Comparison of sewer modelling approaches within IUWS modelling

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by

Bastian Kamradt

Technische Universität Darmstadt
Institut für Wasserbau und Wasserwirtschaft
Fachgebiet für Ingenieurhydrologie und Wasserbewirtschaftung

Supervisor: Dr. -Ing. Dirk Muschalla

Université Laval
Département de génie civil / model*EAU*

Supervisor: Prof. dr. ir. Peter A. Vanrolleghem

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Bastian

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1. Introduction

Wastewater management has the task to transport the storm- and wastewater out of the cities without harming the population and the ecological status of the receiving water. It has been developed and improved over a long period of time.

Methods for sewage disposal already existed in ancient times. Often drainage systems to transport stormwater out of the cities were built to avoid flooding. The drainage systems of the Romans were also constructed to transport wastewater out of the city. In medieval times, these systems were neglected due to the fact that many people moved out of the cities and lived in provincial settlements, so that water infrastructures were considered as unnecessary. The people threw their solid waste and wastewater into gutters in the street, which were flushed away by stormwater. As the population density in the cities grew again, the bad hygienic conditions provoked epidemics. During the industrialization in the middle of the 19^{th} century the first combined sewer system was constructed in London. The wastewater was directly conducted into the river Thames. Thus, the wastewater problematic was shifted from the streets to the receiving water body, whose ecological condition was degraded. At that time the relation between sewage disposal and epidemics were recognized. At the beginning of the 20^{th} century, when the conditions in the receiving water became unbearable, wastewater treatment plants were developed. The sewage was treated in these plants by percolating filter processes. In 1910 the first activated sludge process was applied to treat wastewater. This process significantly improved the receiving water quality and has been further developed to the present day.

Nowadays, urban wastewater management has to deal with design optimization and control of the sewer and wastewater treatment system with the goal of reaching the increased requirements of the water quality in the receiving water body. Measures that can be realized to improve the performance of the urban wastewater system can be classified in the categories source control and end-of-pipe solution.

To the former category belongs the decrease of incoming rainwater by reducing the degree of impervious surfaces. The dry weather flow reduction by reduced water consumption through for example high water prices or a mentality change in the population to save water is another measure.

Measures like the construction of storage tanks in the sewer system or the real-time-control (RTC) of the sewer system and/or the treatment plant fit in the latter category.

In the context of the European Water Framework Directive, which passed in 2000, a holistic approach of all urban systems (sewer system, wastewater treatment plant and receiving water body) is imposed. Therefore integrated modelling will be useful to help extensively improve the water quality by optimization of the urban wastewater system.

In February 2007 the fourth assessment report on climate change was accepted by the IPCC (Intergovernmental Panel on Climate Change). It states that the mean temperature is rising. The impacts resulting from this fact will be more extreme weather events, i.e. the likelihood rises that more floods and droughts will occur. Hence, better water management will be required to provide sufficient potable water with adequate quality while facing scarcer water resources. Also, the urban wastewater system will have to be capable of dealing with increased amounts of stormwater.
To deal with all the above mentioned pressures more and more research focusing on the development of urban wastewater systems needs to be done.

Meanwhile modelling tools are being used to assess, optimize or design wastewater systems and can therefore be used to achieve the objective of water quality improvement. These models are representing the different subsystems of the urban wastewater systems and their interactions with each other in an integrated way and are able to compute water flow and quality. With the aim of improving their accuracy and reducing the calculation time, they are continuously being enhanced.
The processes in the urban wastewater system including urban runoff, water transport, wastewater treatment and river water quality processes are represented in mathematical models for the subsystems. With field data describing the individual characteristics of each of these subsystems, these models have to be calibrated.

This thesis focuses mainly on modelling the sewer system within an integrated view of the urban wastewater system. It is a follow-up study of the PhD of Anne-Marie Solvi (2007) that focused on integrated urban wastewater systems. Within this PhD the KOSIM model, which is a hydrological based sewer model (ITWH, 2000), was implemented into the simulation software WEST® developed at the Biomath department of Ghent University (Belgium) in collaboration with MOSTforWATER N.V. (Kortrijk, Belgium).
The aim of this thesis is the evaluation and the further development of the KOSIM-WEST® implementation. Therefore SMUSI, another hydrological sewer modelling software, developed at the IHWB department of Darmstadt University of Technology is used to identify similarities and differences with KOSIM-WEST®. First the theoretical approaches behind both hydrological models are compared with each other. By using a case study the results of the two models will be compared and judged. Moreover the results of the hydrodynamic sewer modelling software SWMM developed by the U.S. environmental protection agency (EPA) will be the reference. The main focus of this thesis lies on the water transport process and the question whether it is possible to take backwater effects into account in a hydrological sewer model. To this end the results of the conceptual backwater models included in KOSIM-WEST® and SMUSI will be evaluated on the SWMM-results and the mode

of operation of these backwater models will be illustrated. Another aim is to ease the implementation of the water transport submodel in KOSIM-WEST®, as it now requires the use of an external Excel-sheet, what complicates the model development process.

2. Integrated modelling of the Urban Wastewater System

2.1. The EU Water Framework Directive (2000)

The Water Framework Directive of the European Union (WFD), which was approved by the European parliament on the 23rd October 2000, is a directive concerning the protection of water. It commits the member states of the European Union to achieve a good qualitative and quantitave status for all water bodies, i.e. surface waters and groundwater, by 2015. This affects the chemical, biological, physical and ecological status of the water body. For instance the good ecological status is defined locally as a condition without any human pressure, i.e. no anthropogenic influence.

The Water Framework Directive considers the whole water cycle and introduces integrated river basin management. For this purpose the WFD uses a combined emission-immission based approach to set limit values for the effluents of the sewer system and the wastewater treatment plant (WWTP).

The emission-based approach sets limit values at the source of the discharge, e.g. the pollutant concentrations in the effluent of the WWTP are limited, e.g. (CEC, 1991, 1996). In this approach the properties of the receiving water body are neglected to determine these limit values.

In the immission-based approach the environmental quality standards of the water body, which take the natural capacity of the water at the point of discharge into account, serve to set the standards for the treatment of wastewater.

When applying this emission-immission based approach it is necessary to consider all elements of the urban wastewater system (sewer system, WWTP and river) together in an integrated view. For instance, if these elements are not considered together the reduction of direct discharges from the sewer system into the river can lead to a hydraulic overload of the WWTP. This would decrease the plants efficiency and so negatively affect the water quality in the river. The integrated approach increases the number of degrees of freedom to optimize the urban wastewater system and is therefore a motivation to create an integrated model with the aim of more effectively evaluating and optimizing the performance of the urban wastewater system (see section 2.3).

2.2. The integrated urban wastewater system

In view of integrated modelling the structure of the integrated urban wastewater system (IUWS) is classified in the following elements, which are linked with each other: the urban catchment, the sewer system, the wastewater treatment plant (WWTP) and the river. The stormwater runoff as result of rainfall along with the wastewater produced by residential and industrial areas plus the infiltration water flow together from the urban catchment in the sewer system. Pipes transport the wastewater from the sewer system to the treatment plant. Stormwater tanks are installed in the sewer system to store water during rain events, so that the WWTP is not hydraulically overloaded by a too high inflow. Whenever the storage capacity of these retention basins is reached through a heavy rainfall all additional inflowing water is spilled directly into the river. Another source for direct discharges of untreated water are combined sewer overflows (CSOs), which are placed in the sewer system to reduce the amount of transported stormwater. The wastewater is purified in the WWTP by passing physical, chemical and biological treatment processes before the treated water gets discharged into the river. In an emission-based approach certain limit values for the pollutant concentrations in the effluent have to be met. Contrary to this, these values are determined by the water quality of the river in an emission-immission based approach, as described in section 2.1. Further inflows into the river are the natural runoff from rural areas and the runoff from diffuse sources like agricultural areas. The IUWS is illustrated in Figure 1.

