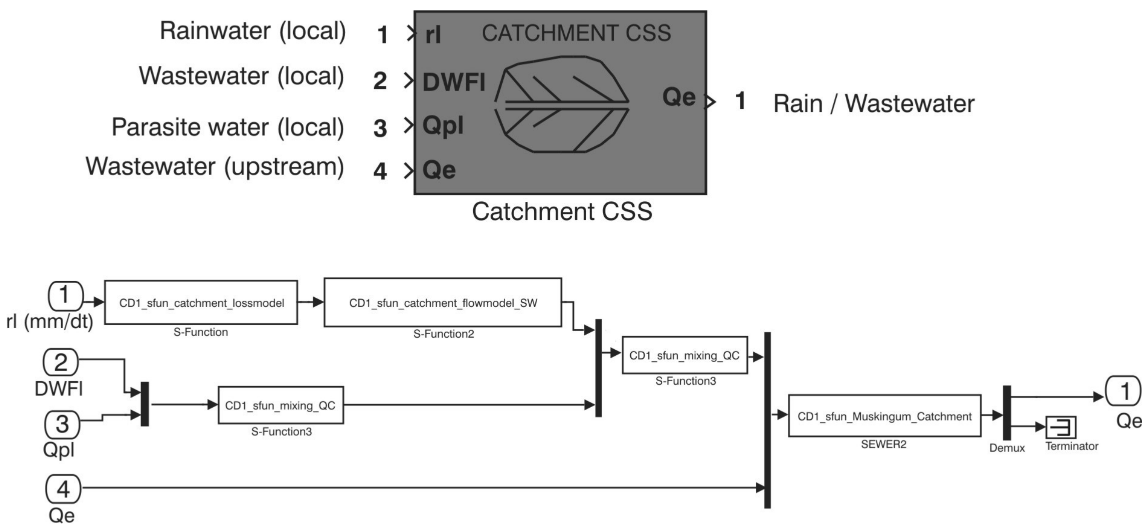


Figure 29: Screen shot: Catchment block for CSS and underlying submodels

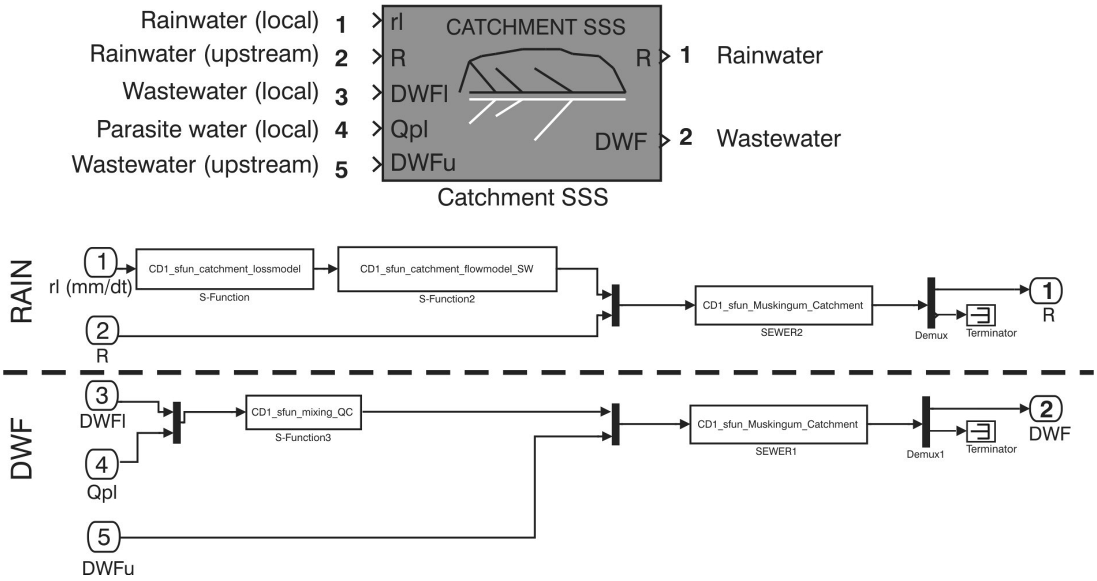


Figure 30: Screen shot: Catchment block for SSS and underlying submodels

The original Muskingum scheme allowed feeding the uppermost block only. The modified scheme allows both, feeding of the uppermost block ($Q_{I,U}$) as well a distributed feeding of blocks ($Q_{I,L}$). Thus, inputs provided such as the rain intensities (*rl*) acting on the catchment, the dry weather flows generated in the catchment (DWF_L) and parasite water infiltrating into the sewer system (*Qpl*) are distributed homogeneously within the catchment. Flows from upstream of the catchment are all the way routed through and thus are fed to the uppermost sub-block. In case of the CSS block an upstream wastewater stream *Qe* may be provided as dynamic input. For the SSS block two ports allow the dynamic inputs to the storm and waste-water sewer (*R* and *DWFu* respectively).

4.3.6 CSO - SIMPLIFIED NUMERICAL DISCRETE MODEL

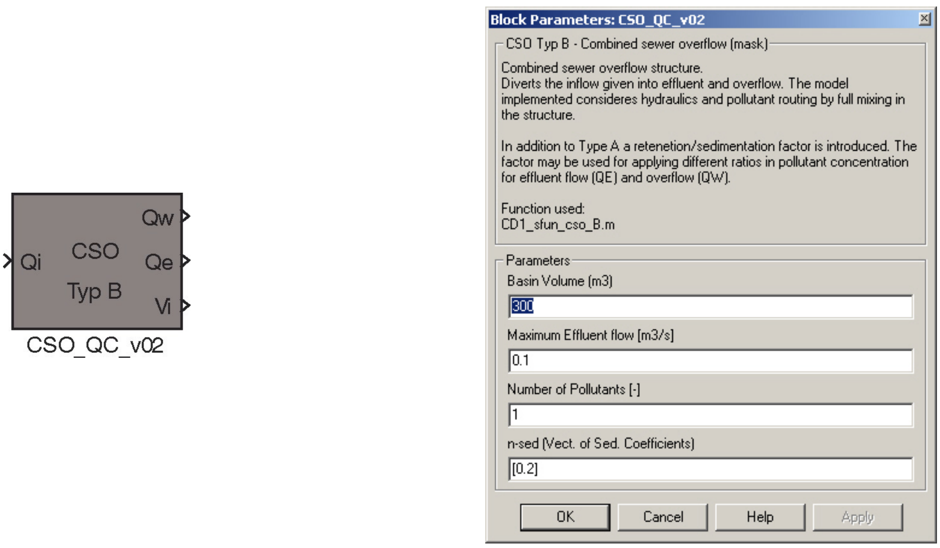


Figure 31: (a) CSO - block and (b) parameter mask

The flows entering the CSO ($Q_{I,i}$) and leaving the CSO as excess flow ($Q_{E,i}$) and overflow ($Q_{W,i}$) are balanced against the volume change. Additional inputs other than the (dynamic) inflow are the CSO parameters basin volume (V_{MAX}), maximum effluent flow ($Q_{E,MAX}$), the number of pollutants (n_{COMP}) used and the sedimentation coefficients (η). Excess flow is either diverted to a wastewater treatment plant or a downstream sewer system. The number of pollutants transported can be freely chosen by the user.

4.3.6.1 Hydraulics

The basic differential equation of the hydraulic mass balance for the CSO structure is written as

$$\frac{\partial V}{\partial t} = Q_I(t) - Q_E(t) - Q_W(t)$$

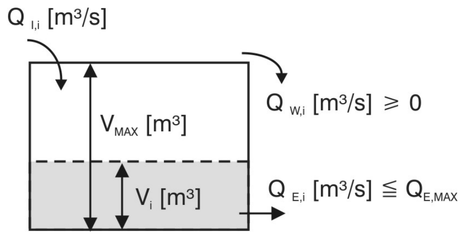


Figure 32: Variable definitions of discrete CSO model

The flows Q_I , Q_E and Q_W are considered as mean flows occurring during the discrete timely period. Herein the volume is related to discrete points of time. The mass balance equation therefore is

$$\frac{V_i - V_{i-1}}{\Delta t} = Q_{I,i} - Q_{E,i} - Q_{W,i}$$

where i denotes the time step considered. Restrictions are usually given by the maximum Volume of the CSO structure (V_{MAX}) and the maximum outflow from the structure $Q_{E,MAX}$.

Depending on the magnitude of inflow $Q_{I,i}$ and the previous Volume stored (V_{i-1}), three different cases apply. The cases can be differentiated considering the hydraulic mass balance with no overflow $Q_W = 0$ and fully developed outflow $Q_E = Q_{E,MAX}$. The virtual volume V_i' from this mass balance denotes

$$V_i' = \left(Q_{I,i} - Q_{E,MAX}\right) \cdot \Delta t + V_{i-1} \, .$$

Three cases can be distinguished:

Case No.		
1	$V_i' < 0$	$Qw = 0$ … No overflow $Q_E < Q_{E,MAX}$…Outflow Q_E not fully developed
2	$V_i' > V_{MAX}$	$Q_W > 0$…Overflow Q_W developed $Q_E = Q_{E,MAX}$…Outflow Q_E fully developed
3	$0 < V_i' < V_{MAX}$	$Qw = 0$ …No overflow $Q_E = Q_{E,MAX}$…Outflow Q_E fully developed

In case V_i' falls below zero (case 1), the maximum outflow $Q_{E,MAX}$ from CSO structure did not develop throughout the whole time and the structure falls dry. In order to keep the hydraulic mass balance, the outflow rate is adjusted (reduced).
In case of a positive V_i' (cases 2 and 3) the maximum flow rate $Q_{E,MAX}$ develops. An overflow occurs when having a virtual volume large than the maximum ($V_i' > V_{MAX}$).

For calculating the unknowns V_i , $Q_{I,i}$ and $Q_{W,i}$ the following equations apply for the respective case:

Case 1

$$V_i = 0$$

$$Q_{E,i} = \frac{V_{i-1}}{\Delta t} + Q_{I,i} \leq Q_{E,MAX} ; \quad Q_{W,i} = 0$$

Case 2

$$V_i = V_{MAX}$$

$$Q_{E,i} = Q_{E,MAX} ; \quad Q_{W,i} = Q_{I,i} - Q_{E,MAX} - \frac{V_{MAX} - V_{i-1}}{\Delta t}$$

Case 3

$$V_i = \left(Q_{I,i} - Q_{E,MAX}\right) \cdot \Delta t + V_{i-1}$$

$$Q_{E,i} = Q_{E,MAX} ; \quad Q_{W,i} = 0$$

4.3.6.2 Mixing and Settling

The here presented scheme builds on the simplified hydraulic scheme for CSO storage. Flows are taken into account that represent mean flows occurring during a time step Δt. Similar, the concentrations entering or leaving the chamber are mean concentrations over a time step.

Flow entering the basin is assumed to be fully mixed with the volume in the structure. The new concentrations being present in the chamber are then taken for the effluent and overflow concentrations. This is done for (a) simplification of calculations and (b) in order to avoid numerical shortcomings (e.g. negative concentrations).

Hydraulics

$Q_{I,i}$	Inflow for timestep i
V_{i-1}	Volume stored for the previous time step
$V_{QI,i}$	Volume added to the storage by inflow $Q_{I,i}$ during time step Δt
V'_i	Volume stored for the current time step (including overflow volume)
V_i	Volume stored for the current time step
$Q_{W,i}$	Overflow for timestep i
$Q_{E,i}$	Outflow for timestep i

Substance concentrations

C_{QIi}	Concentration in the inflow
$C_{V,i-1}$	Concentration of stored volume in previous time step i-1
$C'_{V,i}$	Concentration of stored volume in current time step i related to the total volume V'_i
$C_{V,i}$	Concentration of stored volume in current time step i related to the stored volume V_i
$C_{QW,i}$	Concentration of overflow volume in current time step i
$C_{QE,i}$	Concentration of outflow volume in current time step i

Mass balancing is done by first mixing the inflow Q_I with the stored volume of the previous time step.

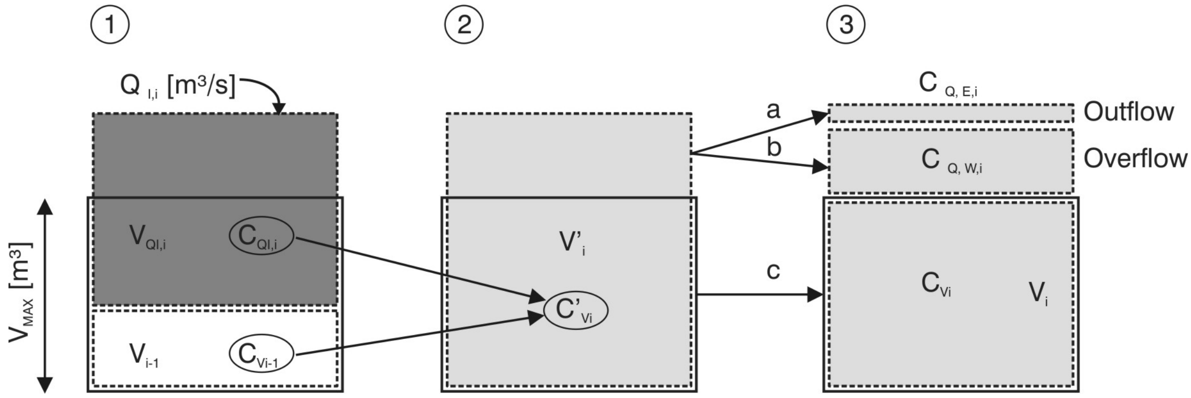


Figure 33: Scheme for flow and pollutant mass balance

$$V'_i = Q_{I,i} \cdot \Delta t + V_{i-1}$$

The concentration in this volume is

$$C'_{Vi} = \frac{C_{QI,i} \cdot Q_{I,i} \cdot \Delta t + C_{V,i-1} \cdot V_{i-1}}{V'_i} \text{ for } V'_i > 0$$

In order to account for settling of matter, the concentrations in the overflow volume are reduced using a settling coefficient η_{SED} to virtually account for the sedimentation. The concentrations present in the overflow are reduced by:

$$C_{QW,i} = C'_{V,i} \cdot (1-\eta_{SED})$$

The mass balance for the substance matter present in the volume V'_i leads to an increased concentration in the remaining volume V_i . Considering the effluent concentration $C_{QE,i}$ being the same as the chamber concentration $C_{V,i}$ leads to

$$C_{V,i} = \frac{C_{QI,i} \cdot Q_{I,i} \cdot \Delta t + C_{V,i-1} \cdot V_{i-1}}{Q_{E,i} \cdot \Delta t + V_i} \cdot \left(1 - \frac{Q_{W,i} \cdot \Delta t}{Q_{I,i} \cdot \Delta t + V_{i-1}} \cdot (1-\eta_{SED})\right)$$

$$C_{QE,i} = C_{V,i}$$

The concentration in the overflow is therefore

$$C_{QW,i} = \frac{C_{QI,i} \cdot Q_{I,i} \cdot \Delta t + C_{V,i-1} \cdot V_{i-1}}{Q_{I,i} \cdot \Delta t + V_{i-1}} \cdot (1-\eta_{SED}).$$

For numerical stability, following terms are required to be > *0*:

$$Q_{I,i} \cdot \Delta t + V_{i-1} > 0 \text{ and } Q_{E,i} \cdot \Delta t + V_i > 0$$

Physical interpretation of the first term would be that no inflow occurs ($Q_{I,i} = 0$) and the basin was empty in the last time step ($V_{i-1} = 0$). Therefore, the concentration in the chamber and the effluent concentration are set to zero ($C_{E,I} = C_{V,I} = 0$). Setting the concentration to zero is done for the overflow quality as well ($C_{QW,i} = = 0$) , since hydraulically no overflow is given.

The second boundary condition fails when no outflow occurs ($Q_{E,i} = 0$) and if the basin is empty at the present time step ($V_i = 0$). Again the concentrations are set to zero ($C_{QW,i} = C_{QE,i} = C_{V,i} = 0$), due to having an empty basin at the present time step, having not outflow and overflow.

4.3.7 A DISCRETE PUMPING STATION

Following the concept of simplified discrete formulations, the numerical formulation of a pumping station is based on mean flows from and to the storage structure. The central idea is to describe a pumping station with a number of pumps, each with it's fixed pumping rate ($Q_{P,k}$) and it's set points (water levels) for turning pump either on ($h_{P,k}^{ON}$) or off ($h_{P,k}^{OFF}$). Figure 34 shows the pump block and the respective block mask.

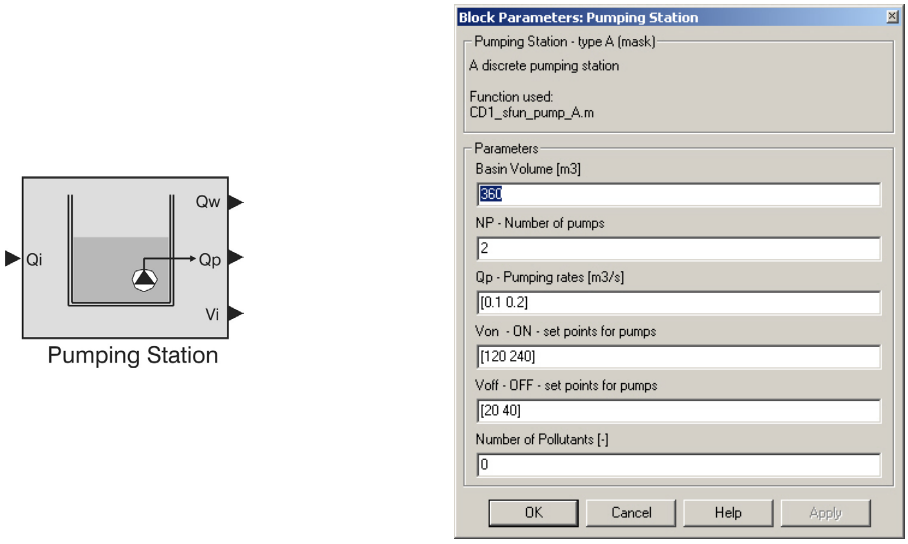


Figure 34: (a) Pumping station block and (b) block mask

The storage volume is defined by the basin volume V_{MAX} of the structure.

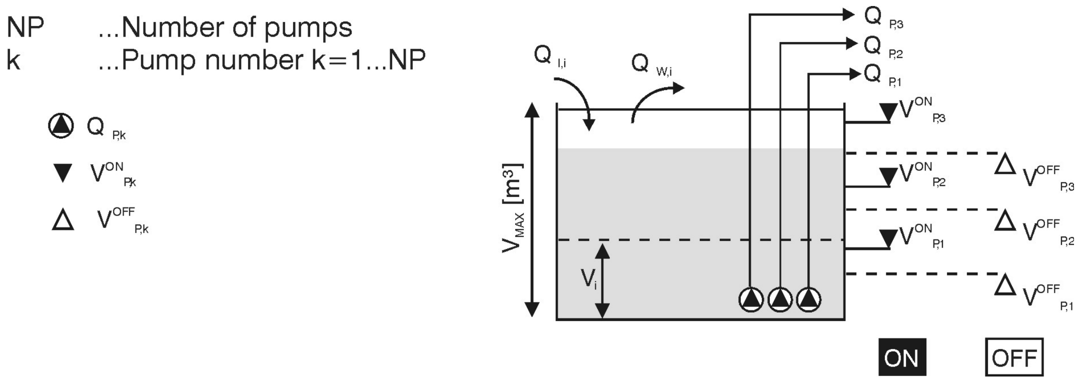


Figure 35: Schematic of pumping station with multiple (here 3) pumps

By definition the pumping rates are defined as

$$Q_P = [\quad Q_{P,1} \quad .... \quad Q_{P,k} \quad .... \quad Q_{P,NP} \quad]$$

Set points (water levels) are

$$h_P^{ON} = [\quad h_{P,1}^{ON} \quad .... \quad h_{P,k}^{ON} \quad .... \quad h_{P,NP}^{ON} \quad]$$

$$h_P^{OFF} = [\quad h_{P,1}^{OFF} \quad .... \quad h_{P,k}^{OFF} \quad .... \quad h_{P,NP}^{OFF} \quad]$$

Alternatively these may be written in terms of volume using the base area *A* of the structure

$$0 \le V_P^{ON} = A \cdot h_P^{ON} \le V_{MAX}$$

$$0 \le V_P^{OFF} = A \cdot h_P^{OFF} \le V_{MAX}$$

where set points are to be within the range of the structure storage capacity V_{MAX}. Requirement for the set points is that the ON points are more elevated than the corresponding OFF points.

$$h_{P,k}^{OFF} \le h_{P,k}^{ON}$$

Further the set points are to be monotonically increasing in elevation.

$$\begin{array}{ccccc} h_{P,1}^{ON} & \le & h_{P,k}^{ON} & \le & h_{P,NP}^{ON} \\ h_{P,1}^{OFF} & \le & h_{P,k}^{OFF} & \le & h_{P,NP}^{OFF} \end{array}$$

The scheme assumes the inflow and outflows as mean values over the last time step. The inflow $Q_{I,i+1}$ is further fully mixed with the structures content from the last time step V_i resulting in a virtual content V'_{i+1}.

$$V'_{i+1} = V_i + Q_{I,i+1} \cdot \Delta t$$

This virtual content represents the volume which would have been accumulated without pumping. Operation of pumps is driven by the water level (or by the stored volume respectively) where the order in which pumps are operating is according to the ON/OFF set points in increasing sequence.