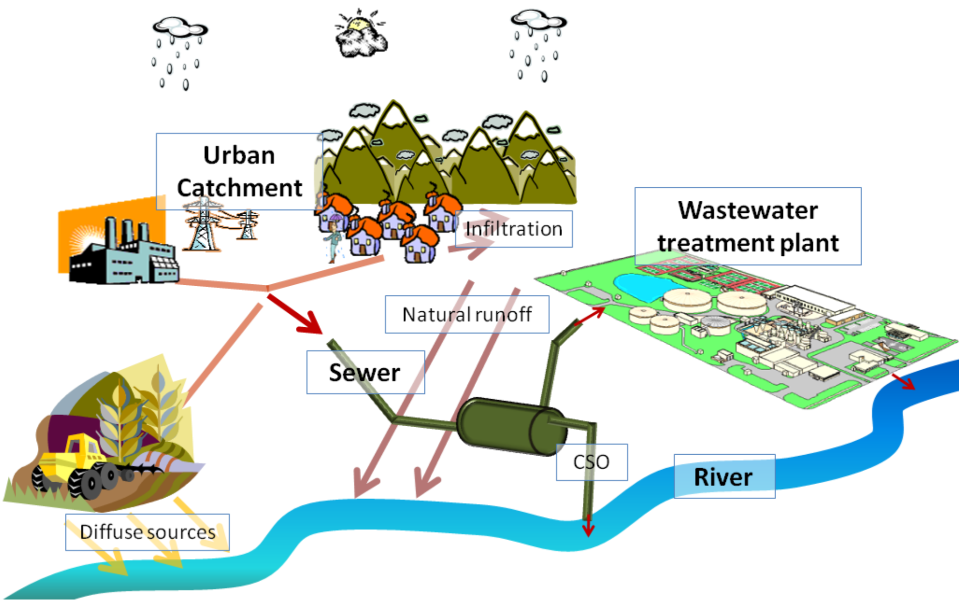


Figure 1: Illustration of the integrated urban wastewater system (Köhler, 2007)

2.3. Integrated modelling

The fact that each receiving water body has its own physical, chemical and biological properties makes that the wastewater management cannot rely on inflexible emission standards for the sewer system and the wastewater treatment plant (WWTP). It is rather necessary to evaluate the quality of the receiving water individually including the relation to the effluents of the sewer system and the WWTP. This combined emission-immission based approach is described in section 2.1. Thus, the urban wastewater system has to be regarded as an integrated system (see section 2.2), whose performance evaluation is driven by the water quality in the river. The dynamic interactions between the elements of this integrated system (sewer system-WWTP, sewer system-river via overflows, WWTP-river) make it difficult to assess the performance of the urban wastewater system. Hence, it is reasonable to build an integrated model representing these dynamics and elements in order to run virtual experiments (e.g. simulations).

The goals for the application of integrated models are manifold. This includes scenarios to evaluate future impacts, e.g. the increase of impervious surfaces or the population density in the urban catchment, as well as the assessment of measures to improve the performance of the system like extending the storage volume in the sewer system. Other objectives, why integrated models can be used, are the evaluation of operation strategies as the implementation of immission-based real-time-control (RTC) and the design of WWTPs.

At present a lot of software packages (KOSIM, SIMBA, MATLAB/SIMULINK, WEST®, etc.) are available, which come with several submodels to represent the elements of the integrated urban wastewater system and to give answers to precise questions. The difficulties in the creation of an integrated model lie in the complexity of the system and the selection of appropriate submodels by the user. Thereby, the objective of the case study and the availability of data influence this choice, i.e. which elements need to be considered and to what level of detail. The general principle in integrated modelling can be summarized in the statement that the submodels and the whole integrated model are supposed to be as detailed and accurate as necessary but as simple as possible (Muschalla, 2006).

3. The Urban Drainage System

The urban drainage system can be subdivided in the subsystems atmosphere where the precipitation is formed, the drained surface, i.e. the catchment and the sewer network where the water and the pollutant loads are transported in. By passing these subsystems the water volumes as well as the pollutant loads undergo certain gains and losses which can be described by several processes, which are illustrated in Figure 2.

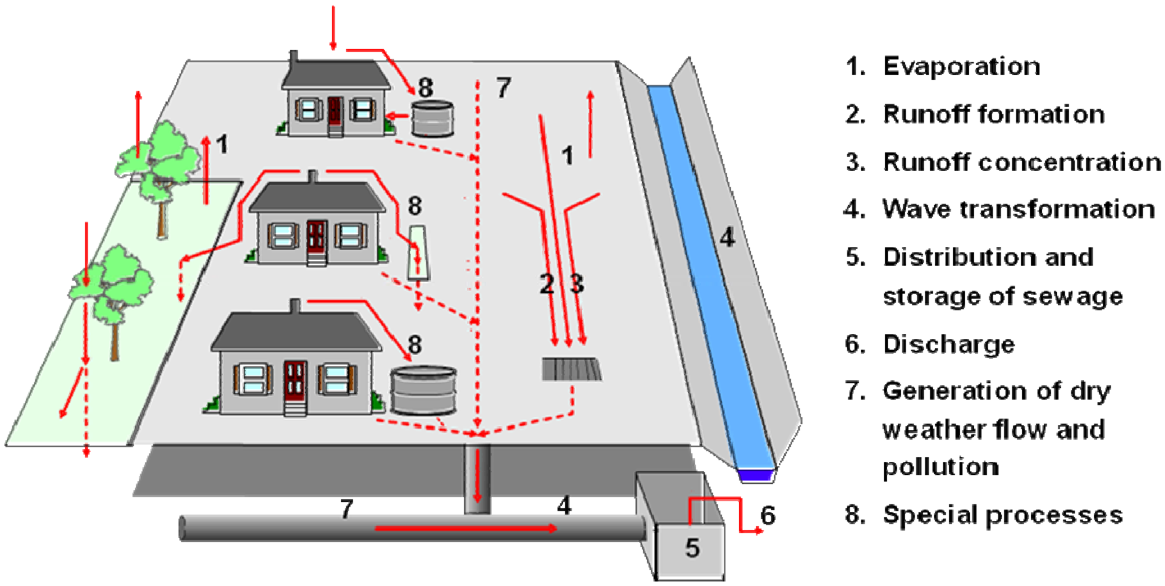


Figure 2: Processes in the urban drainage system with combined sewer network (Muschalla, 2007)

The two main driving forces of the rainfall-runoff process are the rainfall and the evaporation. After the rainfall has reached the surface, first the soil gets wetted and the depressions on the ground are filled up. On pervious surfaces also infiltration of rainwater into the soil is taking place. These processes are summarized under the term runoff formation. The remaining amount of water is called effective rain which washes off particulate matter that has been accumulating on the surface during a dry weather period.

In the runoff concentration phase the effective rain flows together on the surface and then runs off over the surface until it reaches the entries of the sewer network.