Pump 1 is operating in case the virtual volume V'_{i+1} tops the ON set point of the pump.

$$V'_{i+1} = V_{i+1}^1 \ge V_{P,1}^{ON}$$

For homogeneous notation for all pumps, the virtual volume V'_{i+1} is termed V^1_{i+1} since it is compared with the set point volume of the first pump.

In case of no operation the pumping rate for the considered time step ($Q'_{P,1}$) is set to zero.

$$Q'_{P,1} = 0$$

In case the pump is operating it must be checked whether the volume available is sufficient for operating the pump throughout the whole time span Δt. Secondly, the remaining volume must be larger than the OFF set point of the pump ($V^{OFF}_{P,1}$).

$$V_{i+1}^1 - Q_{P,1} \cdot \Delta t \ge V_{P,1}^{OFF} \ge 0$$

Since the OFF set point is required to be greater than zero, both requirements are fulfilled simultaneously. For the case that sufficient volume is available for full operation, the pumping rate for the considered time step is set to the given pumping rate.

$$Q'_{P,1} = Q_{P,1}$$

If operation is not possible for the full period Δt, the pumping rate $Q'_{P,1}$ is reduce instead of partial operation with the original pumping rate $Q_{P,1}$.

$$Q'_{P,1} = \left(V_{i+1}^{1} - V_{P,1}^{OFF}\right) / \Delta t$$

This does not reflect the real operation but is necessary for fitting in the discrete time scheme (time steps Δt) used. Finally the mass balance, respectively the volume withdrawn (V^1_{i+1} - $V^{OFF}_{P,1}$) remains the same.

This procedure is redone for each of the remaining pumps starting at a reduced volume.

$$V_{i+1}^{2} = V_{i+1}^{1} - Q'_{P,1} \cdot \Delta t$$

The procedure using a generalized formulation for processing all available pumps is shown in Figure 36. Therein the procedure is looped for the number of pumps *(NP)* used.

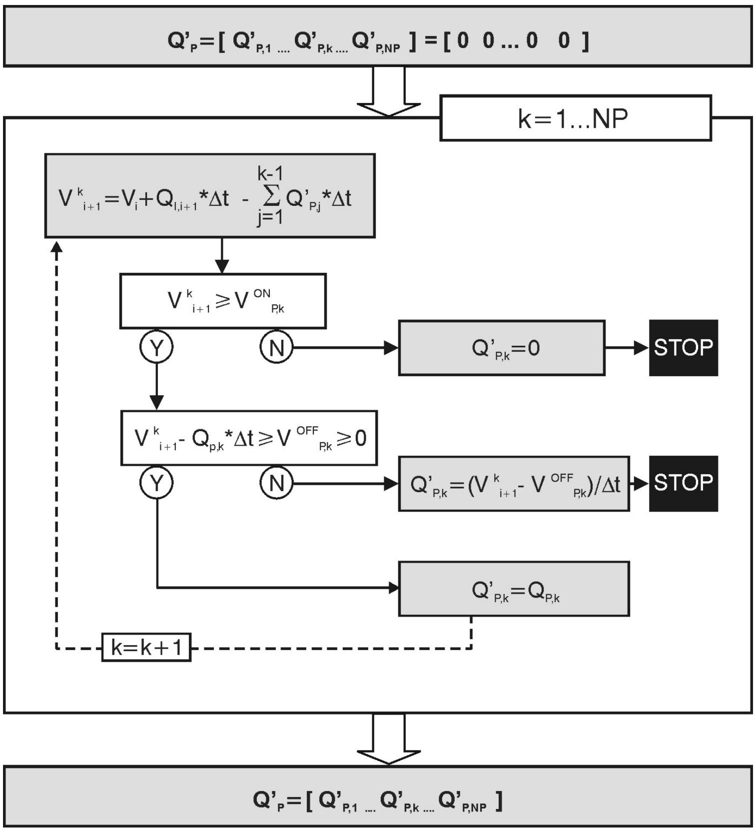


Figure 36: Numerical scheme for a pumping station based on discrete formulation

The resulting vector of pumping rates defines the total volume withdrawn

$$Q'_P = [\quad Q'_{P,1} \quad \cdots \quad Q'_{P,k} \quad \cdots \quad Q'_{P,NP} \quad]$$

$$V'_P = \sum_{k=1}^{NP} Q'_{P,k} \cdot \Delta t$$

The remaining volume in the storage tank denotes therefore as

$$V_{i+1} = V'_{i+1} - V'_P = V_i + Q_{I,i+1} \cdot \Delta t - V'_P$$

Still this volume possibly exceeds the storage volume V_{MAX}. of the structure.

$$V_{i+1} \geq V_{MAX}$$

If this is the case, the exceeding volume is withdrawn via an overflow weir.

$$Q_W = (V_{i+1} - V_{MAX}) / \Delta t$$

Consequently the remaining volume in the tank equals the maximum storage capacity.

$$V_{i+1} = V_{MAX}$$

Figure 37 shows an example for two pumps operating in parallel. No overflow is generated in the presented example.

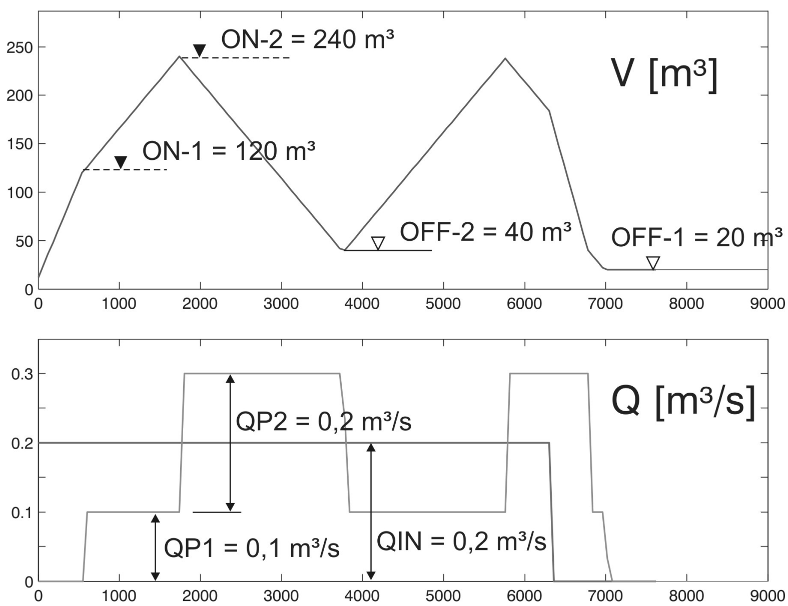


Figure 37: Example pumping period with 2 pumps operating; (top) volume stored and (bottom) in- and outflows from the pumping station

4.3.8 WWTP – WASTEWATER TREATMENT PLANT

Currently an accurate process description incorporating an ASM 1 type process model is developed. Continuous simulation of wastewater treatment plant behaviour under both dry and wet weather conditions, is based on the assumptions of the IAWQ Activated sludge model (Henze *et al.*, 1987). Further various other sub-models are included that allow simulating a complete treatment plant. The implementation follows closely the model/software denoted RUMBA. The software was developed by Rauch (1997) and is used as sub-element for other works (Harremoës and Rauch, 1996; Rauch and Harremoës, 1996, 1997). The used model is based the assumptions of the IAWQ model of Henze *et al.* (1987). Main features of the implemented model are described in the following.

The basic concept implemented is that of a recirculation plant with a primary clarifier in front followed by 2 biological reactors and a final clarifier. The oxygen set points in the two biological reactors can be determined in the setup program, so nitrification - denitrification schemes can be simulated easily. Note that the O_2 set points are not dynamic values but fixed for the simulation. Furthermore there is both an internal recycle and a recycle from the settler to the first reactor. The effluent from the primary clarifier can be directed to both reactors, i.e. the first one can be partially bypassed to model step feeding.

The following 4 quality descriptors characterize the influent to the treatment plant:

- COD soluble
- COD particulate
- total Nitrogen
- total Phosphorus

These quality descriptors are defined in the sewer system model. However, the values must be in the usual range for municipal wastewater and rain runoff. Table 7 provides an overview on the conversion parameters (including default values) used to transfer inflow quality descriptors to ASM fractions.

Table 7: Conversion model parameters

Symbol	Description	Default value	Sum
fs_{SI}	Fraction of S_I in CODsol	0.125	1.0
fs_{SS}	Fraction of S_S in CODsol	0.375	
fs_{XS}	Fraction of X_S in CODsol	0.500	
fp_{XS}	Fraction of X_S in CODpar	0.420	1.0
fp_{XH}	Fraction of X_H in CODpar	0.330	
fp_{XI}	Fraction of X_I in CODpar	0.250	
i_X	Fraction of N in org. matter	0.070	
i_{Xi}	Fraction of N in inert m.	0.020	
i_P	Fraction of P in X	0.01	

In the conversion model the COD components are determined more or less directly from the organic matter fraction parameters *f*. The amount of X_A in the influent is set to zero.

4.3.8.1 Primary clarifier model

Primary clarifiers are widely used in activated sludge systems for the purpose of sedimentation and removal of suspended particulate matter in the wastewater.
The model implemented here assumes the primary clarifier to be a fully mixed tank without any conversion processes taking place but with settling and removal of particulate components. The “settling and removal model” implemented is based on an empirical relationship for the removal of particulate components as a function of the hydraulic retention time (Rauch and Harremoës, 1996; Schilling and Hartwig, 1988). The fraction of particulate matter that is not settled (X'_{IN}) is written as

$$X'_{IN} = X_{IN} \cdot (1-\eta)$$

with the removal efficiency η for particulate matter written as

$$X'_{IN} = \eta_{\max} - (\eta_{\max} - \eta_{\min}) \cdot e^{-\eta_c \cdot V / Q_{IN}}$$

with

η	…Removal efficiency for particulate matter [-]
η_{max}	…maximum η
η_{min}	…minimum η
η_C	…Coefficient for taking the hydraulic retention time into account [T^{-1}],[1/s]
V	…Primary clarifier volume [m^3]
Q_{IN}	…plant influent [m^3/s]
X_{IN}	…particulate matter in the plant inflow [g/m^3]
X'_{IN}	…reduced particulate matter after settling (non-settled fraction) [g/m^3]

The removed amount of suspended solids is calculated directly as a fraction of the inflow. This method for describing the settling process has the advantage to take the limited amount of settable matter into account directly.

4.3.8.2 Biokinetic model

During the last three decades a large number of models of the biological processes in activated sludge treatment plants have been developed. A modified version of the IAWQ model No.1 is used for modelling biodegradation of organic material as well as biological nitrogen removal. The components implemented are shown in Table 8.

Table 8: Components used in the WWTP's biokinetic model

1	T	Temperature	Celsius
2	S_I	Inert soluble organic matter	$gCOD/m^3$
3	S_S	readily biodegradable organic matter	$gCOD/m^3$
4	S_{NO}	nitrate (NO_3^-)	gN/m^3
5	S_{NH}	ammonia ($NH_4^+ + NH_3$)	gN/m^3
6	S_P	Inorganic soluble phosphorus (PO_4)	gP/m^3
7	X_H	heterotrophic biomass	$gCOD/m^3$
8	X_A	autotrophic biomass	$gCOD/m^3$
9	X_S	slowly biodegradable organic matter	$gCOD/m^3$
10	X_I	Inert particulate organic matter	$gCOD/m^3$

Dissolved oxygen not introduced as component

One of the main differences to the full ASM1 model is that dissolved oxygen in the water phase is not treated as a component (i.e. state variable of the model) but as a boundary condition. The dissolved oxygen concentration S_O of both biological reactors is therefore specified in the plant outline and is not assumed to change during the simulation. Still, the given S_O values influence the process rates thus allowing to distinguish between aerobic and anaerobic conditions. Stoichiometry and process rates implemented are shown in Table 9 and Table 10 respectively.

Table 9: Stoichiometry

		1	2	3	4	5	6	7	8	9	10
	Process	T	S_I	S_S	S_{NO}	S_{NH}	S_P	X_H	X_A	X_S	X_I
1	Aerobic het. growth			$-1/Y_H$		$-i_x$	$-i_P$	1			
2	Anoxic het. growth			$-1/Y_H$	$-(1-Y_H)/2.86/Y_H$	$-i_x$	$-i_P$	1			
3	Aerobic aut. growth				$1/Y_A$	$-1/Y_A - i_x$	$-i_P$		1		
4	Decay het.							-1		1-fp	fp
5	Decay aut.								-1	1-fp	fp
6	Hydrolysis			1		i_x	i_P			-1	
7	P. precipitation						-1				

Table 10: Process rates

	Process	Process rates
1	Aerobic het. growth	$\mu_H * S_S/(K_S+S_S) * S_O/(K_{OH}+S_O)* S_{NH}/(0.01+S_{NH}) * S_P/(0.01+S_P)* X_H$
2	Anoxic het. growth	$\mu_H * S_S/(K_S+S_S)* K_{OH} /(K_{OH}+S_O) * S_{NO}/(K_{NO}+S_{NO})$ $* S_{NH}/(0.01+S_{NH}) * S_P/(0.01+S_P)*\eta g*X_H$
3	Aerobic aut. growth	$\mu_A * S_{NH}/(K_{NH}+S_{NH}) * S_O/(K_{OA}+S_O) * S_P/(0.01+S_P) * X_A$
4	Decay het.	$b_H * X_H$
5	Decay aut.	$b_A * X_A$
6	Hydrolysis	$kh * X_S/X_H/(K_X+X_S/X_H) * X_H$
7	P. precipitation	k_P*S_P

Main differences to the original ASM1 are:

Switching functions of Monod type growth processes
All growth processes (of Monod type) contain switching functions for nitrogen and phosphorus. That is, in the absence of those the growth processes are stopped. This is in accordance with ASM3. The *K* values of the Monod type switching functions are set to 0.01 directly in the software code and not subject to variation.

Ammonification process
The ammonification process is neglected here and all soluble organic ammonia (nitrogen fraction in S_S) is assumed to be directly converted to S_{NH}. Due to the coupling of nitrogen fractions to the organic matter the hydrolysis of particulate organic nitrogen (X_{ND} in ASM1) is modelled indirectly in the hydrolysis process of X_S.

Hydrolysis
The hydrolysis process rate was taken as is from ASM3 (Henze *et al.*, 2000).

Phosphorus precipitation
A very simple phosphorus precipitation process was included. The rate expression is a 1.order removal function without any effect by other components. This should reflect in a crude manner the precipitation process. Biological *P* removal is not included.

4.3.8.3 Secondary clarifier

Covering the relevant physical processes in the secondary clarifier under unsteady flow conditions is an essential part of a WTP-model when used in integrated urban drainage simulations. A one-dimensional settler model is implemented based on solid flux theory. The model background is described in (Takacs et al., 1991) and with respect to details the reader is referred to this publication.

4.3.8.4 Waste sludge removal

Sewage treatment plants convert a substantial part of the influent waste into bacterial biomass that has to be removed subsequently in order to keep an appropriate concentration level in the reactors. The implementation of waste sludge removal is based on the concept of imposing a certain sludge age as described in ASM 1 (Henze *et al.*, 1987).

4.4 CONCLUSIONS

In this chapter, the modelling approaches implemented in CITY DRAIN were presented. For more details the reader is referred to the corresponding publication and the user manual attached as Appendix B and E:

Appendix B

> **Achleitner S.**, Möderl M. and Rauch W. (2006).
> CITY DRAIN © - an open source approach for simulation of integrated urban drainage systems. *Environmental Modelling & Software*, (accepted)

Appendix F

> **Achleitner S.** and Rauch W. (2005).
> *CITY DRAIN © - an open source integrated simulation of urban drainage systems - User Manual*. Institute of Environmental Engineering, University of Innsbruck, Austria.

As CITY DRAIN is realized within the Matlab/Simulink environment, the basic concepts of state-space modelling were introduced. The concept of state space modelling was formulated by Kalman (1960) and deals with the determination of a system's output (y) based on its input (u) and state (x). This concept is utilized for urban drainage models allowing a block-wise description of the urban drainage system.

CITY DRAIN and the implemented library blocks use a discrete time scheme. Constant and discrete time steps are used throughout all blocks. A central block manages the runtime and time steps within the scheme provided by Simulink. All other blocks provided in a library represent parts of the drainage system, where the user connects them to mimic his system under study.

Utilisation of data on flow rates and associated pollutants is discussed. Special attention was put on dealing with different types of data such as grab samples or composite samples. A differentiation between the types of data is of importance, especially when interpolation of data points is required. The derivation of the respective equations for interpolations is provided enabling a correct handing including mass balances for water and substances.

As flows of water and matter in CITY DRAIN are considered as mean values over the last time step, discretisation of models is done using a different discrete scheme. The advantage is that all model formulations become simpler and gain numerical stability. The Muskingum method presented here requires only two parameters (C_X and C_Y) compared to three parameters *(C_1, C_2 and C_3)* in its original form. Additional modifications allow to deal with upstream and compartment-wise inflows. Compartment-wise inflows can be rainfall or DWF. Other implemented models presented are loss models, catchment models for combined and separate sewer system, CSO structures, pumping station and WWTP.

5 DATA ASSESSMENT AND MODELL CALIBRATION

This chapter is built around the data assessment of the case study Vils, focusing on different types and sources of data used, their assessment and assumption/simplifications included. Data assessment and calibration is shown as well in Achleitner *et al.* (2006b) which is attached as

Appendix B

Achleitner S., Möderl M. and Rauch W. (2006).
CITY DRAIN © - an open source approach for simulation of integrated urban drainage systems. *Environmental Modelling & Software*, (accepted)

The calibrated model was further used in Achleitner *et al.* (Achleitner *et al.*, 2006c) dealing with the waste design by urine separation. The work is attached to this thesis as

Appendix E

Achleitner, S., Möderl, M. and Rauch, W. (2006).
Waste design by urine separation – the development of control options for the case study Vils/Reutte. *Urban Water Journal*. (submitted).

5.1 DATA ASSESSMENT AND MODEL SETUP FOR THE CASE STUDY VILS

CITY DRAIN was applied for modelling the integrated urban drainage system of the catchment area Vils/Reutte. The model is to be used to test a variety of measures in the frame of the EU-project CD4WC (CD4WC D 7.2, 2006). The project deals with optimisation of the efficiency of urban wastewater systems based on an integrated approach including investment and operation costs (Benedetti *et al.*, 2004). The Vils/Reutte catchment was therein one out of four case studies, used to evaluate a variety of options to improve the wastewater system.

The urban catchment is located in the North-West of Tyrol at the border to Germany. Figure 38 provides an overview on the location and the catchment.

The catchment is located in a mountainous area, at a height of 800 m to 900 m above sea level. Subcatchments are distributed along two main sewer lines “Vils” and “Reutte”. The waste water treatment plant (WWTP) is located at the river Vils upstream of its confluence with the river Lech. Within the system seven pumping stations are in operation to overcome elevation differences towards the WWTP.

The total catchment comprises of 67% combined sewer systems (CSS) and 33% separate sewer systems (SSS). In contrast, the Vils area is almost exclusively comprised of CSS. The reduced (impervious) area of the catchment is A_{IMP} = *~100 ha*. Maximum flow times (t_f) of the sewer system are in the order of ~3 hours.

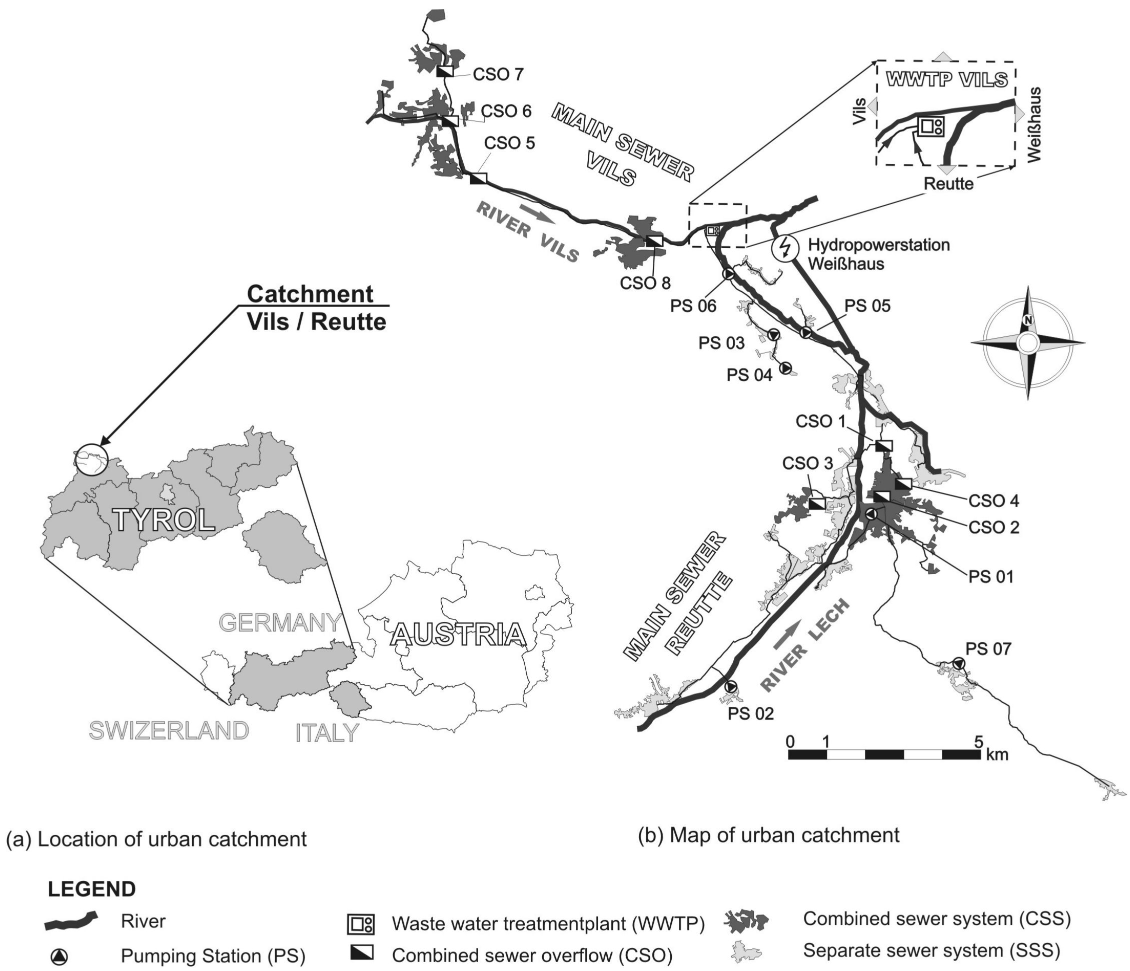


Figure 38: Overview on the urban drainage catchment Vils/Reutte: (a) location and (b) map

Inhabitants connected account for 37000 PE distributed among 27 subcatchments. Around 73 % of the total inhabitants are located in areas that discharge via CSS. Seven CSO structures with a total storage volume of *~3000 m*^{3} are installed in the system. Overflows generated during wet weather are discharged to the rivers Vils and Lech respectively.

The rivers are alpine rivers with a flow capacity of 6.6 m^3/s and 36.9 m^3/s respectively for Vils and Lech in the annual mean. For operation of the hydropower station Weisshaus, the river Vils is bypassed further downstream. The generated low flow stretch between the intake and the WWTP receives no overflows from CSS but from SSS.

For simulation of the catchment Vils Reutte with CITY DRAIN catchment data has been prepared in order to meet the data requirements of the different block compartments used. Where a detailed (pipe-wise) assessment of the sewer system is not required, a set of simple descriptors is required. Table 11 gives an overview on the required system data for the modelling.

Table 11: Data requirements for system description and hydraulic modelling

			Data type
Subcatchments (CA)			Parameter values
PE	Population equivalents	[-]	
t_f	Flow time in the catchment	[s]	
A	Subcatchment area	[ha]	
φ	Runoff coefficient	[-]	
$h_{V,I}$	Initial loss	[mm]	
$h_{V,D}$	Permanent loss	[mm/Δt]	
Transport sewers (sewer mains in between catchment)			
t_f	Flow time between catchment	[s]	
Combined sewer overflows (CSO)			
V	Basin volume	[m³]	
Q_{DR}	Maximum effluent flow	[m³/s]	
Pumping stations (PS)			
V	Basin volume	[m³]	
Q_P	Mean pumping rate	[m³/s]	
$V(h)_{P,ON}$	Level of ON set point of pumps	[m]	
$V(h)_{P,OFF}$	Level of OFF set point of pumps	[m]	
Rain data (r)			Time series
The rain data is to be provided as continuous measurement			
Flow data			
DWF	Dry weather flow hydrograph	[m³/s] and [g/m³]	
River	Flow times of the river sections	[m³/s]	

5.1.1 ASSESSMENT OF TOPOGRAPHY AND STRUCTURES

For the total 27 subcatchments the "mean" flow time within each catchment is to be estimated. In a hydrological model approach as used in CITY DRAIN the waste water originating from upper catchments and the parts generated in the catchment (dry weather flow and storm water) are routed along a common flow path. In reality, the sewer system of a catchment comprises of a main channel and a number of side branches delivering dry and storm water flow to the sewer main. Flows generated in the catchment are routed along the side branches and parts of the sewer main. In contrast, flows originating from an upstream catchment are only routed via the sewer main. Thus, when describing a sub-catchment using a single hydrological model, the flow path stated is always a compromise of different flow paths to be covered.

The approach applied in the case study Vils was as follows: Flow velocity along the sewer main (v_1) was estimated using a cross section filling of ¾ of the height. For fluxes aside the sewer main, sewer and surface flow velocities (v_2) were estimated to be 1 m/s.
The corresponding flow distances are estimated as length of the sewer main (l_1) and the maximum distance perpendicular to the sewer main (l_2).

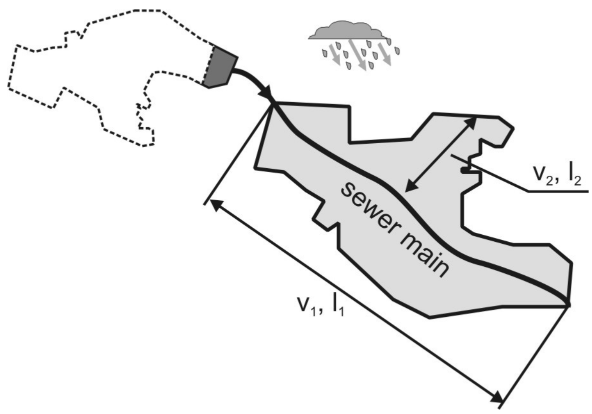


Figure 39: Flow path estimate in subcatchments.

The total flow time is calculated as

$$t_f = \frac{1}{2} \cdot \frac{l_1}{v_1} + \frac{l_2}{v_2}$$

First estimates of runoff coefficients and losses were based on engineering assumptions. In the course of calibration, these parameters were used as starting point and were corrected in the calibration. Transport sewers in between catchments that have no or negligible lateral inflow along the flow path were considered as well. The flow times were estimated with a cross section filling of ¾ of the height. For the CSO structures, data assessment was limited to storage volume and throttle flow. For the pumping stations in the system, pump characteristics and *ON / OFF* set points are to be assessed. As the pumping rates vary with the pressure conditions, fixed (mean) pumping rates Q_P were taken.

5.1.2 Rain input data

For precipitation input to the models, a long-term rain series (1998-2004) is available as cumulated volumes [mm/m^2] at 10 minute time intervals. The gauge was located in Reutte (the southern part of the catchment). No gauge was available for the Vils subcatchments. The mean annual rainfall in the area is around 1300 mm/a. The data provided by the National Meteorological Institute ZAMAG („Zentralanstalt für Meteorologie und Geodynamik“) was in UTC time scale.

5.1.3 River flow input data

The flow data was provided as flow rate [m³/s] for each of the 15 minute time steps. The available data was a continuous long-term series from 1998-2004 (UTC time scale) provided by the National Hydrologic Institute („Hydrologischer Dienst Tirol“). Two measuring stations are located in the region. The Vils gauge was located rather downstream along the river course. The Lech gauge was located half way at the modelled river course.

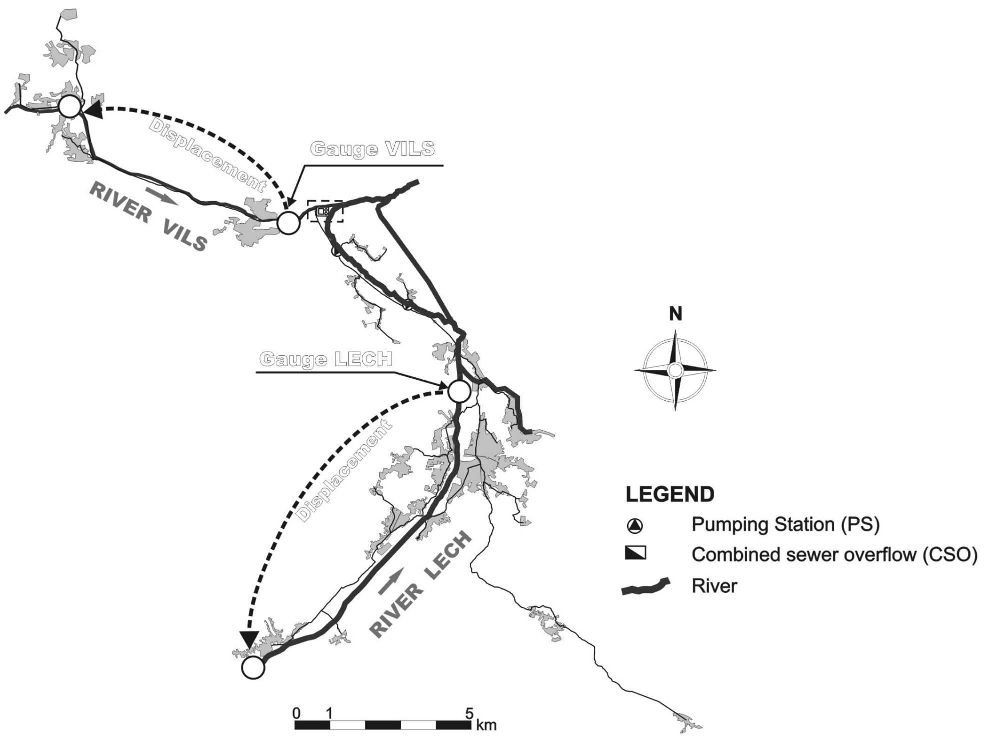


Figure 40: River gauge stations of Vils and Lech – Virtual displacement for modelling purpose

For modelling purpose, the downstream gauge measurements were displaced and used as upstream input. The temporal delay between the two points was taken into account. Comparison of downstream measured flows and modelled flows (incl. the temporal and spatial displacement) were found to be nearly identical.

5.1.4 POLLUTANT CONCENTRATIONS

The specified pollutants were chosen on basis of

- problem oriented focusing on the aim of the modelling and
- requirements of the models used themselves

NH_4 has been chosen in order to cope with the given problem of N-elimination at the WWTP. To simulate the WWTP based on the modified ASM1 model (see chapter 4.3.8) requires additionally P_{TOT} and COD_{TOT} as input variables. Distinction between particulate and soluble fractions of COD is considered at the WWTP where fractioning of inflow quality descriptors is done as described in chapter 4.3.8. Copper (*Cu*) is carried through the system as a tracer component. Thus, pollutants used in the model were P_{TOT}, Cu, COD_{TOT} and NH_4N.

The model setup is presented in Figure 41 utilizing the estimated parameters as a starting point for further calibration. Within the model, hydraulic and pollutant inputs are required for dry weather flow, storm water flow and the river. Aside inputs to the model, flows at the WWTP were taken for model calibration.

Table 12 gives an overview on the type of assessment used for the different hydraulic conditions and pollutant fluxes. For hydraulics, the measurements of the daily dynamics were available at the treatment plant for streams from Vils and Reutte.

Table 12: Overview on different assessment of pollutants fluxed in different system parts

	Dry weather flow	Storm water flow	Rivers
Flow Q [m^3/s]	calibrated	calibrated	measurements[2] 6.6 m^3/s and 36.9 m^3/s Vils and Lech
P_{TOT} [g/m^3]	calibrated[1] 3.675 mg/l	Literature values[3] 0.418 mg/l	Literature values[3] 0.103 mg/l
Cu [g/m^3]	Literature values[3] 97.5 μg/l	Literature values[3] 48 μg/l	Literature values[3] 2 μg/l
COD_{TOT} [g/m^3]	calibrated[1] 488 mg/l	Literature values[3] 81 mg/l	Literature values[3] 9,3 mg/l
NH4 [g/m^3]	calibrated[1] 29.6 mg/l	Literature values[3] 1 mg/l	measurements[4] 0.016 mg/l (VILS) - 0.020 mg/l (LECH)

[1] Calibrated for WWTP inflow using data from the 24 h measurement campaign
[2] Historical data series from a WGEV station was used as hydraulic input
[3] Literature values from Brombach and Fuchs (2003) (median values used as constants)
[4] Measurement taken in the course of the 24h measurement campaign

Dry weather flow conditions were calibrated on basis of hydraulic information provided by the treatment plant operator. For pollutant calibration, a 24h measurement campaign has been conducted. The campaign included measurements of P_{TOT}, COD_{TOT} and NH_4. Copper in the DWF has not been measured and had to be estimated by literature values.

The hydraulics during rain weather conditions was subject to calibration as well. Pollutant measurements during rain periods were not available. Thus, they were estimated as constant concentrations in the storm water according to literature. Brombach and Fuchs (2003) provide a database for concentration levels in combined and storm sewer systems. Within the data base a wide variation can be found. Such measurements are not made on a regular basis, but often problem related in the course of investigations facing a specific deficit. Thus, a wide range of pollutant levels is found. Similar findings are made by (Achleitner *et al.*, 2006a; Welker and Dittmer, 2005) who addressed heavy metals Cu, Zn, Pb and Cd in surface flow from streets and parking areas.

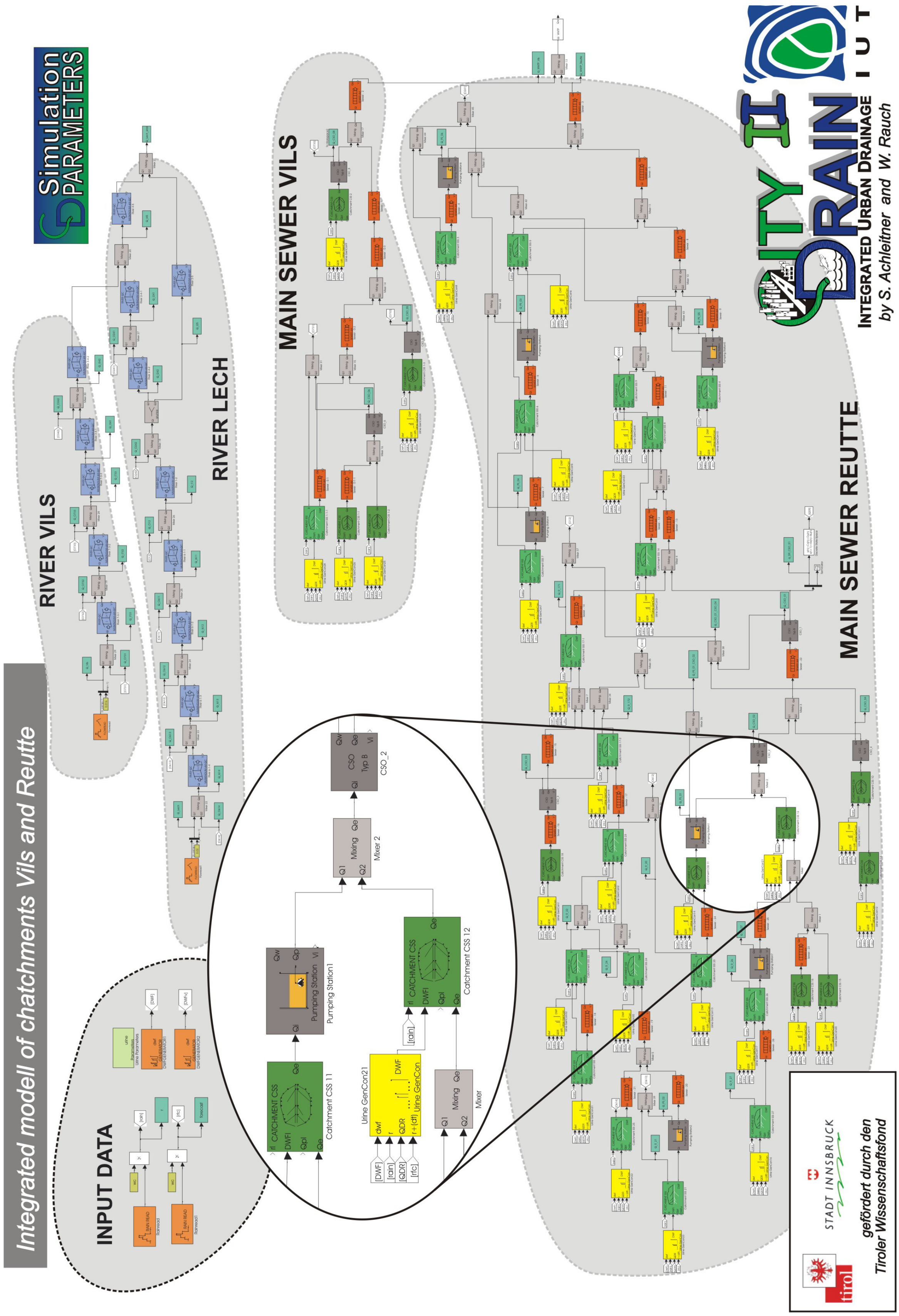


Figure 41: CITY DRAIN model of catchment Vils/Reutte

5.2 CALIBRATION PROCEDURE AND QUALITY INDICATORS

5.2.1 QUALITY INDICATORS FOR CALIBRATION

To evaluate the quality of the calibration, quality indicators are defined to compare the measured data *(M)* with the simulated data *(S).* The indicators used were taken from Grecu and Krajewski (2000).

Table 13: Quality indicators for calibration

Indicator		Mathematical equation	Range
E	Coefficient of efficiency (Nash-Sutcliffe Efficiency)	$E = 1 - \frac{\sum_{i=1}^{N}(M_i - S_i)^2}{\sum_{i=1}^{N}(M_i - \overline{M})^2}$	[-∞ , 1]
d	Index of agreement	$d = 1 - \frac{\sum_{i=1}^{N}(M_i - S_i)^2}{\sum_{i=1}^{N}(\lvert M_i - \overline{M}\rvert + \lvert S_i - \overline{M}\rvert)^2}$	[0 , 1]
B	Bias	$B = \frac{\overline{S}}{\overline{M}}$	[-∞ , +∞]
with			
	S_i	…element in the time series of simulated data	
	M_i	…element in the time series of modelled data	
	$\overline{S}$	…mean value of simulated data	
	$\overline{M}$	…mean value of modelled data	

The Nash Suttcliffe efficiency *(E)* returns values in the range of (-∞) to 1. Larger values tending towards one indicate a good fitting of the simulated data. A negative value of *E* can be interpreted such that the simulated data *(S*) matches the measured data worse than the mean value of the measured data.

The index of agreement (*d*) ranges from 0 to 1. The larger the numerical value the better is the fit of simulated data *(S)* to measured data *(M).*

Last the bias is used, being the ratio of the mean values of *S* and *M.* Although the bias as an index that is very simple it is also considered as being valuable. In the course of calibration, the bias indicates the quality in meeting the mass balance between simulated and measured data. Best fit – in terms of mass balance - is obtained when having a bias of “1”. In case of flow calibration, this would indicate that the same mean flow is obtained for simulation and measurement.