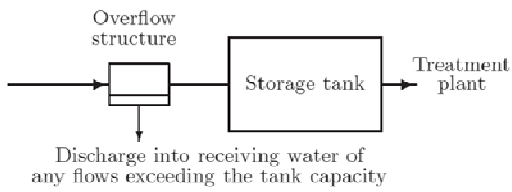
The transport process of water and pollutants inside the pipes of a sewer system or an open ditch is called wave transformation. During this process pollutants also settle and resuspend and biochemical transformations are taking place.

The dry weather flow and pollution is a result of the wastewater discharged by domestic residences, commercial properties, industry and/or agriculture.

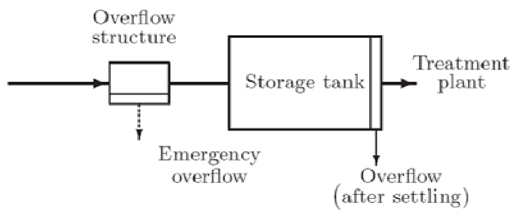
One can distinguish between two different kinds of sewer systems, the combined and the separate sewer system. In the former stormwater and wastewater are mixed and transported in one sewer. The latter consists of two separate sewers, the sanitary sewer in which the wastewater is transported and the storm sewer by which the stormwater is conducted directly to the river.

Because it is unfeasible to provide enough storage capacity in the sewer network during rain events, combined sewer systems are equipped with combined sewer overflow (CSO) structures. These structures are designed to discharge all water, which exceeds the sewer capacity directly to the river.

The water and the transported pollutants are combined, distributed or stored inside the sewer system. Figure 3 shows two different kinds of stormwater tanks, which can be found in a combined sewer system. These tanks can be placed in-line, i.e. all wastewater flows through the tank, or off-line, i.e. only the excess water enters the tank, inside the sewer system.



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



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

Figure 3: Two kinds of stormwater tanks (ATV, 1992)

The by-pass tank (BPT) shown in Figure 3(a) is designed to retain the first flush in the beginning of the runoff, which contains water with highly concentrated pollutants. Therefore it is mostly found in sewer networks with a flow time less than 15-20 minutes. The excess water does not flow through the tank and gets discharged by the overflow structure located upstream of the tank. The pass-through tank (PTT) in Figure 3(b) is built in catchments with a larger flow time where pollutant concentrations are more distributed over time and first flushes are less important. These tanks are supposed to treat the sewage mechanically by settling of suspended solids. Contrary to by-pass tanks the excess water will pass through the tank before being discharged and the overflow to the river is treated mechanically. An emergency overflow is located upstream of the storage tank to limit the maximum inflow to the tank (ATV, 1992). Special processes like greywater reuse or specific infiltration of rainwater from roofs serve to reduce the amount of water entering the sewer system. In the following chapter these processes are described in more detail and the way they are modelled in KOSIM-WEST® and SMUSI is described.

4. General comparison of the models KOSIM-WEST® and SMUSI

In this chapter the theoretical approaches behind the two sewer-models KOSIM-WEST® and SMUSI are compared. Thereby the KOSIM manual (ITWH, 2000) and the PhD by Solvi (2007) are the references for the part concerning KOSIM-WEST®. The part about SMUSI refers mainly to the SMUSI-documentation (Muschalla et al., 2006) and Muschalla (2006).

KOSIM-WEST®

The KOSIM-WEST® model is derived from the KOSIM modelling tool (ITWH, 2000, Paulsen, 1987) and has been implemented into WEST® by Meirlaen (2002) and Solvi (2007). It is designed for long-term simulations of dry weather generation, rainfall-runoff from the surface and transport in the sewer system. With this software it is possible to evaluate both water quantity (flow) and water quality (pollutant loads) inside the combined sewer system and its effluent going to the WWTP or its overflow leaving directly into the receiving water. Beside water the model contains each of the following components in particulate and soluble fractions: chemical oxygen demand (COD), nitrogen and phosphorus. The model is able to simulate pollutant loads of these components in response to individual rain events. The aim of the translation from the KOSIM model to WEST® has been to create a tool to simulate the flow and pollutant loads of the urban drainage in an integrated view (see chapter 2), i.e. also including WWTP and river. To this end, the discrete timestep equations behind the conceptual KOSIM modelling tool had to be transformed to the original underlying ordinary differential equations so that they can be combined with other submodels of the IUWS and numerically solved by the solvers contained in WEST®. The results given back have a continuous evolution. Simplifications of the model are that no evaporation is taking place during rain events, pollutants stem only from impervious surfaces, infiltrated water is clean and surface flow times are the same for pervious and impervious surfaces. Figure 4 illustrates the division of the sewer model into a catchment model and a sewer transport model which are linked together and shows the several processes which have been modelled in KOSIM-WEST®.

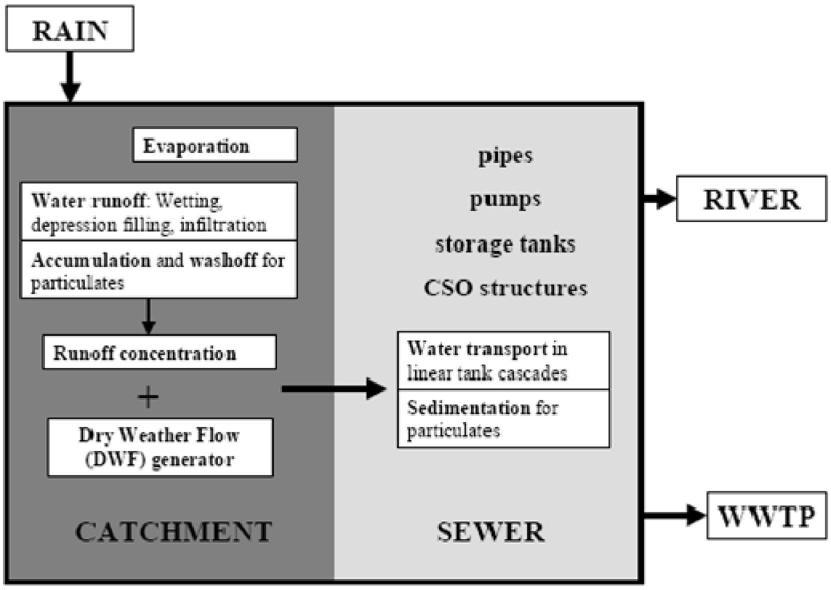


Figure 4: Elements and processes within the KOSIM-WEST® sewer model (Solvi, 2007)

SMUSI

The version of SMUSI used in this study is the research version SMUSI 5.0 (Muschalla et al., 2006). It is a detailed hydrological deterministic rainfall-runoff and pollution load model which is based on discrete time step equations. It simulates the dominant characteristics like pollutant loads, amount of discharged water, duration and frequency of discharge, which are needed for the assessment of the effect of overflow structures on receiving water bodies. The simulated processes include runoff formation and concentration from pervious and impervious areas, superposition of dry weather flow and stormwater runoff in collecting pipes and structures as well as translation and retention of hydrographs and pollutographs in the sewer system (Muschalla et al., 2007).

SMUSI contains the pollutant components TSS (total suspended solids), BOD (biological oxygen demand), COD (chemical oxygen demand), TOC (total organic carbon), ammonia (NH_4-N) and orthophosphate (PO_4-P).

4.1. Urban catchment

The urban catchment is composed of all wastewater producing units with their local sewer system and their surrounding area which are connected to the sewer system. The catchment's outflow which enters the main collector consists of rainwater from surface runoff and municipal wastewater produced by households, commercial and/or industrial areas. Figure 5 illustrates the different submodels which are part of the urban catchment model in KOSIM-WEST®. It contains a potential evaporation model, a runoff formation model which takes several losses of the rain on the surface into account, a runoff concentration model which represents translation and retention of the runoff hydrograph and a model to generate the dry weather flow (DWF) which is superposed with the runoff originating from the rain.

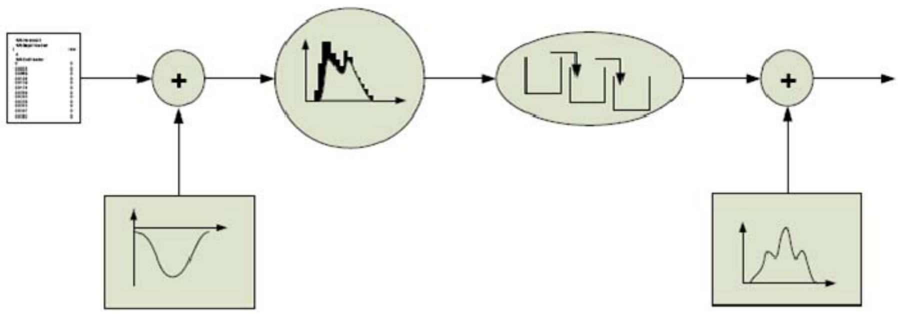


Figure 5: Submodels within the catchment model in KOSIM-WEST®: rain as input, potential evaporation model, runoff formation model, runoff concentration model and a dry weather flow generator (Solvi, 2007)

4.1.1. Rain and Evaporation

The rain input data in KOSIM-WEST® has to be specified in an external file in terms of a simple time-rain vector using any time interval the user wants. It can be chosen whether the values between two timesteps are interpolated or the value of the last timestep should be used until the next time instant where data is given. In SMUSI rain series are used as input data and get time discretized. The time interval should be as small as possible and is fixed to 5 minutes in SMUSI so that the outflow processes inside the sewer system can be described sufficiently exact. In both models the rainfall is uniform all over one subcatchment but a different rain input can be defined for each subcatchment of the drainage area.

Evaporation varies with the season and the time of the day and also depends on the mean annual evaporation which is different for every region. To calculate the potential evaporation of a certain time instant in both models the evaporation for a specific day is multiplied by an hourly factor. The potential daily evaporation is determined using a sinusoidal distribution with high values in summer and low ones in winter. The evaporation recovers storage capacities for wetting, depressions as well as the infiltration capacity during dry periods and reduces the rainfall. The latter is neglected in KOSIM-WEST® because these losses were considered small compared to other losses.

4.1.2. Runoff formation

During the runoff formation the rainfall is reduced by wetting, filling of depressions and infiltration into soil. The latter occurs only on pervious surfaces. The amount of the remaining rain water depends mainly on the connected area to the sewer system, the ratio of impervious and pervious surfaces as well as on the characteristics of the soil. The runoff water also transports pollutant loads.

The Soil-Conservation-Service (SCS) method, which is used for pervious surfaces in SMUSI, represents the whole runoff formation process including wetting, infiltration and depression fillings. The characteristics of the soil such as the soil group and land use are represented by the curve number (CN). Depending on the antecedent moisture condition expressed by a 21 days pre-rain index and the CN-value, the runoff coefficient ψ at the beginning of a rain event is calculated. With increasing rainfall the runoff coefficient also increases, which is taken into account by its dependency on the accumulated precipitation during a rain event. For each time step the effective rain can be calculated by multiplying the incoming rainfall with the current value of the runoff coefficient $\psi(t)$.

Wetting losses are modelled by a maximum wetting storage in KOSIM-WEST® and in SMUSI. The rain is reduced by the free space available in this storage. On pervious surfaces the rain gets also intercepted by vegetation what again depends on the time of the year and also on the type of vegetation. Therefore in KOSIM-WEST® the maximum wetting loss for pervious surfaces is higher than for impervious surfaces and also a seasonal reduction factor is implemented.

As soon as the wetting storage is filled on pervious surfaces the amount of rainwater is further reduced by infiltration into the soil. The infiltration capacity, i.e. the amount of water that can infiltrate in the soil, depends on the nature of the soil and its moisture content. At the beginning of a rainfall event it will be higher than at the end and the same is true for a dry soil compared to a wet one.

In KOSIM-WEST® the infitration model is based on the time-dependant Horton equations. The approach within these is that the infiltration capacity decreases during a rain event and raises again in dry weather conditions. These changes are considered to be exponential and the current value of the infiltration capacity is determined depending on the weather conditions and lies between a maximum and a minimum infiltration capacity. Finally, the incoming rainfall is reduced by the current infiltration capacity $f(t)$ and the remaining rain can be obtained.

After the infiltration capacity is exploited, the filling of depressions starts. It can be observed that in the beginning of this process the runoff begins although the depressions are not completely filled. The reason for this effect is that the depressions are not uniformly distributed over the surface of the subcatchment.

In KOSIM-WEST® the filling state D is modelled to have an exponential behaviour, i.e. with increasing amount of rain less water can be stored in depressions and so the runoff increases. The initial runoff coefficient ψ_0 serves to obtain a runoff in the initiation of the depression filling process. This process is equal on pervious and impervious surfaces, except that parameter values are different.
In SMUSI the maximum depression loss D_{max} is split in three storages with different size. The runoff begins when the smallest storage is filled, but before the whole storage capacity is covered.

Figure 6 shows all losses taking place during the runoff formation and the resulting runoff from a pervious surface.

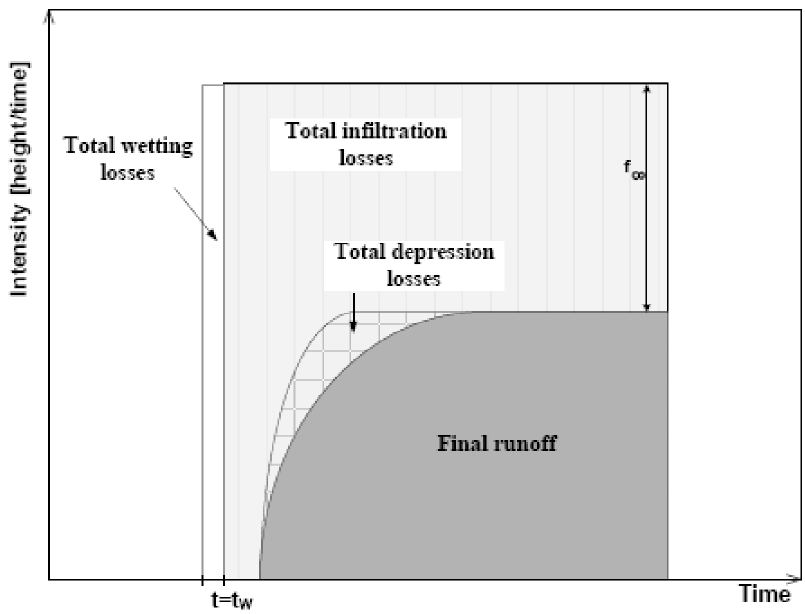


Figure 6: Runoff and losses during the runoff formation on a pervious surface (Solvi, 2007)

The storage capacities for wetting, for depressions and the infiltration capacity recover by evaporation taking place during dry periods. The emptying of the depressions on pervious surfaces through infiltration is neglected in KOSIM-WEST®.