For investigating the sensitivity of the presented quality indicators, a large set of random generated time series *M(t)* was created, representing virtual measured values. To obtain time series representing simulated data *S(t),* the virtual measurement time series *M(t)* were modified using a random deviation.

$$\vec{S} = \overrightarrow{M}\left(1 + (randn(0,1) - 0.5) \cdot k\right)$$

The random deviation included positive and negative deviations from the measured time series where *k* was varied in the range from 0.1 to 2. Thus, the generated failures are characterised as white noise of different intensities around the measured data.

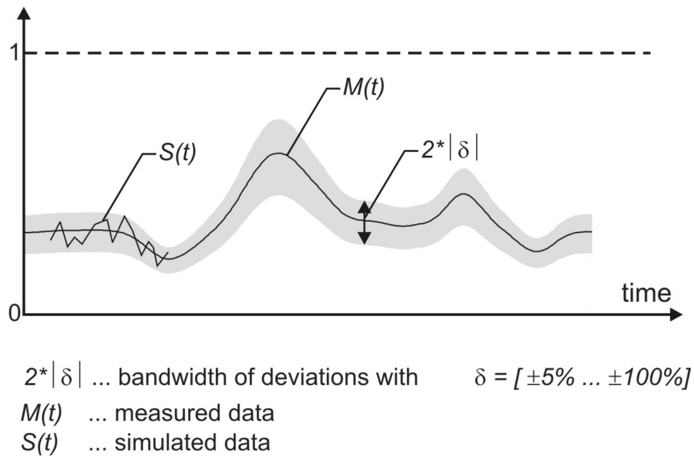
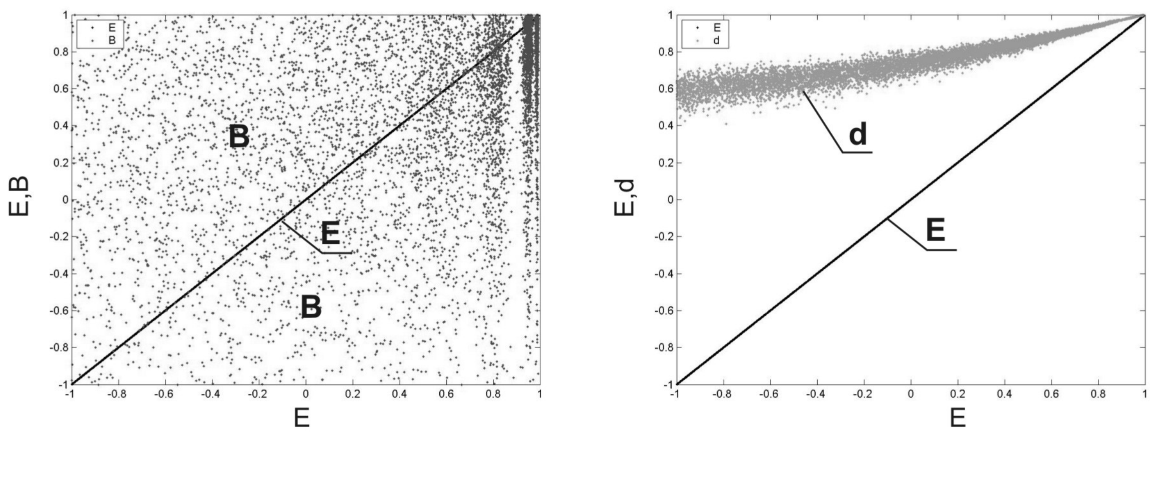


Figure 42: Schematic on stochastic generation of simulated data

Figure 42 illustrates the generation of simulated data series (*S(t)*) from measured data (*M(t)*). The generated series *S(t)* are varied within a bandwidth of 2*δ ranging from ±5% to ±100% of the measured data *M(t)*.

For each of the randomly generated data sets of *S(t)* and *M(t),* calibration indexes *E*, *d* and *B* were calculated. In order to evaluate how the three indexes correspond, plots of the parameters with the Nash Sutcliffe Efficiency *E* as basis were made.



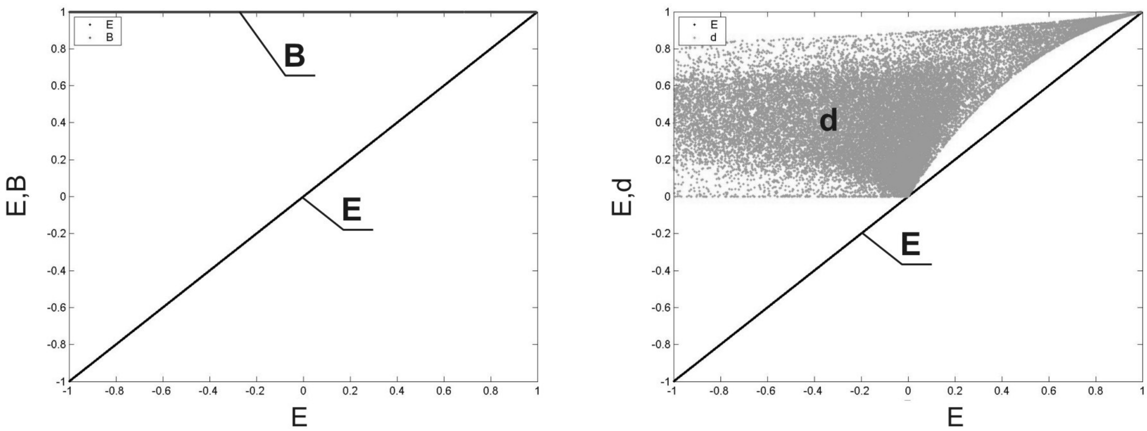
(a) (b)

Figure 43: Relation of calibration quality index E with the (a) Bias B and the (b) Index of agreement d

In Figure 43 (a) the Bias *B* is plotted against *E*. The diagonal line represents *E* plotted against itself. It is seen that the Bias *B* (that is based on the overall means of *S* and *M*), does not correlate at all to *E*. Thus, using *B* as an indicator of quality reduces to judge on whether a mass balance is given between measured and simulated data. Comparing values of *E* and d (see Figure 43 (b)) reveals that d overestimates the calibration quality compared to the index E.

In the course of calibration, the adjustment of the overall volume discharged can be made prior (and independently) from adjusting the temporal dynamics. Doing this, calibration is made such that the Bias *B* is close to "1". This of course changes the calibration quality and subsequently the remaining indicators *E* and *d*. To investigate how *E* and *d* change with the simulated data *S* being optimized for a Bias *B* equal to 1, the time series of *S* are scale with respect to the Bias *B*.

$$\overrightarrow{S_B} = \vec{S} \cdot \frac{1}{B} = \vec{S} \cdot \frac{\overline{M}}{\overline{S}}$$



(a) (b)

Figure 44: Relation of calibration quality index E with the (a) Bias B and the (b) Index of agreement d for simulation data calibrated for the annual volume

In Figure 44 (a) *E* and *B* is plotted, where all Bias are "1" as intended. The index of agreement *d* still overestimates the quality compared to *E*, but in a much wider range. For random distributed bias *B*, indexes of agreement *d* were all located in a relatively narrow plume above the diagonal (Figure 43 b). After bias optimization, *d* covers the full spectrum between 0 and 1, and thus correlates less than before (Figure 44 b).

Thus, quality indication using *E* or *d* may lead to different statements and possibly worse results after a first adjustment for e.g. the yearly discharged volume (represented by the bias *B*).

5.2.2 THE CORRELATION COEFFICIENT

The quality indicators presented so far express the quality of calibration differently or at a different intensity. Still, all of them are eligible to be used in the course of calibration. A different quality indicator is the Pearson correlation coefficient *C*, which is standard in most software applications for statistical analysis:

$$C = \frac{N\left(\sum_{i=1}^{N}(M_i \cdot S_i)\right) - \sum_{i=1}^{N}(M_i) \cdot \sum_{i=1}^{N}(S_i)}{\sqrt{\left[N \cdot \sum_{i=1}^{N} M_i^2 - \left(\sum_{i=1}^{N} M_i\right)^2\right] \cdot \left[N \cdot \sum_{i=1}^{N} S_i^2 - \left(\sum_{i=1}^{N} S_i\right)^2\right]}}$$

Performing the investigation of the index's sensitivity as done for *E*, *d* and *B* results in a similar picture as for the index of agreement *d* (Figure 45). Thus, *C* overestimates the calibration quality compared to *E*.

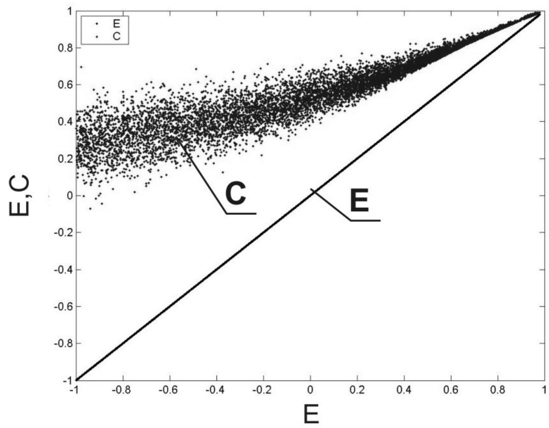


Figure 45: Relation of calibration quality index E and the corr. coeff. C for random Bias B

For a calibrated Bias *B=1*, the range of correlation coefficients expands as well (see Figure 46). Thus, on the first view, the correlation coefficient *C* seen to a quality index similar to *d*, but more sensitive.

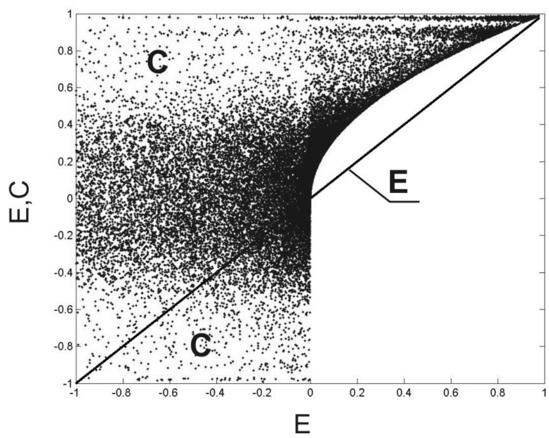
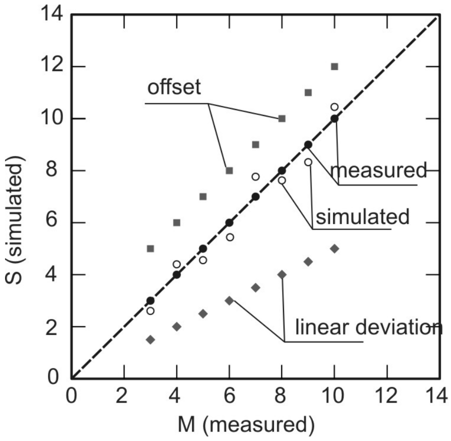


Figure 46: Relation of calibration quality index E and the corr. coeff. C for optimized Bias B=1.

In fact C is a dimensionless index, suitable to describe the correlation between a two sets of independent variables. The index returns values in the range of –1 to 1. As independent variables are required, the coefficient is not suitable to be used for correlation of measured and simulated data. This fact is illustrated using a simple example presented in Figure 47. It is shown how misleading it may be to use C as a quality indicator for calibration.



	C	E	d	
●	1.0	1.0	1.0	M ...measured data
○	0.985	0.968	0.992	M+r ...simulated data (random offset)
◆	1.0	-1.261	0.605	M*φ ...linear deviation from measured data
■	1.0	0.238	0.844	M+δ ...offset from measured data

Figure 47: Example for misleading use of the correlation coefficient C for data calibration.

Starting from a perfect simulation where simulated data match the measured for all points, measured values M have been modified. For a random deviation of data from perfection it seems that the correlation coefficient is feasible as an indicator.

Besides random discrepancies between simulation and measurement data, systematic discrepancies often occur. This can be an offset in the data or a scaling due to mismatching units for just to name two examples. Evaluating data with such a systematic error using the correlation coefficient provides a full misleading picture. For both, scaled and offset data, the correlation coefficient is one.

5.3 CALIBRATION OF THE CATCHMENT VILS/REUTTE

5.3.1.1 Calibration of DWF hydraulics

Generating the daily dynamics of flow and substance concentrations under dry weather conditions was done using a cubic spline $\delta(t)$ that was scaled to unity (with a mean value $\delta_M=1$). In absence of detailed – catchment wise – data, the same spline was used in all catchments at no temporal deviation from each other. Thus, dynamics of wastewater generation are the same in all subcatchments within the areas Vils and Reutte. Figure 48 shows the unity spline used in the catchment Reutte.

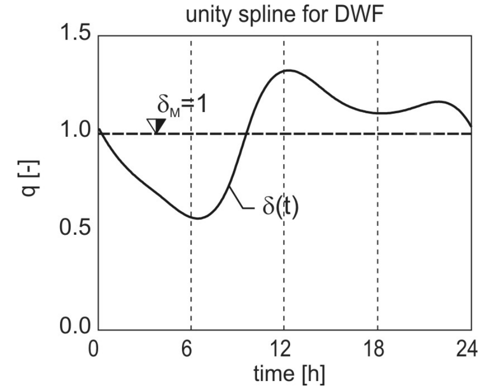


Figure 48: Unity spline for DWF generation used in the catchment Reutte

Figure 49 shows a schematic on the DWF generation as done for in the catchments Vils and Reutte. The unity spline is scaled for the mean DWF per PE of the catchment. At each of the subcatchments the relative DWF dynamics were scale for the associated PE.

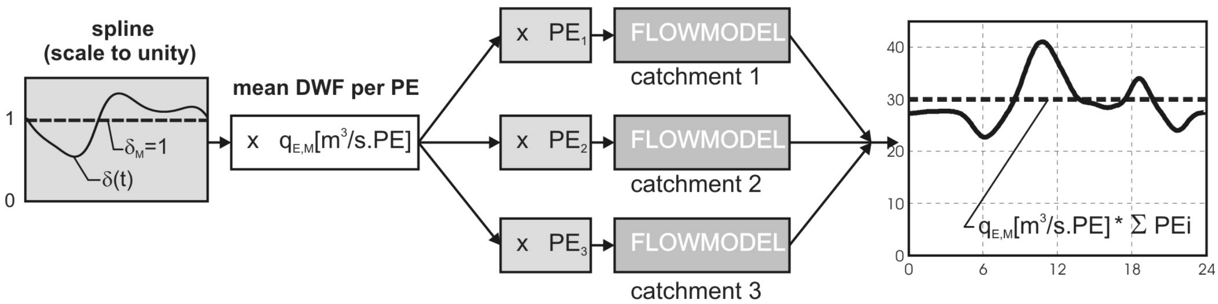


Figure 49: Schematic on DWF flow generation using a unity spline as basis

For a catchment *i*, the DWF would then be

$$Q_i(t) = \delta(t) \cdot q_{E,M} \cdot PE_i$$

with

$\delta(t)$	…unity spline [-]
$q_{iE,M}$	…mean DWF per people equivalent (*PE)* [m^3/s.PE]
PE_i	…people equivalents at catchment i
$Q_i\,(t)$	…DWF at catchment *i* [m^3/s]

After routing the flow via different catchments, the total flow is collected. The mean of the DWF at the catchment outline $Q_{E,M}$ is equal to

$$Q_{E,M} = q_{E,M} \cdot \sum PE_i \ .$$

Besides the adjustment of the discharged volume (modification of $q_{E,M}$), the shape of the spline is to be adjusted to meet the downstream dynamics. These splines were adjusted such that a best fit was obtained for dry weather conditions measured at the WWTP. The

DWF hydraulics were calibrated for Vils and Reutte separately (different unity spline and mean DWF) as hydraulic measurements were available.

A similar procedure can be applied not only to DWF hydraulics, but to concentrations associated to the DWF as well. There only measurements of the total inflow were available. Consequently only one spline for both catchments was used.

The DWF hydraulics were calibrated for measurements at the WWTP. Figure 50 shows the calibration of the DWF hydraulics for the catchments Reutte and Vils separately.

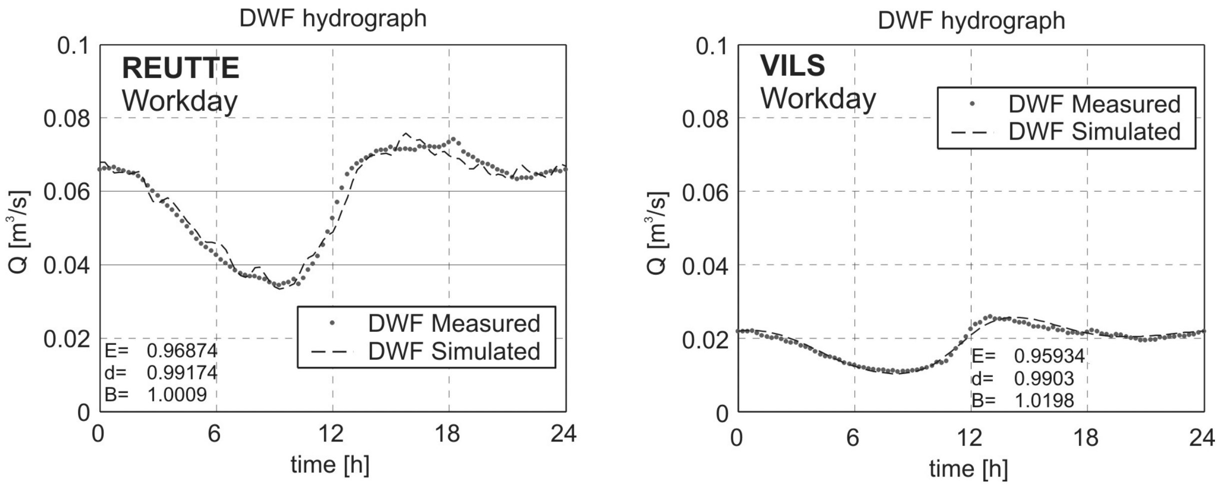


Figure 50: Dry weather inflow at WWTP – measured vs. simulated data

Evaluation of the calibration quality was made by using calibration indexes *E*, *d* and *B* introduced above.

5.3.1.2 Calibration of storm water hydraulics

Rain weather conditions were calibrated as well for hydraulic measurements made at the WWTP. If available, other measurements such as overflow events or recorded pumping events may be used as well for calibration. As such data was not available for the Vils case, the calibration was limited to measurements taken at the WWTP.

The model parameters which were modified to calibrate the system are:

- the runoff coefficient (φ) and the initial loss ($h_{v,i}$) of the catchment blocks to handle the amount of the wet weather flow hydrograph,
- the maximum effluent flow (Q_{DR}) of the CSO to handle maximum flows delivered downstream,
- and the number of subreaches of the sewer blocks to handle the flow time (t_f) of the sewer system.

The resulting calibrated parameters for the system can be found in Table 14, Table 15, Table 16 and Table 17.

Table 14: Calibrated data of transport sewers (sewer mains in between catchment)

ID	*l [m]*	*t_f [min]*	*ID*	*l [m]*	*t_f [min]*	*ID*	*l [m]*	*t_f [min]*
0-1	1200	16	8	1579	20	18	572	3
0-2	1800	35	9	281	4	19	2590	42
0-3	1400	30	10	1835	21	20	961	15
1	3169	40	11	496	10	21	630	8
2	1863	40	12	579	14	22	529	4
3	1453	20	13	385	10	23	1421	10
4	573	12	14	1038	16	24	277	5
5	1064	9	15	325	2	25	5257	87
6	323	3	16	762	8	26	4685	78
7	684	8	17	399	6			

with
l length [m]
t_f flow time [min]

Table 15: Calibrated data at combined sewer overflow (CSO) structures

	CSO-ID	V [m³]	Q_{DR} [l/s]
REUTTE	1	800	190
	2	280	325
	3	134	20
	4	270	65
VILS	5	800	80
	6	374	60
	7	0	0
	8	200	80

with
V storage volume [m^3]
Q_{DR} maximum effluent flow [m^3/s]

Table 16:Calibrated catchment data for subcatchments in Vils and Reutte

	Village	#	Catchment	Type	PE	A	t_f	φ	$h_{v,i}$	$h_{v,d}$
						[ha]	[min]	[-]	[mm]	[l/(s*ha)]
VILS	Pfronten		Pfronten		13105	211.6				
		1-1	Pfronten 1	CSS	1618	26.1	30	0.10	1	1.25
		1-1-1	Pfronten 1-1	SSS	2556	41.3	30	0.10	1	1.25
		1-2	Pfronten 1-2	CSS	4961	80.1	28	0.10	1	1.25
		1-3	Pfronten 1-3	CSS	3970	64.1	22	0.10	1	1.25
REUTTE	Vils	2	Vils	CSS	1731	94.5	25	0.10	1	1.25
	Pinswang			SSS	485	19.5				
		3	Unterpinswang		295	11.8	33	0.19	1	1.25
		4	Oberpinswang		190	7.6	20	0.20	1	1.25
	Musau			SSS	374	23.0				
		5	Musau		256	15.8	29	0.18	1	1.25
		6	Brandstatt		90	5.5	12	0.19	1	1.25
		7	Roßschläg		28	1.7	3	0.21	1	1.25
	Pflach			SSS	1277	59.4				
		8	Pflach O		781	36.3	28	0.18	1	1.25
		9	Pflach W		496	23.1	31	0.16	1	1.25
	Reutte			CSS	8289	187.5				
		10	Reutte N		3195	72.3	24	0.15	0.8	1.25
		11	Reutte SW		2959	66.9	29	0.15	0.8	1.25
		12	Reutte SO		2135	48.3	30	0.15	0.8	1.25
	Breitenwang				2728	101.1				
		13	Breitenwang	CSS	814	30.2	33	0.15	0.8	1.25
		14	Mühl	SSS	1619	60.0	40	0.16	1	1.25
		15	Lähn P	CSS	150	5.5	7	0.15	0.8	1.25
		16	Lähn L	CSS	146	5.4	4	0.15	0.8	1.25
	Lechaschau				2148	79.9				
		17-1	Nord	SSS	967	36.0	21	0.19	1	1.25
		17-2	Süd	SSS	1181	44.0	23	0.19	1	1.25
	Wängle				1394	35.2				
		18	Wängle	CSS	1355	34.2	19	0.15	0.8	1.25
		19	Holz	CSS	39	1.0	6	0.21	1	1.25
	Ehenbichl				1521	31.1				
		20	Ehenbichl	SSS	1138	23.3	20	0.18	1	1.25
		21	Rieden	SSS	383	7.8	14	0.23	1	1.25
	Höfen				1756	66.3				
		22	Höfen NW	SSS	481	18.2	9	0.16	1	1.25
		23	Höfen NO	SSS	715	27.0	28	0.16	1	1.25
		24	Höfen S	SSS	560	21.1	15	0.19	1	1.25
	Weissenbach	25	Weissenbach	SSS	1588	52.8	55	0.21	1	1.25
	Heiterwang	26	Heiterwang	SSS	707	27.6	24	0.18	1	1.25
	Bichelbach	27	Bichelbach	SSS	1264	5.0	6	0.18	1	1.3

Table 17: Calibrated data for pumping stations

	PS-ID	Pumping Station	Sewer	Q_P [m³/s]	V_{off} [m³]	V_{on} [m³]	V [m³]
REUTTE	1	Klosterweg	02	0.150			18.50
	2	Rieden	08	0.008	0.40	3.27	2.87
	3	Musau	09	0.011	0.22	0.92	0.70
	4	Roßschläg	09	0.009	0.23	0.91	0.68
	5	Oberpinswang	10-1	0.011	0.40	2.71	2.31
	6	Unterpinswang	10-2	0.011	0.40	2.85	2.45
	7	Heiterwang	12	0.0110			22.15

with

Q_P pumping rate [m^3/s]
V_{on},V_{off} ON/OFF points [m^3]
V storage volume [m^3]

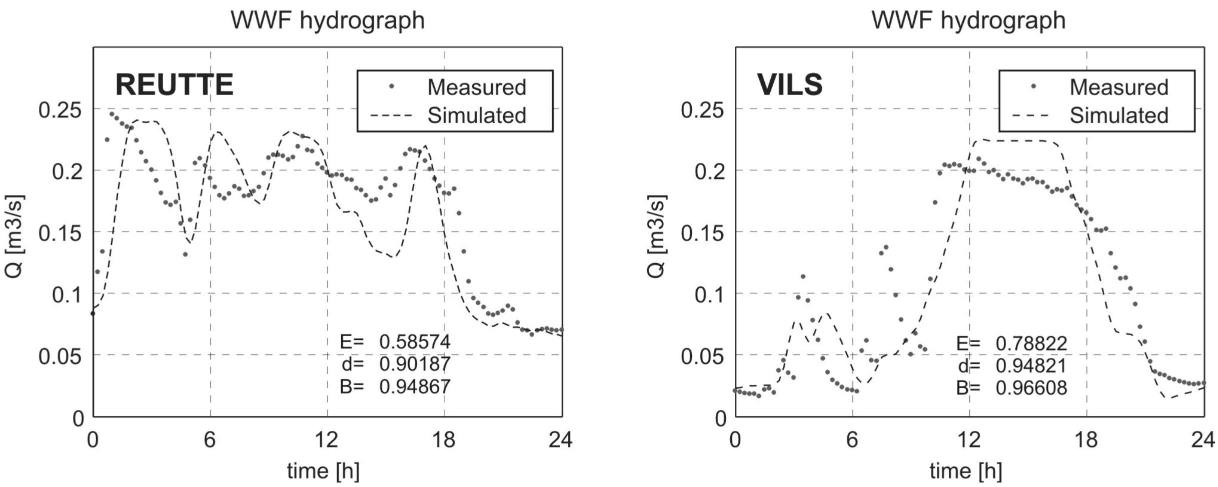


Figure 51: Two day samples for wet weather inflow at WWTP – measured vs. simulated data

For wet weather conditions, an event based evaluation was made with regard to calibration quality. Figure 51 shows a one day sample of wet weather flow in the catchments Reutte and Vils. In Figure 52 and Figure 53 measured and simulated values of event wise (a) flow maxima and (b) discharged volumes are compared.

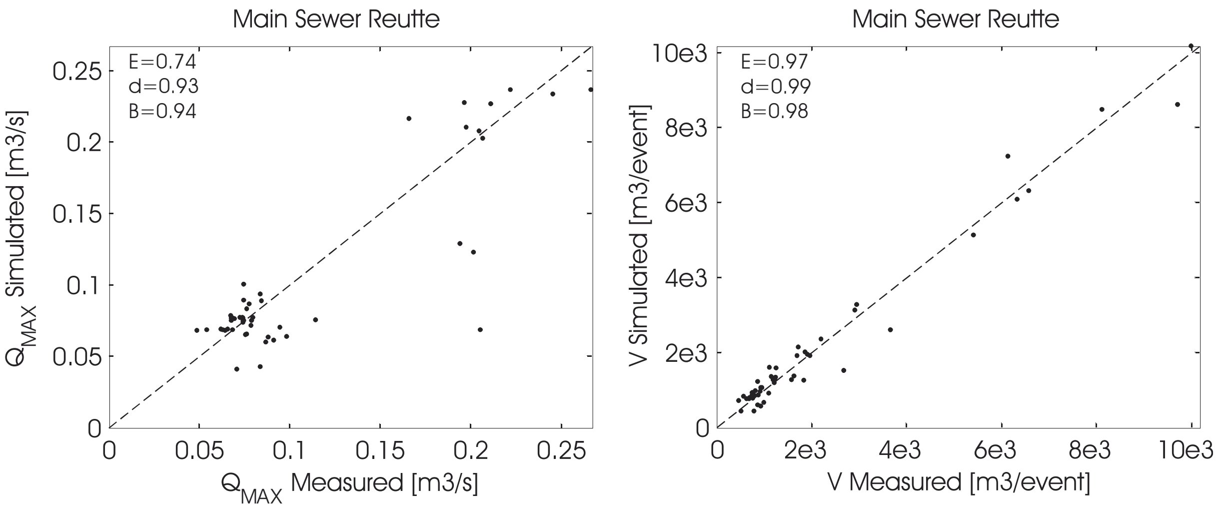


Figure 52: Measured versus simulated data of event wise (a) flows Q_{MAX} and (b) volumes for the catchment Reutte

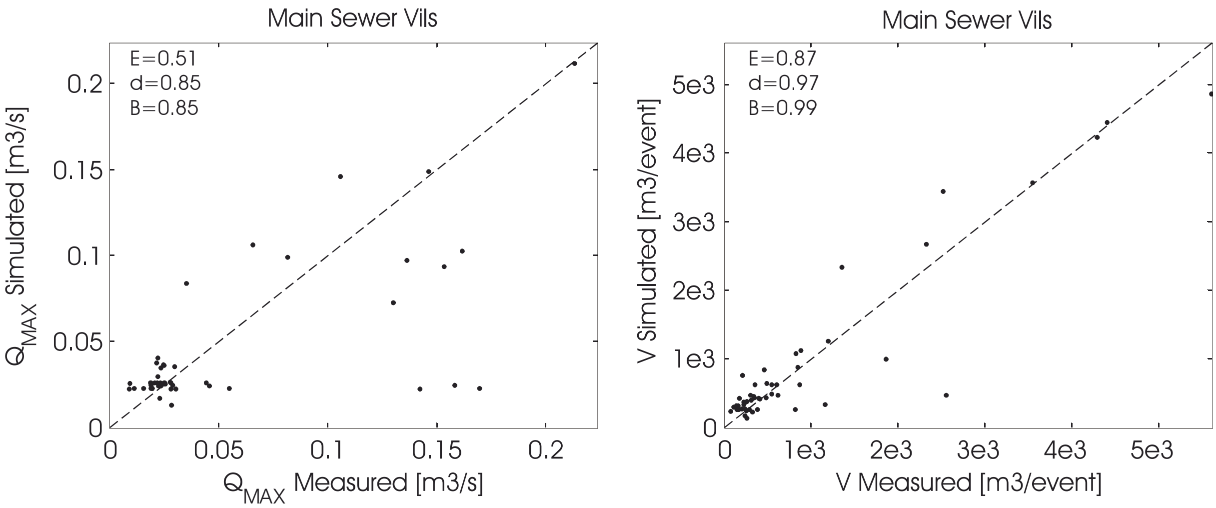


Figure 53: Measured versus simulated data of event wise (a) flows Q_{MAX} and (b) volumes for the catchment Vils

It is seen that calibration quality in the catchment Vils is less than in Reutte. This may be attributed to the single rain gauge available to be located in Reutte.

5.3.1.3 Pollutants calibration

For calibration of pollutographs in the dry weather flow, 2h composite measurements from the 24h measurement campaign were available for NH_4, P_{TOT} and COD_{TOT}. All other substances in the DWF were taken as constants according to median values found in Brombach and Fuchs (2003). Pollutants in the rain water were taken from the same source. Background concentrations in the river were taken as found in the Vils and Lech river, where mean values were used as constants. Calibrations of NH_4, P_{tot} and COD_{tot} in the DWF were made using a similar scheme as used for DWF hydraulics.

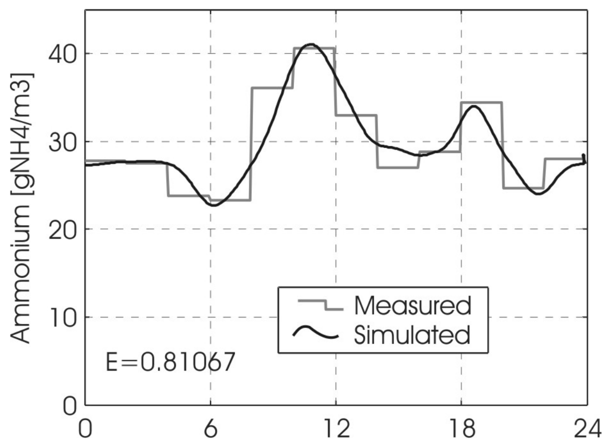


Figure 54: NH_4 concentration in DWF at the treatment plant, measured and simulated pollutograph

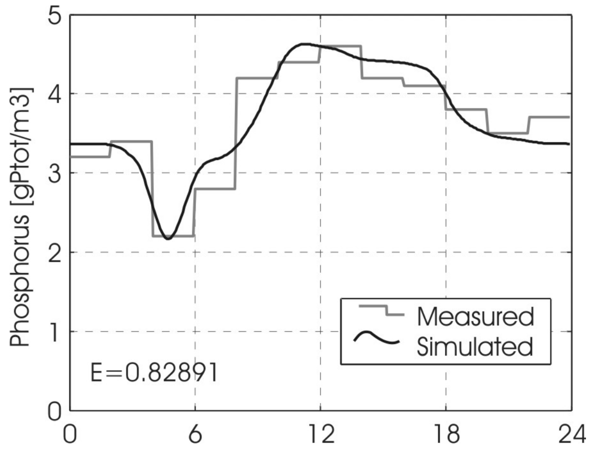


Figure 55: P_{tot} concentration in DWF at the treatment plant, measured and simulated pollutograph

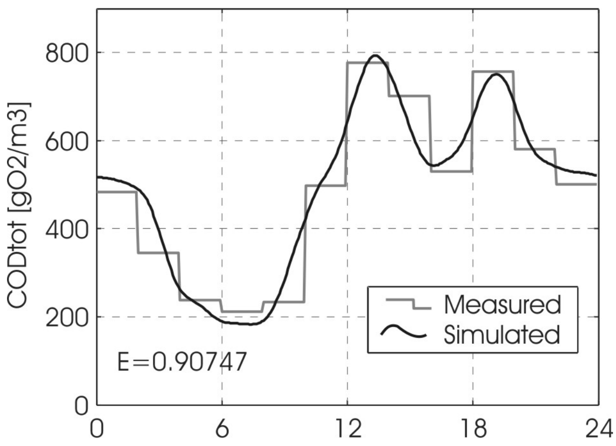


Figure 56: COD_{tot} concentration in DWF at the treatment plant, measured and simulated pollutograph

Time proportional composite samples of 2h periods were available for calibration. On the contrary, simulation results are given at a much finer temporal resolution (here 5 minute values). When evaluating the calibration quality, data sets having equal time steps are required.

As an example, NH_4 calibration quality is presented. Figure 57 (a) shows the evaluation of 2h composite samples vs. 5 minute values from the simulation. The Nash Sutcliffe Efficiency came out to be *E=0.8107*. In contrast, the simulation data were modified in Figure 57 (b), having 2h mean values as in the measurements. In the example, the numerical value of *E* representing the calibration quality increases.

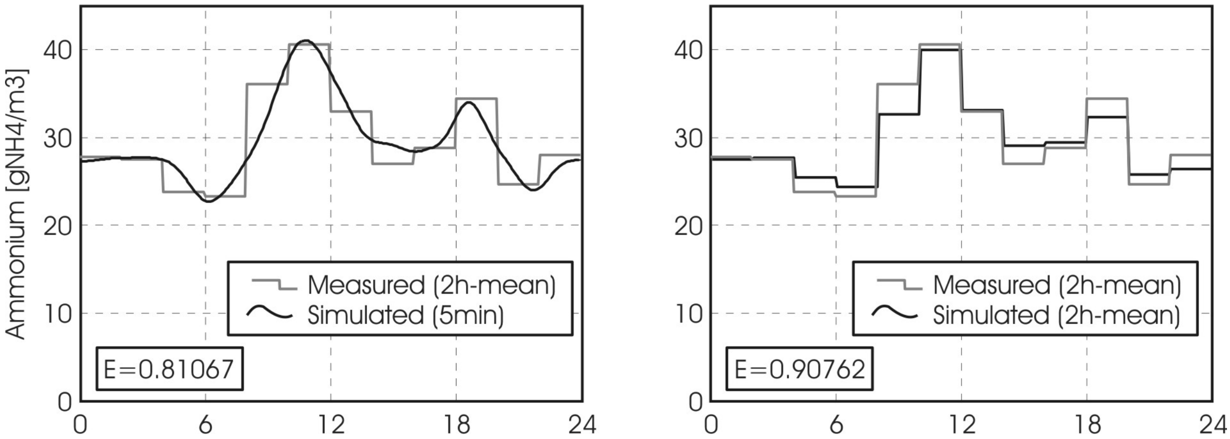


Figure 57: Example calibration qualities using different time scales for composite sample periods.

Thus, it can be seen that it makes a difference on what time scale the evaluation is made. For the example of NH_4 the improvement was ~+12%. An increase in calibration quality was also as well observed for COD_{TOT} and P_{TOT}.

Table 18: Change of the Nash Sutcliffe Efficiency depending on the time scale

	E		
	M (2h) vs. S (5min)	*M (2h) vs. S (2h)*	Δ
NH_4-N	0.8107	0.9076	+11.9%
COD_{TOT}	0.9075	0.9635	+ 6.2%
P_{TOT}	0.8290	0.8711	+ 5.1%

5.4 CALIBRATION OF THE WWTP

Calibration of the WWTP showed to be a more complex task than expected in advance. The modified ASM 1 type model used is, as all models of the ASM family, designed for temperatures ranging between approximately 10 – 20 °C. A number of bacteria the growth and decay rates implemented in the model depend on the temperature. In general, bacterial activity is decreasing with the temperature, where activity under ~8 °C is considered as negligibly low.

Temperatures faced at the WWTP Vils are considerable lower than 8 °C during winter time. The mean temperature of the WWTP outflow, which roughly represents the temperature in the aeration tank, is around 4 °C. This is attributable to (a) the inflow being of low temperature already and (b) an additional heat loss in the treatment plant during winter periods. As seen in Figure 58, air temperatures during the winter period drop frequently below zero. This causes a drop in wastewater temperature in the treatment plant during that time. Wastewater temperature drops for another 3 - 4 °C between inflow and outflow.

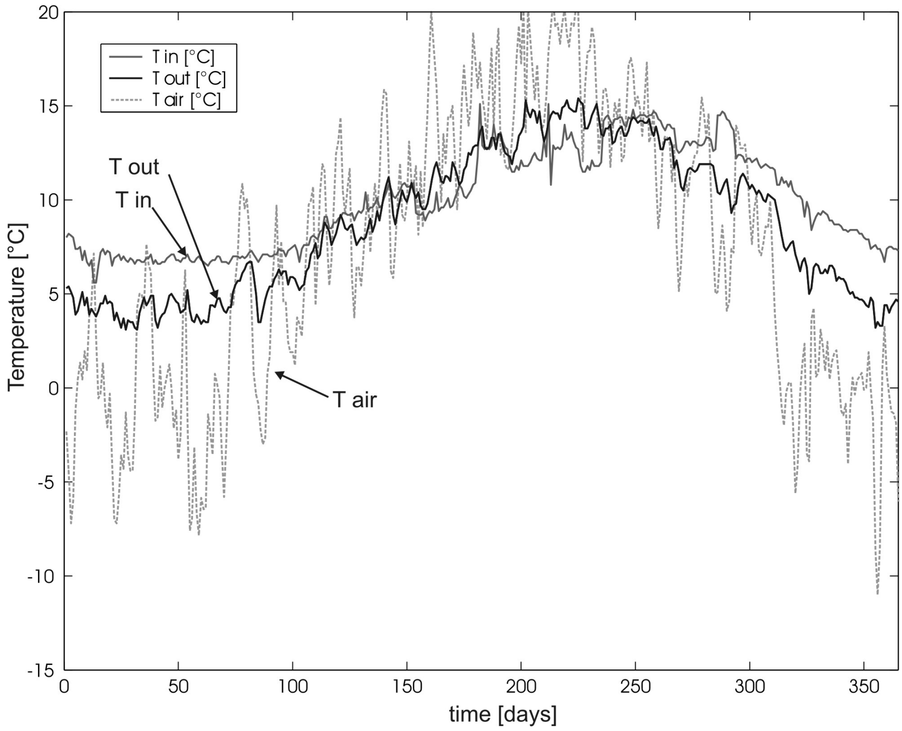


Figure 58: Yearly dynamics of temperature in inflow, outflow and air at the WWTP Vils

Modelling of the WWTP, using the standard temperature dependencies for ASM process rates as given in the literature, was found not to be adequate. It was not possible to simulate the behavior of the treatment plant correctly using the standard configurations as used in the literature. As an example, the outflow concentrations of NH_4N over the period of one year is shown in Figure 59.