The water remaining after the runoff process transports pollutant loads which originate from the surface. Concerning the water quality it is possible to derive the concentrations of the non-water components from constant stormwater concentrations or from accumulation and wash-up processes. The following simplifications are made in both models: The pollutants stem only from impervious areas and the surface runoff from pervious areas as well as the infiltration water are considered as unpolluted. All pollutants are considered conservative, i.e. no conversion of them inside the sewer system is taken into account. Furthermore, in SMUSI any pollutants accumulated in sewer pipes are included in the surface processes.

The accumulation of particulate matter is taking place during dry weather periods and depends on many factors like urbanisation, traffic, nature of the street, particle size, etc. In KOSIM-WEST® the increase of the accumulated mass is modelled to rise linearly by an accumulation rate. This is only implemented for the particulate COD component and all other components are set in relation to it. An exponential increase of the accumulated mass which converges asymptotically to a maximum accumulation potential is underlying the accumulation process in SMUSI.
In wet weather conditions the accumulated solids get washed off from the surface with the runoff water. In both models the wash-off process depends on the mass available on the surface and the rain intensity and is modelled to decrease exponentially. The latter is taken to the power of a shape factor ω in SMUSI ((Muschalla et al., 2007)).

4.1.3. *Runoff concentration*

During the runoff concentration the effective rain flows over the surface and through the local sewer system whereas the peaks of the hydrographs and pollutographs are changed by the phenomena of translation and retention.

This process is described in KOSIM-WEST® by a linear tank cascade with n = 3 tanks in series. In each tank the model of a linear reservoir (see section 4.2.1), which consists of a continuity equation and a linear volume-outflow relation, is applied. Hereby the outflow of tank n-1 is the inflow of tank n. The parameter k represents the relation between volume and outflow of each reservoir. It is the residence time of the water in one tank. With the concentration time t_c, which is defined as the time the rain needs to flow from the farthest point in the subcatchment to the entrance of the main collector, the parameter k can be estimated to:

$$k = 0{,}25 \cdot t_c \qquad (1)$$

This approach is extended in SMUSI for canalized areas with another three-linear tank cascade which is arranged in parallel to the above described cascade (see Figure 7). The split-up factor β which distributes the inflow to the two cascades as well as the retention constants k are determined internally depending on the characteristics of the subcatchment. Herewith slow and fast draining areas are represented. For outer areas which discharge the surface runoff over rills into the sewer system a three-cascade model, each with two tanks in series, is used.

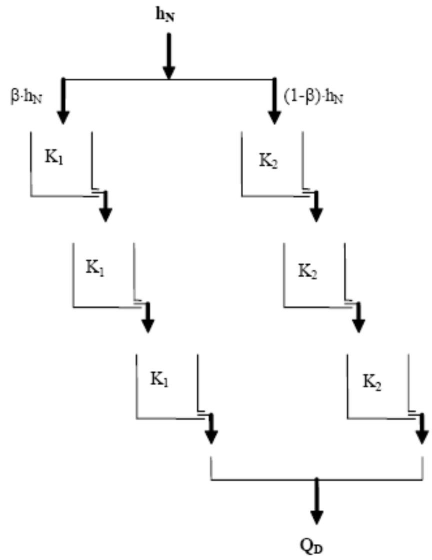


Figure 7: Scheme of the parallel tank cascade for canalized areas in SMUSI (Muschalla et al., 2006)

4.1.4. Dry weather flow

The dry weather flow and pollution are a result of the produced wastewater in the catchment. The amount and composition of wastewater mainly depends on the population density along with the size of the catchment, the time of the day and the kind of source it stems from (domestic, industrial or commercial). To represent the varying flow and pollution in both models daily patterns with hourly factors are used, that can be modified by the user. A different distribution on the weekend is taken into account by a weekend factor.

Furthermore, in KOSIM-WEST® all patterns and factors for flow and pollution are independent from each other and can be defined separately. However, it is assumed that the diurnal course is the same for every pollutant. Also a tourism factor is implemented, which represents different water consumption in a special interval in the year according to high tourist activities. To this end the factor is limited by a start and end day in the year. In SMUSI, however, the chosen daily pattern is the same for both the flow and the pollutants.

The amount of infiltrated water, resulting from infiltration into the sewer system, is modelled as a mean flow per connected area. In both models also a yearly pattern can be defined by the user to represent different amounts of infiltration water in winter or summer.

4.2. The sewer transport model

At the entrance of the main collector the hydrographs of the dry weather flow and the stormwater remaining after the runoff process are added and get transformed during the transport. This transformation process is composed of a time translation and a damping of the hygrograph's peaks, the so-called translation and retention phenomena. Furthermore, the water and the transported pollutants are combined, split up or stored inside the sewer system.

4.2.1. Pipe flow

To model the pipe flow one can distinguish between hydrodynamic and hydrological methods of calculation. The physical processes the water undergoes during the wave transformation process in the sewer system are represented by the "Saint-Venant" equations. These are first order partial differential equations, consisting of a continuity equation (2) which describes the conservation of mass and a momentum equation (3) expressing the energy conservation.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{2}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\frac{\partial y}{\partial x} - gA\left(S_0 - S_f\right) = 0 \tag{3}$$

y	= water level [m]	Q	= flow rate [m³/s]
A	= cross-sectional area of flow [m²]	t	= time [s]
x	= distance [m]	S_0	= bed slope [-]
S_f	= friction slope [-]		

In hydrodynamic models these equations are solved numerically and provide information on the spatial (variable x) and temporal change (variable t) of the flow rate Q and the water level y. Herewith, wave attenuation, backwater effects, pressurized flows along with flow acceleration can be modelled. The need for numerical methods to solve the equations leads to long calculation times due to the small time steps required for accurate and stable numerical solutions. To achieve accurate results many data regarding the pipe dimensions, slope as well as location are required and the real sewer system has to be represented in a high resolution.

The approach behind the hydrological calculation methods is that the pipe is modelled as a "black-box" model, i.e. the water transport is described by an empirically determined transfer function. Thus, the physical processes in the pipe are not exactly represented. The continuity equation (2) is replaced by a mass balance (4) and the momentum equation (3) by a linear Flow-Volume relationship (5). The result is an ordinary differential equation, so that the outflow can be computed without using numerical methods. The approach of the commonly used Kalinin-Miljukov method is to consider that the unsteady flow is steady in stretches of a certain length, the so-called characteristic length. Through this the pipe is regarded as a linear reservoir cascade and the outflow of each reservoir can be calculated by using equations (4) and (5).

$$\frac{dV}{dt} = Q_{in}(t) - Q_{out}(t) \tag{4}$$

$$Q_{out}(t) = \frac{1}{k} \cdot V(t) \tag{5}$$

Q_{in}	= inflow [m³/s]	Q_{out}	= outflow [m³/s]
V	= volume in a certain stretch [m³]	k	= retention constant [s]

The required parameters such as the number of tanks n and the retention constant k, which is the time the water lasts in one tank, are determined from the physical properties of the pipe. The hydrological methods only calculate the flow, the influence of the water level on the outflow of the pipe is not considered. For this reason special situations like backwater or pressurized flows cannot be directly taken into account. Due to the simplification of the physical processes the results of the hydrological models are less exact than these of the hydrodynamic methods. In case backwater effects in downstream structures influence the behaviour of the water upstream, i.e. the system is not under ideal flow conditions, the results become inaccurate.

The advantages of these hydrological calculation methods compared to hydrodynamic models lie in the lower calculation times, the calculation stability as a result of the mathematically simpler representation and the lower need of input and calibration data. Mentionable is also the simplicity and transparency of the model structure. Therefore, these models are often used in integrated modelling of the sewer system.

The pipe flow in KOSIM-WEST® is modelled by using the above described Kalinin-Miljukov method. According to the characteristics of the pipe the parameters n and k are calculated in an external Excel file. With this information the user can choose between an individual tank and a tank cascade of up to 10 tanks, all available in the WEST®-modelbase. The user can enter the k-value.

In SMUSI three different calculation methods of the pipe flow are available. The first option only considers the translation effect, i.e. the incoming flow is delayed by the flow time in the pipe. When the second option is selected, the outflow will be calculated with the Kalinin-Miljukov method. For this the diameter, the roughness, the slope and the length of the pipe are needed.

In the third option a non-linear hydrological method developed by Mehler (2000) is applied. The main idea behind this method is that the linear Flow-Volume relationship (5) is replaced by a partially linearized relationship (see equation 6). Figure 8 illustrates how the course of the Flow-Volume-curve (dotted line) is approximated by several straight lines with different gradients k_i, that each are only valid between two specific nodes. In comparison to the Kalinin-Miljukov method the pipe is modelled as one reservoir with a quasi-non-linear relationship between Volume and Outflow. The parameters k_i are determined within the software on the basis of the characteristics of the pipe (diameter, slope, roughness).

$$Q_{out}(V(t)) = Q_{out,i-1} + k_i \cdot (V(t) - V_{i-1}) \qquad (6)$$

Q_{out} = Outflow [m^3/s]

V = Volume [m^3]

k = Retention constant [s]

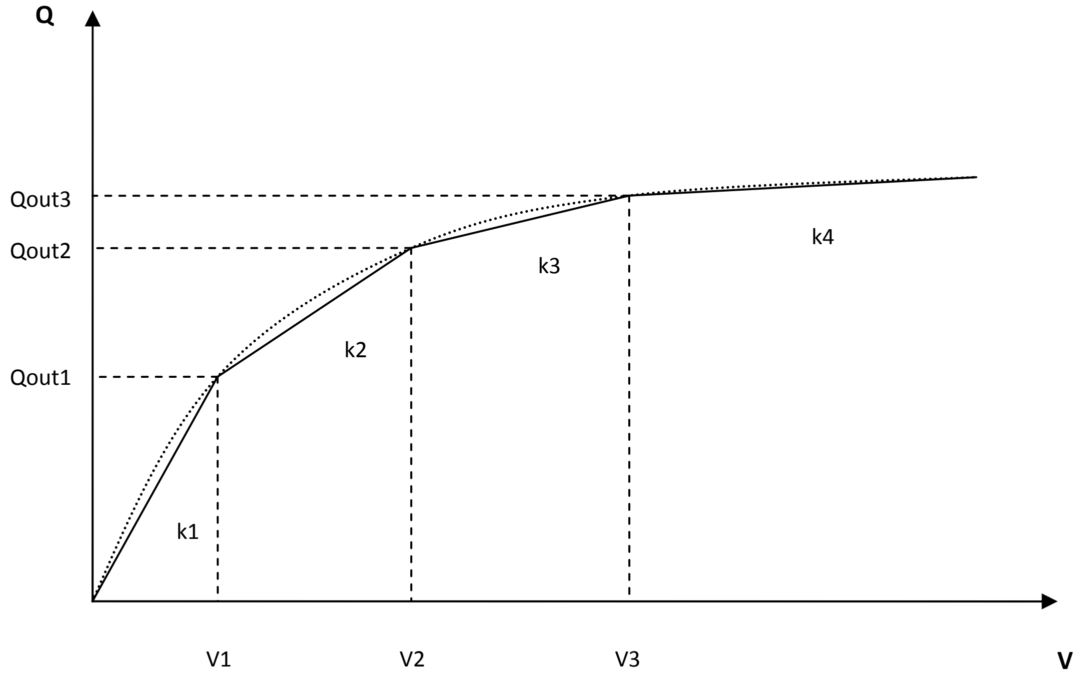


Figure 8: Partially linearized Flow-Volume relationship

Inside the sewer system sedimentation and resuspension of particulate matter are happening. In KOSIM-WEST® the build up of particulate material is considered to be exponential and to take place only below a certain dry weather flow $Q_{DWF,max}$. If this flow is exceeded resuspension of the accumulated pollutants, depending on the available material, is proceeding. The more material is settled the more intense the first flush effect will be.

In SMUSI accumulated pollutants in sewer pipes are included in the accumulation and wash-off processes on the surface.

4.2.2. Backwater model

According to Engel (1994) the term backwater describes the situation that the maximum pipe flow is not sufficient to conduct the incoming flow downstream. The excess water gets stored in the adverse direction of the flow by filling the retention volume that can be activated in the pipe located upstream. Thereby the water head increases which itself leads to a higher flow in the pipe, i.e. pressurized flow. In case there is no more retention volume available and the water head rises above the top ground surface, a flood arises. The reasons for the occurrence of backwater effects can be divided into two categories: operational and system-induced causes.

Examples of operational causes are the presence of sediments in the pipe and a constructional damage. These causes cannot be detected or taken into account by a sewer model because the information about the random appearance of each cause is not available.
Underdimensioned pipes, an obstructing structure downstream as well as a throttle induced backwater belong to the group of system-induced causes. These causes have to be described by a sewer model because especially in a flat sewer system backwater can induce CSO events that have a significant impact on the river water quality. As already mentioned, backwater effects taking place in the sewer system are typically not considered in hydrological models. These models thus tend to overestimate flow maxima due to the non-consideration of the retention volume in pipes lying upstream and the non-recognition of overloaded collectors. In this case the CSO frequency and volume are also assessed too high. Therefore there is a need to upgrade these hydrological models by adding a conceptual backwater model without losing the advantage of fast calculation times.

The backwater model implemented in KOSIM-WEST® (Solvi et al., 2005) consists of a combiner-splitter combination which is located on top of the tank cascade representing the pipe (see Figure 9). The splitter only allows the set maximum outflow capacity Q_{back} to flow to the collector downstream and any excess water is set back to the upstream combiner. The combiner adds the incoming flow and the backwater.

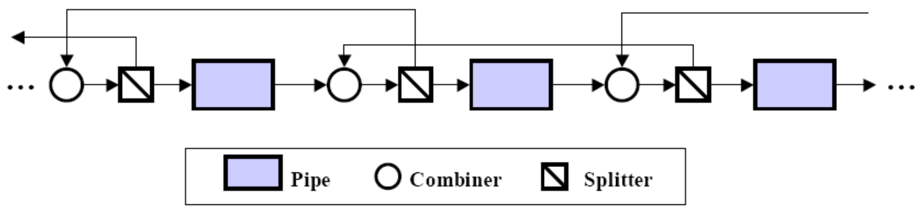
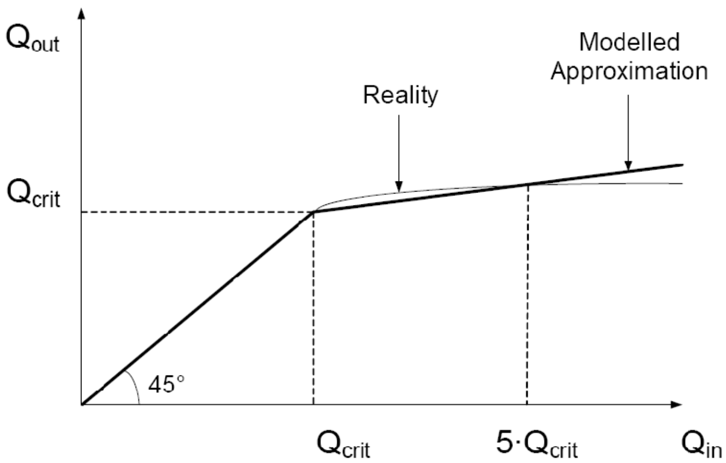


Figure 9: Backflow model implemented in KOSIM-WEST® (Solvi et al., 2005)

The approach behind the simplified consideration of backwater effects in SMUSI is that the backwater level in the sewer system, provoked by a rainwater retention structure, is regarded as nearly horizontal. Then the retention capacity, which can be activated inside the pipes lying upstream, is determined by a horizontal section through the above lying system. As bottom boundary condition the overflow height of the downstream structure is used. Finally this volume is added to the storage volume of the structure. For this method geometric characteristics of the pipe are needed, so that this backwater model is only available if the pipe flow is calculated by the above described Kalinin-Miljukov or non-linear method (Muschalla et al., 2006).