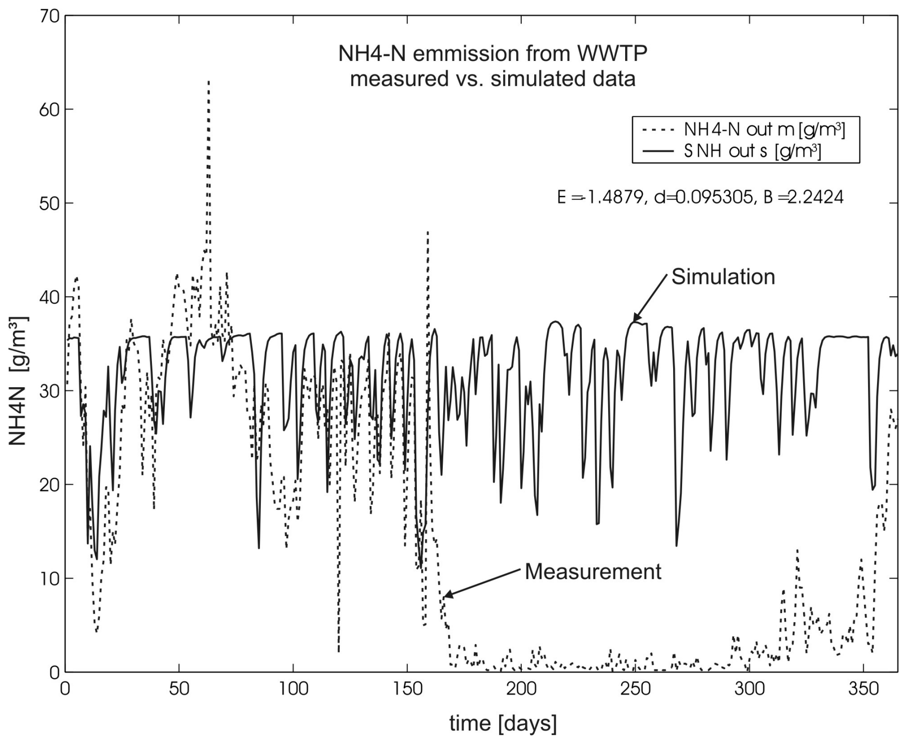


Figure 59: Measured vs. simulated NH4-N dynamics under “standard” ASM 1 conditions

A literature review on the usage of the ASM model, did not help to solve the problem. Approaching literature databases for low temperatures aspects in treatment plant modelling lead to the conclusion that “low temperature” is considered to be around 12 °C. Thus, this underlines again the specifics of the case study presented herein.

In order to overcome the shortcomings of conventional modelling approaches, simulated fluxes and concentrations were analysed. The major shortcoming is the fact that start-up of bacteria activity in the transition from cold to warm conditions does not take place, not even at a reduced rate as in reality. A number of runs were carried out, to test the behavior when changing reaction rates of decay and growth processes. Practically no reasonable improvement was visible. Fact was rather that, during the winter season the autotrophs are decaying down to zero population, making a start-up in summer impossible.

At the inflow to the WWTP, numerical fractioning of COD and N_{TOT} for ASM 1 components is done. As autotrophs are included in the wastewater only to a negligible extent, it is common practice to define zero fraction for X_A components.

In contrast to setting X_A to zero, a low but constant concentration was fed into the plant throughout the year, enabling the restart of the plant in spring. A concentration of 1.0 mg/l was used at the inflow which is considered feasible. The feeding concentration is chosen such that no unnatural artificial accumulation of autotrophs in the tanks is possible, but the restart is enabled.

To get the drop of NH_4-N concentration modelled in the best possible way, temperature dependent coefficients $k_{(T)}$ for decay and growth rates were modified compared to usually used ranges of these coefficients.

$$k_{(T)}=k_{(20°C)}.exp^{(\theta.(T-20))}$$

Calibration parameters in treatment plant model were modified as followed:

μ_A	maximum aut. growth rate	(1.0 in d^{-1}; θ = 0.098)→(0.9 in d^{-1}; θ = 0.098)
b_A	aut. decay coefficient	(0.15 in d^{-1}; θ = 0.08)→(0.10 in d^{-1}; θ = 0.08)
X_A	aut. Conc. - WWTP inflow	(0 mg/l)→(1.0 mg/l)

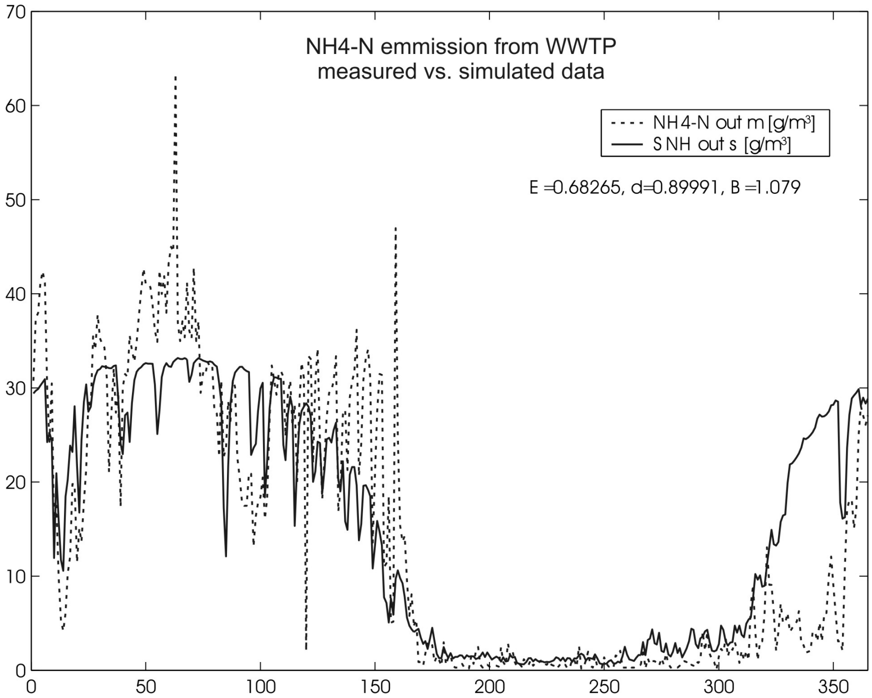


Figure 60: Measured vs. simulated NH4-N dynamics under calibrated ASM 1 conditions

5.5 CONCLUSIONS

This chapter dealt with data assessment and model calibration. The case study Vils was used as an example. The different data sources are outlined, as well as data that were rarely or not available at all. The required parameters to describe the system were presented, not aiming for a detailed (pipe-wise) assessment of the system. With regards to required pollutant concentrations, partly measured and partly literature values were used depending on the availability.

The calibration of the model had to be based on the measurement at the WWTP inflow and partly observations of overflow events. Three quality indicators were used to evaluate the calibration quality. Aside the indicators' application, the nature and sensitivity of indicators was discussed on the basis of randomly generated data series. It was shown that the Nash Sutcliffe Efficiency *E* and the Index of agreement *d* are correlated for unbiased data series of simulated (*S*) and measured data (*M*). Therein *d* indicates a better calibration than the corresponding *E*. The Bias *B* showed no correlation to either one of the two indicators *E* or *d*. For data series (*S* and *M*) that are optimized for the Bias the correlation between *E* and *d* decreases. Thus, quality indication (with *E* and *d*) may lead to a different picture after a first adjustment for e.g. the yearly discharged volume (reflected by the bias). Further it was shown that the Pearson correlation coefficient *C* is not a suitable indicator for calibration quality as it is designed for use with independent variables. Especially systematic differences between simulated (*S*) and measured data (*M*) such as linear deviation or offset are troublesome. For both a misleading good correlation is indicated even when a large deviation between data sets.

For the case study Vils calibration of the sewer system was made stepwisely by calibrating the DWF hydraulics, the storm water hydraulics and the pollutant concentrations.

For modelling of DWF hydraulics and pollutant fluxes a unity spline was used at the subcatchments. The spline's shape was maintained for all subcatchments where it was scaled for the connected PE of each catchment. The quality indicators used were found to be sensitive to the time scales of measurement. Comparing 2h composite samples with simulation results at 5 minute scale leads to a different result than using 2h mean values for the simulation as well. In the examples presented, an increasing quality indicator was observed.

Calibration of the WWTP was a special task due to a low inflow temperature to the WWTP being around 6-7 °C during winter season. Due to the very low air temperatures, an additional drop in wastewater temperature is observed, resulting in a resulting temperature of on average 5 °C during winter. The ASM models commonly used are not designed to cope with such low temperatures. Autotrophs are – numerically – decimated down to a zero population during the winter season, unable to recover thereafter. Modifications of temperature coefficients of autotrophic growth and decay did not lead to the desired improvement whereas inoculation of autotrophs (X_A) at low concentrations did. This inoculation occurs in reality as well at low concentrations via the WWTP inflow.

6 CITY DRAIN – APPLICATIONS AND INTERPRETATION OF RESULTS

A model created in CITY DRAIN can utilize almost all features provided within the Simulink environment. The simplest way of using CITY DRAIN is to set up a model and run it from a defined start and end point. This is usually done via the user interface of the model file.

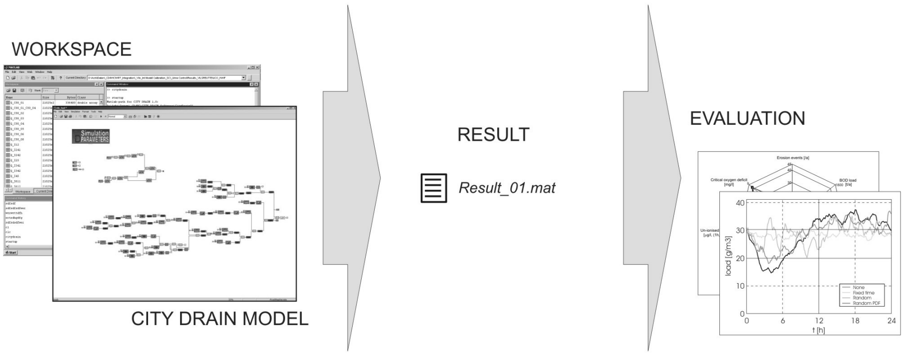


Figure 61: Application of CITY DRAIN via the user interface

In the following a number of possible applications beyond the "straightforward application" with fixed parameters is shown. The applications range from CITY DRAIN being used for testing multiple scenarios to featuring real time control (RTC) and model-based predictive control (MBPC) technology.

Common for all model applications is the need for interpretation of results. As result an abundance of data is generated. Flow series with associated pollutant concentrations are given for different locations of the urban drainage system as e.g. WWTP outflow, overflows, etc. Appropriate post-processing and interpretation of the produced data is indispensable. The goal is to condense the data gained as modelling results to a feasible level allowing judgement on the situation.

6.1 APPLICATIONS

6.1.1 SCENARIO ANALYSIS BY SYSTEMATIC PARAMETER VARIATIONS USING BATCH MODE

Once a model is set up, it must not exclusively be run via the graphical interface but can as well be run via a Matlab script. Further input parameters in any block mask are not necessarily to be entered as fixed numerical values. Variables can be used instead which are specified using the Matlab workspace. Combining these two options allows to run a number of simulations using the same model in batch mode (see Figure 62).

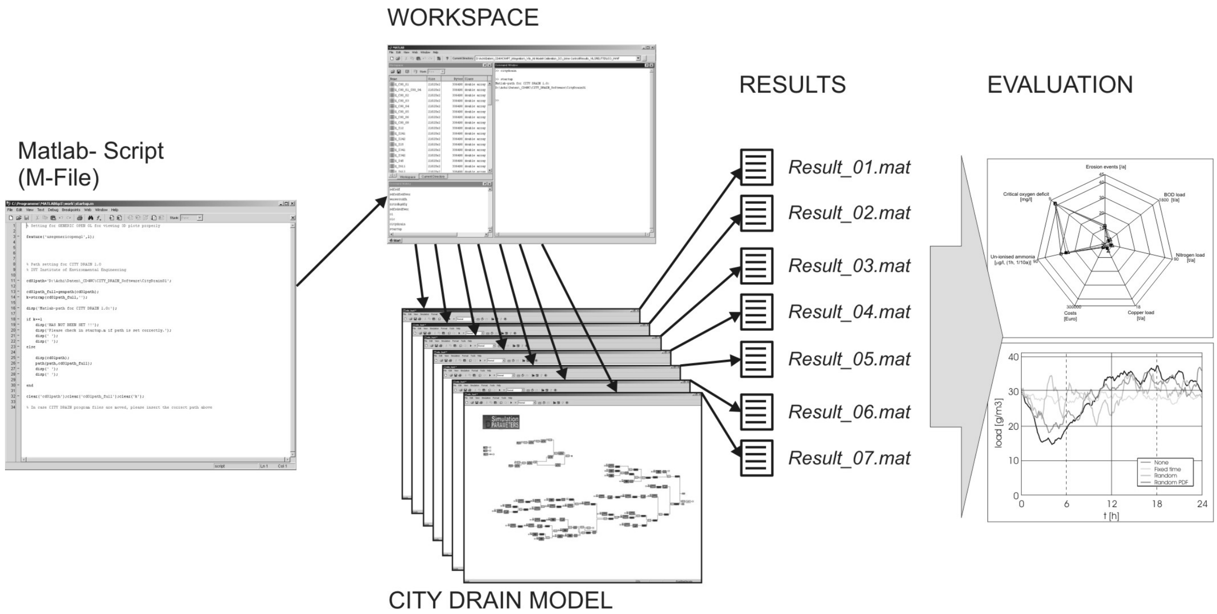


Figure 62: Schematic Matlab based script to run of a CITY DRAIN model with varied parameters

Following steps are to be in the batch coding:

(1) definition of variables used by the CITY DRAIN model (mdl-file)
(2) running the model
(3) storing the produced results
(4) redefine one or more variables in a systematic way

By systematically changing one or more variables in between runs, it is possible to perform for instance a sensitivity analysis of a system. Evaluation of results may be done after each run or at the end depending on the purpose of the modelled study. Engelhard *et al.* (2006) have used the described batch simulation in their work. Aim was the structured evaluation of CSO performance indicators and their compliance with ambient water quality targets. CITY DRAIN was used in batch-mode was made as well within the case study Vils (CD4WC D 7.2, 2006). Therein different measures were tested and evaluated in the latter.

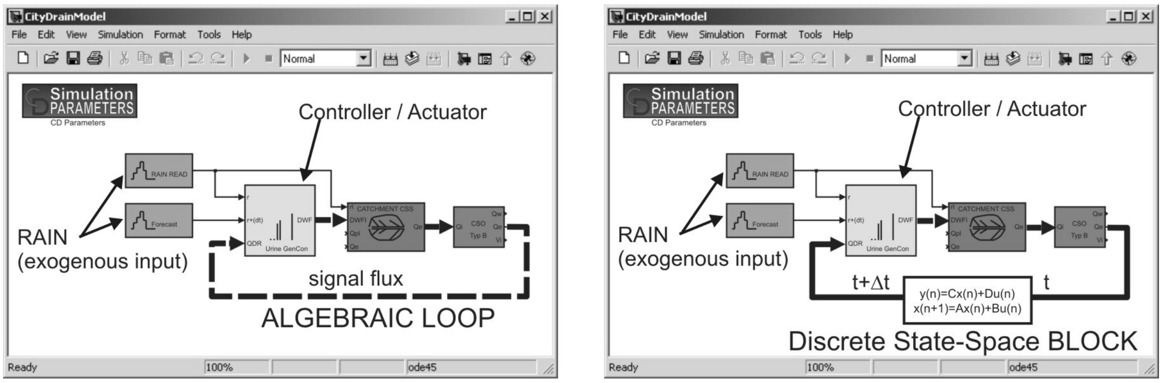
6.1.2 Real time control (RTC)

In (Schütze *et al.*, 2004) RTC is defined as followed:

"An urban water system is controlled in real time if process variables are monitored in the system and continuously used to operate actuators during the process."

Principal concept is to manipulate parts of a system using a control strategy with defined set points and rules. The measured state of a system (e.g. water level, flows,…) as well as ex-ogenous inputs (e.g. measured rain data) may be used within the applied control algorithms (Butler and Schütze, 2005). Depending on the objective, RTC can be classified for different types such as volume-based RTC, pollution-based RTC or immission-based RTC (Vanrolleghem *et al.*, 2005).

All in common is that they can be schematized as control loops comprising fluxes of water, mass or signals. Thus, the simultaneous calculation of different system compartments is vital to properly account for feedback information from further "downstream" located compartments.



(a) (b)

Figure 63: Scheme of RTC implementation in CITY DRAIN (a) creation and (b) break up of algebraic loop using a discrete state-space block.

Figure 63 (a) shows such a setup realized in CITY DRAIN, dealing with the controlled discharge of ammonia to the sewer system. Urine as the main ammonia source is separated and stored by means of NoMix toilets. The controlled discharge of urine is made by utilizing RTC technology. Details on the implementation can be found in (Achleitner *et al.*, 2006c; Möderl, 2006) which is attached as Appendix E being part of this thesis.

<u>Appendix E</u>

Achleitner, S., Möderl, M. and Rauch, W. (2006).
Waste design by urine separation – the development of control options for the case study Vils/Reutte. *Urban Water Journal*. (submitted).

When realizing such information flow numerically, this typically results in an algebraic loop. In an algebraic loop the output of an equation is needed as input information. This creates a situation which cannot be solved by standard procedures. Such loops are required to be broken up at a certain point, which is in CITY DRAIN done using a "Discrete State-Space" block. (see Figure 63 (b)). The block essentially retains information flow for a time step, allowing the numerical engine to solve the equation. The drawback is that the applied control algorithm utilises not actual information, but one time step Δt old.

6.1.3 MODEL-BASED PREDICTIVE CONTROL (MBPC)

Model-based predictive control (MBPC) is characterized by the calculation of the system's future behaviour using a deterministic model of the wastewater system (Onnen *et al.*, 1997; Rauch and Harremoës, 1999; Rossiter, 2003).

The idea is – as in RTC - to influence the system's actuators to optimize the behaviour. Instead of applying rule based decisions utilizing measurements (as in RTC), decisions are based on simulations of the future state of the system. Simulating the behaviour is therefore required within a prediction horizon T_P. All relevant processes that are considered are to be accomplished within this period. The obtained output (either final state or time series within T_P) is then evaluated with regard to a defined (one or more) objective function *O.* For the actual implementation in reality, a simulation platform such as CITY DRAIN may be used.

Achleitner and Rauch (2005a) have used CITY DRAIN to test a MBPC strategy offline. For offline testing, a main model (representing reality) and a sub-model for modelling the future behaviour (model-based prediction) are needed.

As a general methodology for setting up and testing MBPC applications, two parallel and independently running models should be used. The main model (representing reality) is to be run stepwise until the MBPC-(sub)model is to be started in order to find the optimum setting of the controlled variables (pump rates, gate opening,…). Multiple runs of the sub-model are then to be used to find the optimum controller setting for the subsequent simulation period in the main (reality) model. The main model runs by subsequently implementing the calculated control strategy until it is time again to restart a new system optimization (see Figure 64)

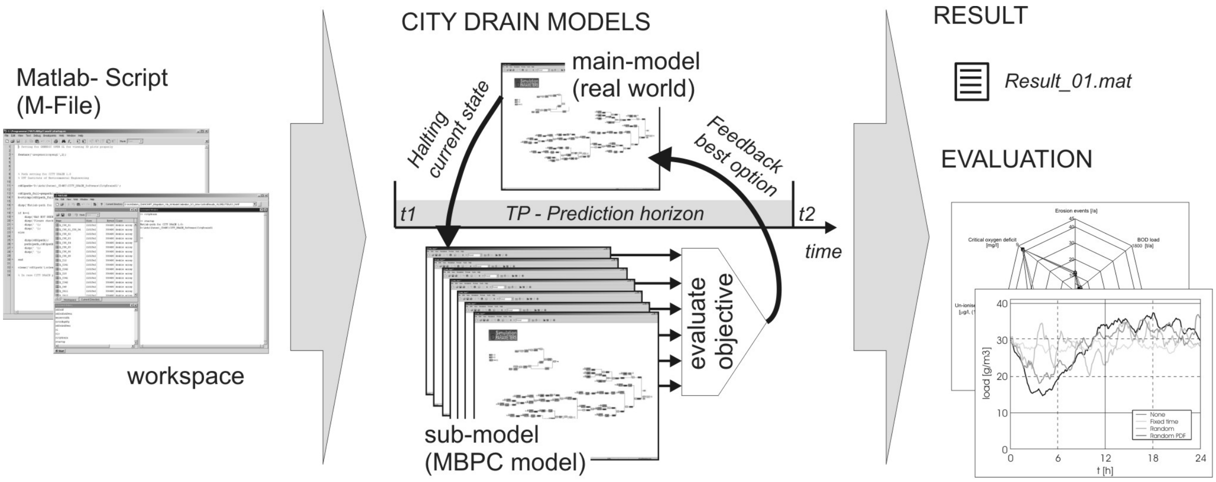


Figure 64: Schematic on how to realize model-based predictive (MBPC) within CITY DRAIN

Aim in the application of Achleitner and Rauch (2005a) was to control the discharge of an upstream located hydro power station to increase the river base flow. Corresponding publications are attached as Appendix C and D as they are part of this thesis.