4.2.3. Distribution

As described in chapter 3, combined sewer overflows (CSO) are installed inside combined sewer systems to reduce the amount of stormwater to be transported by the sewer network. This structure splits the inflow Q_{in} in an outflow Q_{out} and an overflow Q_{over} when Q_{in} is higher than a critical value Q_{crit}. The excess water discharges directly over the overflow weir in the river. With increasing inflow the water head inside the structure rises, which leads to a higher outflow than the critical flow Q_{crit}. This fact is taken into account in KOSIM-WEST® by a linear correction factor δ, as illustrated in Figure 10.



$$\delta = \frac{Q_{out}(Q_{in}=5\cdot Q_{crit})}{Q_{crit}}$$

Figure 10: Definition of the linear correction factor δ (Solvi, 2007)

In addition to this, in SMUSI, it is possible to choose two other calculation methods for the flows of a CSO.

The first of these is the input of user-defined characteristic curves of the inflow and outflow of the structure in relation to the water level. These curves have to be determined from a hydraulic calculation or on the basis of measurements.

The second alternative is the automatic calculation of these characteristic curves according to the ATV A-111 guideline. For this, geometrical characteristics of the structure are needed.

The same methods of calculation can be chosen for any other sewerage splitter structure, where the water stays in the system and gets distributed to two pipes. In KOSIM-WEST® the user has the choice between an absolute splitter and a relative splitter. The former splits the flow in a flow set by the user and all water exceeding this value will be led to the second pipe. The latter divides the flow into two fractions according to a given flow fraction.

4.2.4. Storage

The different types of rain retention basins which can be found in a sewer system are the by-pass tank (BPT), the pass-through tank (PTT) and the storage tank (STT). They are described in chapter 3.

The first option to calculate the flows in a tank in SMUSI is an approximate way of calculation, where only the volume and the throttle discharge, i.e. the maximum outflow, of the tank are needed. When the inflow is higher than the throttle discharge the water gets stored in the tank until its storage volume is filled up. This method equals a distribution of the flow with a splitting correction factor $\delta = 1$ and is a strong simplification.

In reality the outflow and the overflow of a tank are functions of the water level. For that reason the second option uses the characteristic curves of the outflow, overflow and the tank volume in relation to the water level in the structure. For every timestep the water level has to be iterated to calculate the outflow and the overflow of the tank.

In the third option the characteristic curves are calculated internally with the geometrical characteristics of the tank.

In KOSIM-WEST® the outflow of the tank can be calculated in two different ways. The first one is that the outflow has a fixed maximum value, i.e. only if the inflow is higher than this value water gets stored in the tank. This equals the approximate calculation in SMUSI.

Alternatively, the outflow is calculated depending on the water level in the tank, the cross-sectional area of the downstream pipe and the sluice position in the outlet. The tank overflow is calculated with the overflow equation of a rectangular weir.

Concerning the water quality, sedimentation of particulates in a pass-through tank is taken into account by a sedimentation factor in KOSIM-WEST®. If the volume of the tank is below 25 % of the total volume the settled particles will be flushed away.

In SMUSI the sedimentation rate of the total suspended solids (TSS) depends on the sedimentation efficiency of the tank and all other components are related to this rate. These relation values can be adapted by the user.

5. Analysis of KOSIM-WEST® and SMUSI on a case study

In this chapter the KOSIM-WEST® and SMUSI models are compared by using a case study. Here the focus is put only on the quantity of water, especially on the amount of the direct discharges in the river. The water quality, i.e. the concentration of pollutants will not be considered in this comparison. To this end mainly the flows of the rainwater retention basins and especially their overflows will be examined. For all flows the results of the hydrodynamic sewer-model SWMM, developed by the U.S. environmental protection agency (EPA), are the reference. As rain events used to compare the models a continuous rainfall dated 24.04.1968 (see Figure 11) is taken to represent a "normal" rainfall and a catastrophic rainfall dated 15.06.1968 (see Figure 12) represents a "heavy" rainfall, where backwater effects will occur.