<u>Appendix C</u>

Achleitner, S., DeToffol, S., Engelhard, C. and Rauch, W. (2005).
Model-based hydropower gate operation for mitigation of CSO impacts by means of river base flow increase. *Water Science and Technology*, Vol 52 No 5 pp 87–94.

Appendix D

Achleitner, S. and Rauch, W. (2006).
Increase of river base flow by hydropower gate operation for mitigation of CSO impacts - Potential and Limitations. *Water Resources Management.* (submitted).

The objectives were ambient water quality focused, where direct measurements in the system were not utilisable. Thus, a rain forecast was used as exclusive input to the MBPC submodel, avoiding the creation of control loops. Utilization of internal signals as additional input data is still possible. The requirement will be however to break up the resulting algebraic loops as shown earlier for RTC applications.

6.2 EVALUATION CRITERIA IN URBAN DRAINAGE MODELLING

Evaluation criteria can be manifold. As discussed earlier in chapter 2, the WFD (EU/2000/60/EC-en, 2000) imposes a combined approach, where both emission or water quality are to be considered. Impacts can be either hydraulic or pollution, all having different temporal scales. Grouping of impacts is described by e.g. (Blumensaat *et al.*, 2006; Rauch *et al.*, 1998; Schilling *et al.*, 1997).

An overview on different legal boundaries and associated criteria has been given earlier in this work. Common to all is that a limitation of parameters in one or the other form is stated. This can either be a single limit concentration or more complex relations including reoccurrence intervals as found in the UPM manual (FWR, 1998). When judging the current situation or the improvement due to a specific measure, compliance to these criteria – subsequently termed *legal criteria* – is required.

When judging the effects resulting from applied measures, legal criteria are essential but may not be considered exclusively. Judgment can as well be based on the increase and decrease of certain parameters having no defined limit values. For instance the number of annual CSO events occurring in a sewer system can be mentioned. Although no legal limits are defined, improvements may be indicated by this criterion.

Thus, for the purpose of developing an evaluation scheme, criteria may be categorised as:

- Legal Criteria; Criteria that are related to a certain standard or regulation.
- Criteria that are not related to a standard or regulation.

To evaluate the current situation - subsequently termed baseline scenario (0) – the legal criteria are suitable. When aiming for the evaluation of one or more alternative scenarios, both legal and other criteria may be used. Thus, for each of the scenarios evaluated (I, II, III…), the same set of one or more criteria (1,2,3,4…) is applied.

Magnitudes of numerical values resulting from the different criteria may be quite different. Thus, to bring different criteria together in one evaluation scheme, the use of an index system is proposed. Aim is to normalize (scale) absolute values of different criteria in order enable an intercomparison. For the derivation of dimensionless indexes to adequately compare different measures the index system is drawn for the different types of criteria. Consequently two types of index are proposed:

VI …Violation Index
II …Improvement Index

relating to legal and other criteria used.

A similar approach is proposed by Blumensaat *et al.* (2006), where the actual calculation of indexes presented here differs somewhat.

6.2.1 VIOLATION INDEX FOR LEGAL CRITERIA

The violation index *VI* indicates how "well" limits of different criteria are met. For given maximum allowable limits (e.g. the concentration of NH_3N in the river) the limit value (C_{LIM}) is to be larger than the measured/calculated value (C_X).

$$C_x < C_{LIM}$$

Under these circumstances, the criterion is not violated. To obtain the dimensionless index VI, the ratio of the two can be used.

$$VI = C_x / C_{LIM}$$

where values of $VI<1$ indicate no violation of the limit. In return $VI>1$ indicates a violation. For other criteria, requiring a minimum to be exceeded ($C_x > C_{LIM}$) (e.g. "minimum percentage of reduction" required for WWTP) the above formal reverses to:

$$VI = C_{LIM} / C_x$$

For a given baseline scenario *(0)* as well as for different simulated scenarios (I, II, III…) the violation potential can be calculated for the different criteria (1,2,3,4…) applied (see Figure 65).

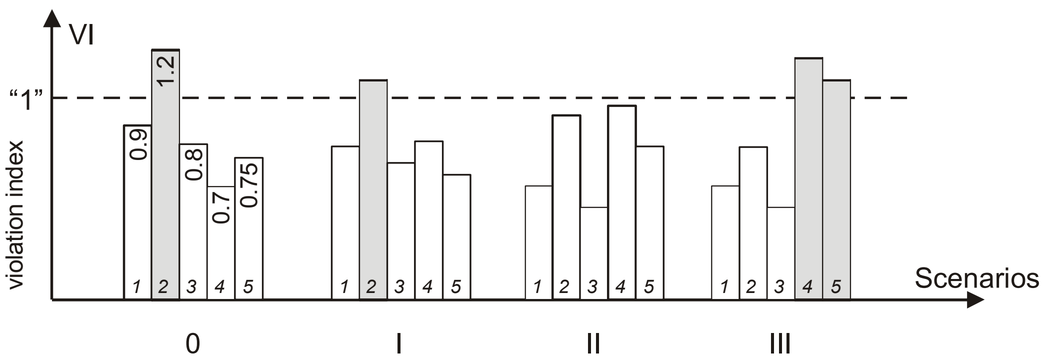


Figure 65: Example evaluation of different scenarios using the violation index VI

The operator thereby gets the opportunity to quickly identify scenarios with little or no violations. The index may be used as an e.g. strict knockout criterion for a scenario, as the used criteria rely on legal constraints. How different criteria are evaluated in a combined way, forming a single index is discussed below.

6.2.2 IMPROVEMENT INDEX FOR NONLEGAL RELATED CRITERIA

Focus of the Improvement index II is the relative comparison of states in a new scenario and baseline scenario. Therein legal criteria used in the violation index as well as others without defined limits can be considered. For a dimensionless comparison, the absolute improvement (from baseline to current scenario) is scaled for the baseline scenario.

$$II = (Co - Cx) / Co$$

Thereby II ranges from 0-1 for gained improvements and from 0 to $-\infty$ for a decrease in a criterion. The stated formula is valid for criteria (such as e.g. concentration) that improve when the numerical value drops. For other criteria (such as e.g. DO content) an improvement is given when the numerical value increases. Therefore the Improvement Index is to be rewritten as

$$II = (Cx - Co) / Co$$

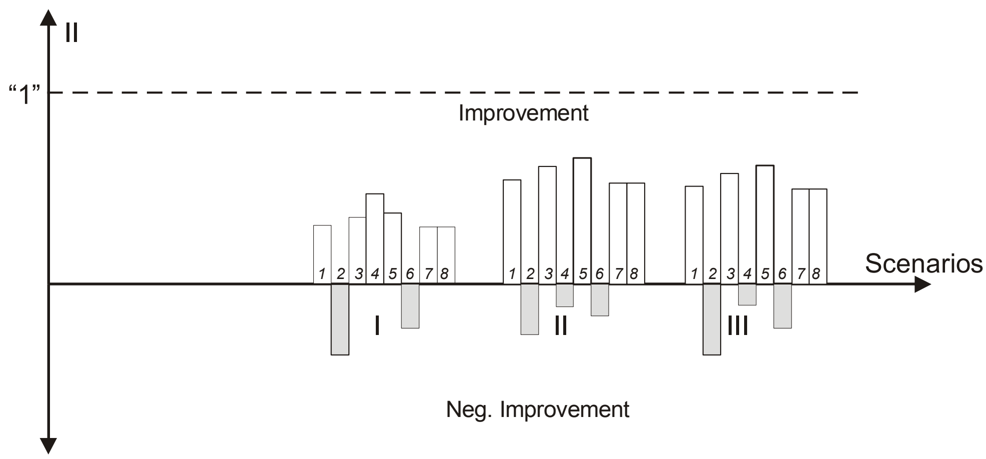


Figure 66: Example plot of improvement indexes II for different scenarios

A negative improvement in a criterion may happen frequently and can be seen as a trade-off for the improvement of other criteria.

6.2.3 COMBINED EVALUATION OF CRITERIA AND COSTS

Combined evaluation of different criteria can be made for the sake of providing an easier overview. Forming a single numerical value for this purpose can be done in various ways.

The simplest approach is to take the mean value of considered criteria. This is a straightforward (and clear) way to obtain a single index. Implicitly the weight “1” is attributed to each of the criteria used. Depending on the situation, different weights may be attributed to the different criteria.

No matter how a single numerical value is obtained, information is lost. The loss in information in the course of the index reduction, is strongly influenced by the application of weights. This may even more hide or warp the original content of information.

When discussing the weighing of criteria and the associated warp of information, it is essential to go a step back to the selection of criteria. The selection of criteria itself has a large impact on the evaluation and is therefore interpretable as a way of putting weights. Depending on the number and types of criteria selected, weight is put onto one or the other type of criteria. This is even more true, when purposely leaving out certain criteria.

Thus, it is very important to show the way (and motivation behind) how the evaluation is done. A combined index can then provide a condensed overview that is valuable for further decision. In Figure 67 the mean improvement index II_{MEAN} is plotted for the scenarios of the virtual example used.

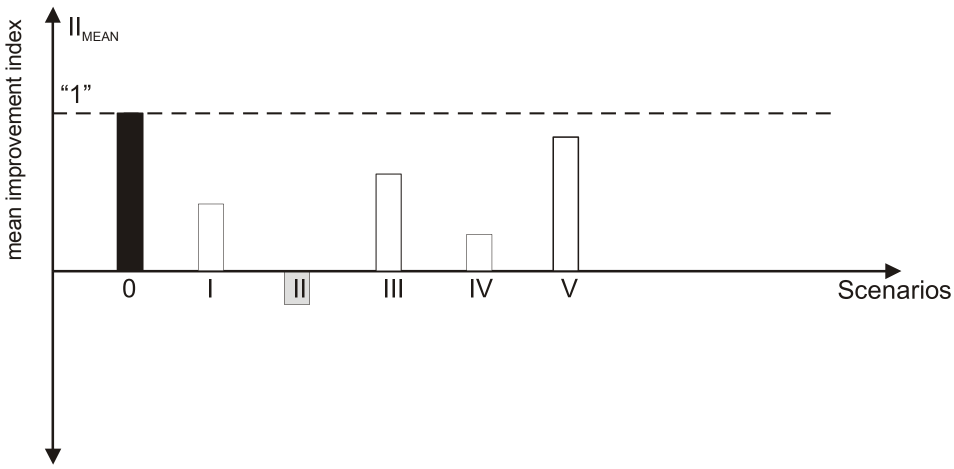


Figure 67: Example plot of mean of evaluated improvement indexes.

Comparing Figure 66 and Figure 67 shows that there is a loss in information in exchange of a more condensed comparison of scenarios. Information loss is clearly seen for scenarios I and III. Therein some criteria showed a decrease which is – in the condensed index – not visible any more. Depending on the situation, single scenarios may show an improvement, although violating a single criteria. Elimination of such a scenario upfront, is the responsibility of the user.

Including costs is even more delicate and requires even more a structured methodology and transparency in “what was done”. In the example below, best scenarios are considered as being of low costs *(M)* and large improvement. Figure 68 shows a plot of the improvement index gained versus the costs associated for each scenario.

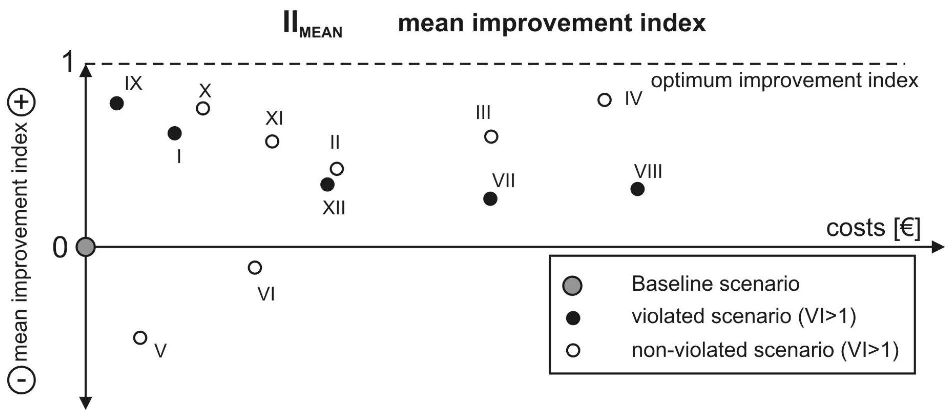


Figure 68: Plot of improvement index II versus costs associated to the scenario

An indication of scenarios that violate legal limits is essential as they may be excluded in any further processes. As violated scenarios are eliminated, the most (cost)effective scenario is characterise by a minimum of costs (ΔM) and an improvement index close to "1".
A possible optimisation function could be

$$O = \Delta M * (1 - II_{MEAN})$$

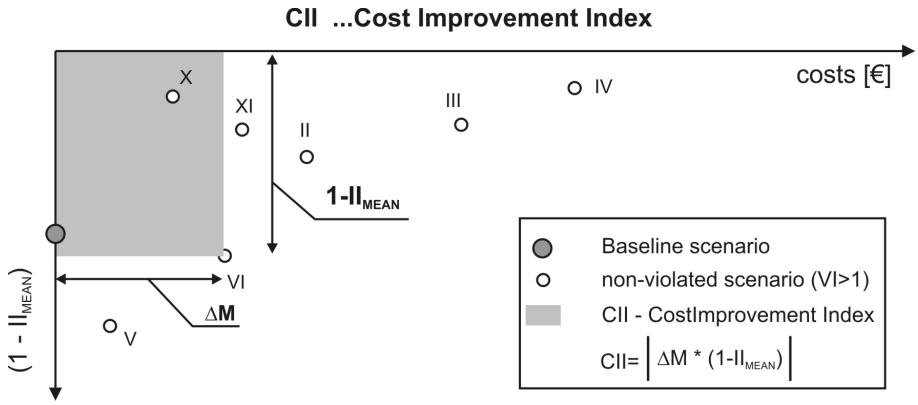


Figure 69: Cost index calculation-interpretation of minimized objective function as area over ΔM and $(1-II_M)$

Figure 69 illustrates the cost index interpreted as a minimum of the area drawn over ΔM and $(1-II_M)$.

6.3 CONCLUSIONS

In this chapter different applications of CITY DRAIN were discussed. The "straightforward application" of running a model from a defined start to end point is the simplest way of using CITY DRAIN. It is doable via the graphical user interface of the model file. More sophisticated applications of CITY DRAIN includes (a) scenario analysis, (b) real time control (RTC) and (c) model-based predictive control (MBPC).

Scenario analysis requires to systematically modify the system's parameters. Therefore system parameters are defined as variables in the model and specified via the Matlab workspace. Systematic variation and running of the model is organized by a Matlab script. Produced data is to be stored in that course. Engelhard *et al.* (2006) used that method to perform a structured evaluation of CSO performance indicators and their compliance with water quality indicators. A scenario analysis using this method was made in the frame of the CD4WC project testing different management options for the case study Vils (CD4WC D 7.2, 2006).

The principal concept of real time control is to manipulate parts of the urban drainage system using a control strategy with defined set points and rules. The state of specific parts of the system or exogenous input may be used as data. For instance measured water levels, flow rates or rain data as exogenous input may be used as input to the algorithms.

RTC technology was applied within the work of (Achleitner *et al.*, 2006c) attached to this work as Appendix E.

<u>Appendix E</u>

Achleitner, S., Möderl, M. and Rauch, W. (2006).
Waste design by urine separation – the development of control options for the case study Vils/Reutte. *Urban Water Journal*. (submitted).

The designed RTC block utilized measured data from the system. Aside rainfall and rainfall forecasts, the throttle flow at one CSO was used as input information.
All RTC measures have in common that the "flow" of feedback information usually creates algebraic loops. Numerically, the output of an equation is required as input, which cannot be solved as is. Breaking up of loops is to be done, resulting in a delay of informations. For systems such as CITY DRAIN that run with fixed time steps, this results in input information being one time step old.

Model-based Predictive Control (MBPC) is characterized by calculating the systems future behaviour by used of a (simplified) model of the system. Optimization of the system is not done by rule-based decisions but decisions based on the model output. For testing MBPC measures offline, a general scheme for testing was presented in this chapter. Corresponding publications (Achleitner *et al.*, 2005a; Achleitner and Rauch, 2006) are attached as Appendix C and D.

<u>Appendix C</u>

Achleitner, S., DeToffol, S., Engelhard, C. and Rauch, W. (2005).
Model-based hydropower gate operation for mitigation of CSO impacts by means of river base flow increase. *Water Science and Technology*, Vol 52 No 5 pp 87–94.

<u>Appendix D</u>

Achleitner, S. and Rauch, W. (2006).
Increase of river base flow by hydropower gate operation for mitigation of CSO impacts - Potential and Limitations. *Water Resources Management.* (submitted).

For this measure, utilization of inputs other than rainfall forecasts was not possible. In general, MBPC strategies may of course utilize other inputs such as measurements from the system.

Common to all model applications is the need for interpretation of results. Simulation results (time series of flow and pollutant concentrations) require to be analysed to enable judgement on the situation. Criteria can be classified in legal criteria and non-legal related criteria.

A possible scheme was presented to deal with the application of evaluation criteria in the frame of scenario analysis. The proposed system is based on the derivation of an index system using criteria being scaled to unity. Two indexes, Violation Index(VI) and Improvement Index(II) are proposed describing the degree of violation or improvement for a specific scenario and criterion. A further simplification to combined indexes was made to provide more condensed information. Simplicity in presenting data is trade for a loss of information. Weighing of criteria within this process causes an additional influence towards the warping of information.

Thus, no matter how information is simplified, it is very important to provide clarity on “how it is done”.

7 CONCLUSIONS AND OUTLOOK

7.1 CONCLUSIONS

The final conclusions presented below are covering in a first section findings regarding the legal framework of urban drainage and implications regarding integrated modelling. Subsequently modelling aspects and models implemented in the software CITY DRAIN are discussed followed by aspects of data assessment and model calibration. The catchment Vils has been used as an example. The corresponding papers are attached as Appendix A and B to this thesis.

The last part concludes on different applications of CITY DRAIN. The application dealt with Real Time Control (RTC) and Model-based Predictive Control (MBPC), both aiming for the optimization of the integrated urban drainage system. Corresponding papers are attached as Appendix C/D and E.

Finally focus was put on the interpretation and evaluation of results. The implications of condensing information in the course of evaluation is discussed.

7.1.1 LEGAL FRAMEWORK

In the last years design procedures of urban drainage systems have shifted from end of pipe design criteria to ambient water quality approaches. The improvement of the receiving water quality became a central point as it is the core element of the Water Framework Directive (WFD).

The historical development and implementation of the WFD have been presented. Details on the different steps and their interactions have been presented in (Achleitner *et al.*, 2005b) attached as Appendix A.

<u>Appendix A</u>

> **Achleitner, S.**, DeToffol, S., Engelhard, C. and Rauch, W. (2005).
> The European Water Framework Directive: Water Quality Classification and Implications to Engineering Planning. *Environmental Management*, Volume 39, No. 4, 517-529.

It was shown that the definition of reference sites and water bodies as well as the subsequent water quality classification strongly depend on each other. Especially the size of water bodies will have a large impact on the quality.

The WFD demands a wider view, especially taking ecological conditions into account. Regarding biological indicators it was found that monitoring these requires more effort and implies larger uncertainties than dealing with chemical parameters. Methods for water quality classification (AQEM, FAME and STAR) concerning biological indicators have been presented, although they are not subjected to any approval by national authorities. The tools presented are valuable for defining boundaries for the different states of the ecology. For planning purpose, tools for prediction of biological conditions will become more important in the light of the WFD. Still, tools linking chemical and biological conditions of streams are limited. One research project (PAEQANN) was found, linking chemical and biological conditions using Artificial Neural Network (ANN) technology.

Regarding increased efforts and associated costs to be taken due to the WFD the combined approach has been discussed. The WFD introduces a combination of emission and immission standards with the common understanding that the more stringent one of the two requirements applies. Based on an example at the alpine river Drau (Austria) (see Appendix A) the stringent application of the combined approach is questioned. According to the legal implementation in Austria (WRG, 2003), where a shift towards a full immission-based approach is legally possible, it was shown that the WFD does not necessarily require increased efforts.

As the WFD does not replace existing standards but bundle them to a uniform approach, standards in the field of urban drainage are presented. A number of regulations can be found that relate to different parts of the urban drainage system. Immission and emission aspects are both covered therein.

Integrated modelling is thereby found to be an appropriate tool to assess the systems behaviour as a whole. Available models allow to predict the behaviour and to optimize the system. This is especially true when considering the interaction of the different parts of the urban drainage system. All of the regulations presented require modelling in one or the other form. To cope with the assessment of impacts and effects in the planning stage, especially long term modelling plays – in the regulations presented – a leading role. Modelling as such is thereby an import tool to predict the future behaviour of a system. As the water framework directive demands the assessment of the complete picture, integrated modelling of the whole system is of major importance.

7.1.2 MODELLING CONCEPTS

In chapter 3, basic modelling concepts were reviewed and presented in order to gain a structured and comprehensive overview. Thereby the fundamental equations and concepts typically used to describe rainfall runoff process, hydraulic transport routing, pollutant transport/routing and pollutant processes were presented.

Physical and conceptual models for hydraulic and pollutant transport were discussed. Focus was on the conceptual models as these are used in the software CITY DRAIN. The main disadvantage of physical models such as the St. Venant Equations is seen in the demanding computational effort. This limits their use for long term simulation, favouring conceptual models as they are less demanding in computational effort. In contrast, for a detailed hydraulic assessment of sewer hydraulics including surcharge and overflow of the sewer system, application of physical models is required. Thus, depending on the aim of the study, different types of models are suitable. This is true for hydraulics, as well as for pollutant routing and pollutant processes.

With regard to the modelling of pollutant processes, the general concept of ASM type models based on the Petersen Matrix was presented. Focus was not on showing the Activated Sludge Model (ASM) type models in detail, but to show the modelling concept behind. The concept shown is standardized and commonly used for modelling of chemical-biological systems. The number of other models such as the River Water Quality Model No.1 (RWQM No.1) use the same modelling concept.

7.1.3 CITY DRAIN – MODELLING ASPECTS

In chapter 4, the modelling approaches used within CITY DRAIN are presented. The corresponding publication regarding the resulting software CITY DRAIN and the user manual are attached to the thesis as Appendix B and E:

<u>Appendix B</u>

Achleitner S., Möderl M. and Rauch W. (2006).
CITY DRAIN © - an open source approach for simulation of integrated urban drainage systems. *Environmental Modelling & Software*, (accepted)

<u>Appendix F</u>

Achleitner S. and Rauch W. (2005).
CITY DRAIN © - an open source integrated simulation of urban drainage systems - User Manual. Institute of Environmental Engineering, University of Innsbruck, Austria.

CITY DRAIN - realized in Matlab/Simulink - has been developed as an open source software. The first visible benefit of CITY DRAIN being open source is, that no costs arise when obtaining a copy, although this is not the most important issue. More important is the possibility to allow full manipulation of codes. The insight into codes avoids that one is forced to use a black box. Further, the open framework provided allows to manipulate, extend and reuse the codes. This is a clear advantage when a new model is required. In turn, this requires advanced skills of the engineer, especially with regard to programming.
The generated code may be used and/or redistributed having a growing library. This is clearly an advantage requiring responsibility when creating and redistributing codes. In the case of CITY DRAIN the scientific community is seen as the platform for this.
The support of open software is a critical point. Especially keeping track on continuously modified codes is difficult and time consuming.

Daily engineering work with modelling software requires not only simplicity in handling. It is important to provide a certain flexibility in the software and the applied models to be adjusted for different scenarios and cases.

Conceptual models were used as they are less demanding in CPU-requirement than physical models. This enables long-term modelling (several years) with continuous time series at high resolution (5-10min) which is hardly possible when using physical models. Further the use of criteria that demand statistical evaluation (e.g. UPM Manual (FWR, 1998) or ÖWAV-RB19 (2006)) would not be possible or only to a very limited content by applying short time series.

CITY DRAIN is realized within the Matlab/Simulink environment. Therefore the concepts of state-space modelling is utilized for urban drainage models allowing a block-wise description of the urban drainage system. Fixed discrete time steps are used throughout all blocks.

Utilisation of data on flow rates and associated pollutants is discussed. Special attention was put on dealing with different types of data such as grab samples or composite samples. A differentiation between the types of data is of importance, especially when interpolation of data points is required. The derivation of the respective equations for interpolations is provided enabling a correct handling including mass balances for water and substances.

As flows of water and matter in CITY DRAIN are considered as mean values over the last time step, discretisation of models is done using a modified discrete scheme. The advantage is that all model formulations become simpler and gain numerical stability. The Muskingum method implemented requires only two parameters (C_X and C_Y) compared to three parameters *(C_1, C_2 and C_3)* in its original form. Additional modifications allow to deal with upstream and compartment-wise inflows.

The type of model used depends, as outlined, on the aim of simulations. Thus, as the software is open source, it is possible to use only the substances/models that are necessary for the case. With CITY DRAIN an open framework is provided that allows to define the substances to be used as well as to modify a model if required.

7.1.4 DATA ASSESSMENT AND MODEL CALIBRATION

Chapter 5 dealt with data assessment and model calibration. Measured data are the basis for model calibration and to get reliable predictions. The case study Vils was used as an example to outline the type of data required. The different data sources were outlined, as well as the data types that were rarely or not available at all. An overview on the required data as well as its assessment was given. With regard to required pollutant concentrations, partly measured and partly literature values were used depending on the availability.

The example of the case study Vils showed that data availability is often “sufficient” rather than “perfect”. As long as there is no intention to run intensive measurement campaigns, one is required to deal with the available. Case dependent, this may limit the calibration to certain aspects of the system. For the Vils case study, calibration of the sewer catchment had to be based on fluxes at the WWTP. Aspects such as e.g. overflow events at CSO structures could not be calibrated in detail. Limitations with regard to measured data raises the question regarding the necessary complexity of the model to be used. Using complex modelling approaches in the case of limited data would not necessarily lead to any better simulation results but would only be demanding in computational time.

Three quality indicators were used to evaluate the calibration quality. Aside the indicators’ application, the nature and sensitivity of indicators was discussed on the basis of randomly generated data series. It was shown that the Nash Sutcliffe Efficiency E and the Index of agreement d are correlated for unbiased data series of simulated (S) and measured data

(*M*). Therein *d* indicates a better calibration than the corresponding *E*. The Bias B showed no correlation to either one of the two indicators *E* or *d*. Further it was proven that the Pearson correlation coefficient *C* is not a suitable indicator for calibration quality as it is designed for use with independent variables. Especially systematic differences between simulated (*S*) and measured data (*M*) such as linear deviation or offset are troublesome. For both a misleading good correlation is indicated even when a large deviation between data sets exist.

For the case study Vils calibration of the sewer system was made stepwisely by calibrating

- the DWF hydraulics
- the storm water hydraulics and
- pollutants.

For modelling of DWF hydraulics and pollutant fluxes a unity spline was used at the subcatchments. Where the spline's shape was maintained for all subcatchments, the scaling was catchment specific according the connected PE. For calibration of DWF pollutants it was shown that quality indicators are sensitive to the time scales of measurement. Comparing measured concentrations from 2h composite samples with 5 minute simulation results leads to a different result than having 2h mean values for the simulation as well. In the examples presented, an increasing value of calibration quality indicators was observed.

Calibration of the WWTP was a special task due to the specific situation at the catchment Vils. The Vils plant is characterised by a low inflow temperature to the WWTP being around 6-7 °C during winter season. Due to very low air temperatures, an additional drop in wastewater temperature is observed in the plant itself down to an average 5 °C during winter.

The ASM models commonly used are not designed to cope with such low temperatures. Autotrophs are – numerically – decimated down to zero population during the winter season, unable to recover and to start up nitrification in spring. Modifications of temperature coefficients of autotrophs' growth and decay did not bring the desired improvements whereas inoculation of autotrophs (X_A) at low concentrations via the inflow did. This inoculation occurs in reality as well at low concentrations via the WWTP inflow.

7.1.5 APPLICATIONS

In chapter 6 applications using CITY DRAIN and the interpretation of results were discussed. The "straightforward application" of running a model from a defined start to end point is the simplest way of using CITY DRAIN. It is doable via the graphical user interface of the model file. Beyond that, the open structure of CITY DRAIN respectively Simulink allows to perform a number of more sophisticated applications than that. The application of CITY DRAIN for (a) scenario analysis, (b) real time control (RTC) and (c) model-based predictive control (MBPC) is discussed.

7.1.5.1 Scenario analysis

Scenario analysis requires to systematically modify the system's parameters. Therefore they are defined as variables in the model and specified via the Matlab workspace. Systematic variation and running of the model is organized by a Matlab script. The produced data is to be stored after each run. Engelhard *et al.* (2006) used that method to perform a structured evaluation of CSO performance indicators and their compliance with water quality indicators. A scenario analysis using this method was made in the frame of the CD4WC project testing different management options for the case study Vils (CD4WC D 7.2, 2006).

7.1.5.2 Real time control (RTC)

The principal concept of real time control is to manipulate parts of the urban drainage system using a control strategy with defined set points and rules. The state of specific parts of the system or exogenous input may be used as data. For instance measured water levels, flow rates or rain data as exogenous input may be used as input to the algorithms.

RTC technology was applied within the work of Achleitner *et al.* (2006c) attached to this work as Appendix E.

Appendix E

> **Achleitner, S.**, Möderl, M. and Rauch, W. (2006).
> Waste design by urine separation – the development of control options for the case study Vils/Reutte. *Urban Water Journal*. (submitted).

The measure tested deals with waste design by manipulating the urine flux. Urine contributes for ~80% of the ammonia loads but for only 1% of the hydraulic load in the DWF. Waste design by urine separation aims to take advantage of these features by developing a controlled release of urine to the drainage system. Separation of urine from faeces is done by special toilets (NoMix toilets). In contrast to classical urine separation, urine is here not taken from the wastewater stream for reuse but stored in the household and released in a controlled way to the drainage system. Goal was an averaging of the daily dynamics in ammonia loads to the WWTP and a reduction of ammonia CSO emissions towards receiving waters. The totally 11 different control strategies for the dynamic discharge of urine were developed and tested. The developed strategies are designed to serve both aims defined above.
Urine production and harvesting was modelled on a microscopic level, balancing all toilets in the system separately. Production itself was based on a stochastic description of the process.

Due to the open source nature of the software, a new block element was introduced in City Drain. This block named "Urine GenCon" is designed to manage the urine generation, storage and the controlled release. Part of the strategies tested included the use of feed-back ("measured") data from the system. Aside rainfall and rainfall forecast, the throttle flow at one CSO was used as input information to the created RTC block.

The strategies covered a wide range from most simple ones to complex schemes. The evaluation of strategies with respect to both aims showed that the simple approaches were most effective. At best a reduction of ammonium emissions of 42% (load based) was possible.

All RTC measures (including the one presented here) have in common that “flow” of feedback information exists that possibly creates algebraic loops. Breaking up loops is to be done, resulting in a delay of information. For systems run with fixed time steps (as CITY DRAIN) this results in input information being one time step old. Software using adaptive instead of fixed time steps may minimize the failure by reducing the time step if necessary. In return the reduction of the time step size in the course of a break-up of algebraic loops, may cause an increase in generated data.

7.1.5.3 Model-based predictive control (MBPC)

Model-based Predictive Control (MBPC) is characterized by calculating the system’s future behaviour by use of a (simplified) model of the system. Optimization of the system is not done by rule-based decisions but decisions based on the model output. The future state of the system is evaluated by means of simulation within a prediction horizon. The evaluation for a defined objective function is done, based on the behaviour within this horizon or the final state.

For testing MBPC measures offline, a general scheme for testing is presented in chapter 6. This scheme was then applied by (Achleitner *et al.*, 2005a; Achleitner and Rauch, 2006) and is attached as Appendix C and D.

<u>Appendix C</u>

Achleitner, S., DeToffol, S., Engelhard, C. and Rauch, W. (2005).
Model-based hydropower gate operation for mitigation of CSO impacts by means of river base flow increase. *Water Science and Technology*, Vol 52 No 5 pp 87–94.

<u>Appendix D</u>

Achleitner, S. and Rauch, W. (2006).
Increase of river base flow by hydropower gate operation for mitigation of CSO impacts - Potential and Limitations. *Water Resources Management.* (submitted).

In this MBPC application an in-stream measure was developed to be applied at low flow stretches downstream of water intakes of hydropower stations. The idea was to mitigate the acute pollution in the stream caused by CSO events by creating a sufficient increase of the river base flow. The framework and implications for having an appropriate real time control (RTC) concept based on model predictive control (MBPC) are discussed. The herein described measure differs from classical RTC applied in sewer systems, as not the wastewater system itself (gates, valves,…) is being influenced, but the river flow rate from the dam instead. With regard to model inputs, this measure relies on rain forecast exclusively. A specially developed algorithm for the operation is presented and subsequently tested off-line with different semi-virtual catchments.

The uncertainty associated to nowcasting (short term forecasting of rainfall) relies on various different factors and is not directly quantifiable. These aspects are not incorporated in the operational scheme where the forecast of rain has been assumed to be perfect using historical rain series as input.

The different scenarios were evaluated for possible ecological limitations of the measure and the associated costs. In contrast, increased CSO volumes were found to be an inappropriate alternative measure, leading to unrealistically large volumes and consequently costs.

Although uncertainties in the rain forecast are case specific, it is discussed that they might be a bottle neck to the measure. Limitations for morphological impacts suggest to apply the measure in interaction with others in the system so as to avoid extreme cases.

7.1.6 INTERPRETATION AND EVALUATION OF RESULTS

Common to all model applications is the need for interpretation of results. Simulation results (time series of flow and pollutant concentrations) require to be analysed to enable judgement on the situation. Criteria can be classified in legal criteria and non-legal related criteria.

Legal criteria may include single limit concentrations as well as reoccurrence interval calculations. This type of criteria can be applied to judge the current situation but also to judge the improvement due to an applied measure. Non-legal criteria can – due to absence of a defined limit – only be used for describing the degree of improvement.

A possible scheme was presented to deal with the application of evaluation criteria in the frame of scenario analysis. The proposed system is based on the derivation of an index system using criteria being scaled to unity. Two indexes, Violation Index(VI) and Improvement Index(II) are proposed describing the degree of violation or improvement for a specific scenario and criterion. A further simplification to combined indexes was made to provide more condensed information. Simplicity in the final output is traded for a loss of information. Weighing of different criteria within this process is responsible for warping original information as well. Thus, no matter how information is simplified, it is very important to provide clarity on “how it is done”.

7.2 OUTLOOK

Typically, it is not necessary to model the whole variety of effects on the receiving water but to focus on a few dominating ones. Only pollutants and processes that have a direct and significant influence on the selected impacts need to be described quantitatively, whereas all other processes can be neglected. Hence, pragmatism is required to avoid unnecessary complexity of the models used. This is of special importance with regard to software applications, which should be adaptable for the relevant processes and not stick to complexity.

7.2.1 INSIGHT IN MODELS AND THEIR DAILY USE

Currently integrated modelling is frequently applied in the scientific community, but to a lesser extent in the engineering practice. At the same time a number of regulations and standards implicitly state a demand for modelling in their application. It is to be hoped that in the near future, the situation may improve such that integrated modelling is more frequently used, not only to fulfil legal requirements but as well to utilize the optimization potential of the tools.

Next to open source products as CITY DRAIN a variety of commercial products are available. These products have a constantly growing amount of features going towards all-one-products. With this increase of features the implemented models increase accordingly. Not Therefore it is discussable whether it is less time consuming to implement what is needed as it is the case for open source or to find out what is implemented.

Bottom line in the daily application of software and models is that the user must be aware of

(a) What model is adequate?
and
(b) Where are the limits of the used model ?

Premise is to use “adequate” models that are “as complex as required”. Future tasks for the scientific community arise therefore, especially in the educational sector. Education of students is one way to avoid the blind use of models by the next generation of engineers. Providing a comprehensive guidance for practitioners in the form of guidelines and lectures is another way to promote integrated modelling.

7.2.2 POST-PROCESSING OF SIMULATION RESULTS

A future task to be approached in the course of a further development of CITY DRAIN is the post-processing of output data. Time series such as simulation results are to be evaluated with regard to different types of criteria. These can be immission criteria as well as emission criteria, based on legal requirements or not. Currently, such evaluations require the user’s knowledge of Matlab to post-process results, where types of evaluation may range from simple annual loads to more complex statistical evaluations.

It is not intended to make software modifications within CITY DRAIN in order to keep it as an open platform. As there is a variety of possible evaluation methods, a modular and extendable library is intended rather than a closed software product. Additional methods may result from divers legal constrains in different countries, or future changes in limits or legislation.

7.2.3 QUALITY OF CALIBRATION FOR DIFFERENT MODELLING AIMS

A number of indicators for assessing the calibration quality were presented. The investigation of the interaction or correlation of indicators showed that they do not necessarily correlate under all circumstances. In fact, the correlation may change when improving one of the quality indicators. This was shown for the correlation of indicators E and d. The correlation was different for simulations that were optimized for the bias or not.

As the Bias relates strongly to mass balances, this situation is certain to have when calibrating. Thus it is worthwhile to put some effort in the future on investigating correlations and reliability of quality indicators.

Extrapolating this thought reveals a number of unanswered questions. The obviously inconsistent interaction between the Bias *(B)* and other calibration quality indicators (*E, d*) suggests that similar effects are the case with regard to what type of parameters are used for calibration. Differences in the calibration are expected when either calibrating for a single annual value such as the total overflow volume or for event-wise parameters such as peak flows.

A model calibration that is specific for the aim of the study would be required. Recalling that data is often of poor quality or rarely available, calibration is limited with regard to measured parameters. Thus, the following question raises:

> *"How reliable is a model calibrated for measurement A when using it for predicting parameter B?"*

To determine whether a model is well calibrated for a specific purpose the link between different parameters, respectively their calibration quality, should to be investigated. In absence of any linkage between different calibration parameters, one relies on the proper assessment and estimation of model parameters. Consequently the question raises

> *"What are the important parameters for different model applications?"*

To answer these questions in a structured way it is worthwhile to initiate investigations in this area.

7.2.4 UTILISATION OF RAINFALL FORECAST IN URBAN DRAINAGE

The work presented as Appendix C and D was based on MBPC using rain forecast as exogenous input. The rain forecast used therein was virtual and assumed to be perfect.

The instrumentation used (Rain gauges, radar,…) and methods applied for forecasting (Numerical weather prediction, Nowcasting, Extrapolation techniques, etc…) influence the quality of the forecast. Further the location, the topography and the size of the catchment have a strong influence, as well as the time horizon required for the forecast. Uncertainties associated with a forecast horizon of *T*+30 min can be found in (Pierce *et al.*, 2004), where various systems/methods were tested at the Forecast Demonstration Project (FDP), held in Sydney, Australia, during 2000. Uncertainties quoted for the rain volume are in the range of 5-10 % (mean square error-MSE) where for rain intensities 45-75 % (MSE) are noted. For larger forecast times, up to 3 hours, larger uncertainties are to be expect. up to some hundreds of percents according to e.g. (Golding, 1998; Golding, 2000).

As uncertainties associate with rain forecast are highly variable, it is of interest to learn how this uncertainties influence the final evaluation of the objective function defined. Uncertainty levels associated to the forecast itself are not necessarily directly transferred to resulting uncertainties in the catchment flow/pollutant flux dynamics. Depending on the system characteristics and on the evaluated variable in the system uncertainties may be reduced down to a certain level. Thus, depending on the case and the specific aim, rainfall forecast can still be utilizable as input to a Model-based predictive control scheme.

The evaluation of forecast quality and the use of precipitation forecasts for urban hydrology is growing, but a structured assessment is currently missing. It is questionable if there is a fully structured way to point out under what circumstances rainfall forecast is utilizable. This is due to a large number of factors influencing the forecast quality as well as the urban drainage hydraulics. A method that is commonly applied to overcome the abundance of influencing factors is the use of case studies. A thorough description on boundary conditions and obtained results helps to judge whether the use of rainfall forecast is feasible for a specific case. Thus, intention is to give guidance in this area and provide a basis for decisions by operators and engineers.

8 LITERATURE

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