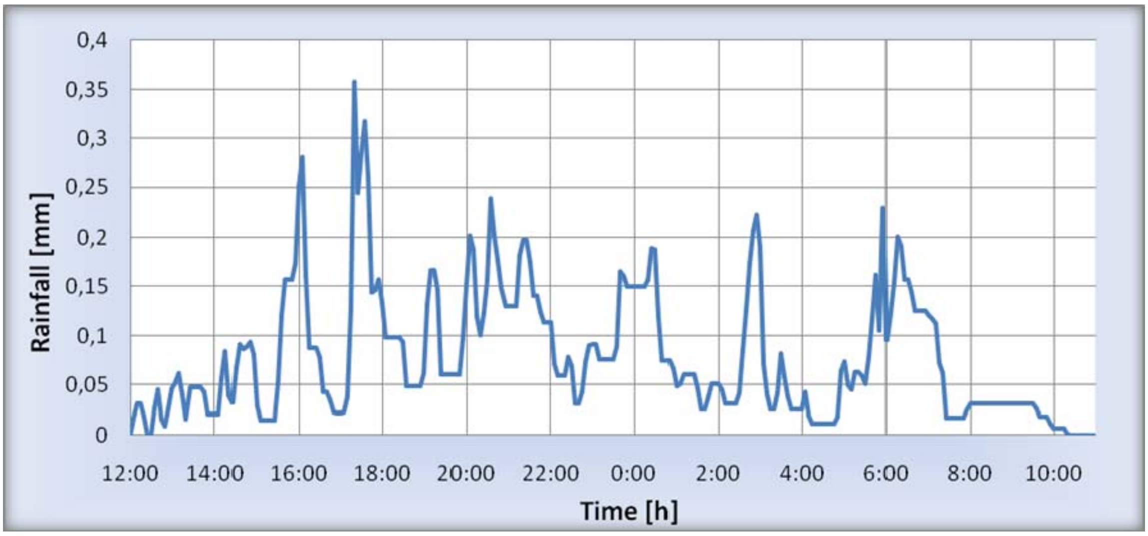


Figure 11: Continuous rainfall on 24.04.1968

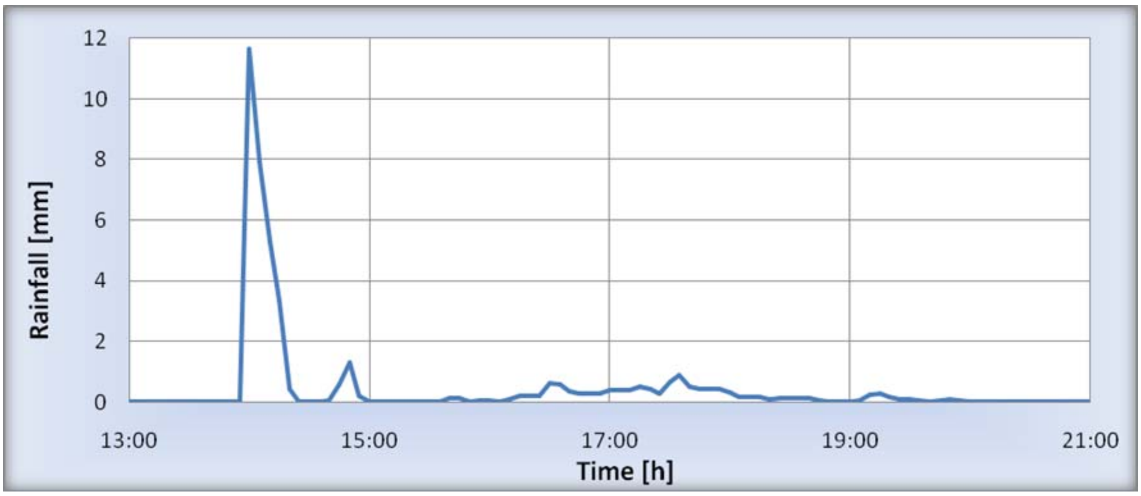


Figure 12: Catastrophic rainfall on 15.06.1968

The aim of this comparison is to evaluate KOSIM-WEST® with reference to SMUSI and detect points of potential improvements. Another interesting point is to compare the different backwater-models of the hydrological models KOSIM-WEST® and SMUSI and evaluate these with the results of the SWMM. This example shall also assess to what extent backwater-effects can be taken into account in a hydrological model and whether this is sufficient in view of integrated modelling.

Table 1 gives an overview of the procedure followed for this comparison. As described in section 4.2.1 it is possible to simulate the pipe flow in SMUSI with both the non-linear method and the Kalinin-Miljukov method (KM). Hence, these two cases were examined. In the first two scenarios both models are compared for the "normal" rain event where no backwater is expected. In the third and fourth scenario the "heavy" rainfall shall cause backwater but the backwater-models are not enabled. With these scenarios the necessity of taking backwater effects into account shall be shown. In the last two scenarios the backwater-models are enabled and the results shall serve to assess the efficiency of the different possibilities to consider backwater effects.

Table 1: Procedure for the comparison between SMUSI and KOSIM-WEST®

scenario	rainfall	SMUSI	KOSIM-	backwater-model
1	"normal"	KM	KM	No
2	"normal"	NL	KM	No
3	"heavy"	KM	KM	No
4	"heavy"	NL	KM	No
5	"heavy"	KM	KM	Yes
6	"heavy"	NL	KM	Yes
KM = Kalinin-Miljukov; NL = non-linear				

5.1. The Case Study

The case study applied in this thesis is taken out of the German guideline ATV-A 128. Figure 13 illustrates the drainage area schematically. It consists of six subcatchments, five of which are drained with a combined sewer system, where wastewater and stormwater are transported together in one sewer. Subcatchment 5 is a drainage area with a separate sewer system, i.e. the wastewater is transported in a sanitary sewer and the stormwater in a storm sewer, which conduits the runoff resulting from rain directly to the river. The sanitary sewer of this area discharges into the main collector of subcatchment 6. A storage tank STT with a volume of 2000 m³ and a throttle discharge of 100 l/s is located after subcatchment 1. Two combined sewer overflows (CSO1 and CSO2) discharge the subcatchments 2 and 3. Subcatchment 4 leads into a by-pass tank BPT with a volume of 180 m³ and a throttle discharge of 12,3 l/s. The outflows of all rain retention basins combined with the wastewater of subcatchment 5, flow in the main collector of subcatchment 6 and from there in a pass-through tank PTT. This tank has a volume of 1200 m³ and a throttle discharge of 98 l/s which corresponds with the inflow of the wastewater treatment plant. The characteristic values of the rain retention basins are summarized in Table 2.

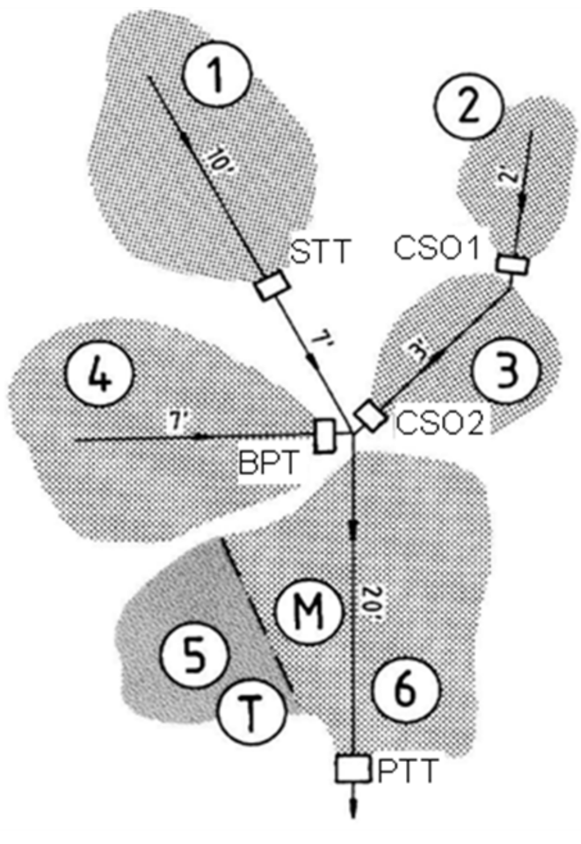


Figure 13: Schematic plan of the drainage area (ATV, 1992)

Table 2: Characteristic values of the rain retention basins

Structure	Volume [m³]	$Q_{throttle}$ [l/s]	δ [-]	A [m²]	h [m]
CSO 1	-	50	1	-	-
CSO 2	-	105,5	1	-	-
STT	2000	100	-	500	4
BPT	180	12,3	-	60	3
PTT	1200	98	-	400	3

CSO = combined sewer overflow
STT = storage tank
BPT = by-pass tank
PTT = pass-through tank
$Q_{throttle}$ = throttle discharge
δ = splitting correction factor (see section 4.2.3)
A = surface of the basin
h = height of the basin

Some simplifications were made for the characteristics of the catchments. Because for pervious areas different runoff models are used in SMUSI and KOSIM-WEST®, as described in section 4.1, these areas have been neglected, i.e. surfaces were assumed impervious. Furthermore, the infiltration rate per connected area i_s is set to 0,1 l/s*ha and is considered as constant over the year. It is also assumed that no infiltration is taking place in the sanitary sewer of subcatchment 5. These simplifications serve to obtain nearly the same amount of water resulting from the catchments and entering the sewer system in SMUSI and KOSIM-WEST®. It is therefore possible to focus on the comparison of the transport models of the pipe flow. The daily variation of the dry weather flow is chosen to be that of a small city (5000 - 25000 inhabitants). The characteristics of the six catchments can be found in Table 3, in which the retention constant k was determined using equation (1).

Table 3: Input parameters of the catchments

Catch-ment	A_{imp} [ha]	NG [-]	PE [IE]	PD [IE/km²]	w_s [l/IE*d]	Q_{pe} [l/s]	Q_i [l/s]	t_f [min]	t_a [min]	k [min]
F1	14	1	2240	16000	180	4,7	1,4	10	3	3,25
F2	3	2	550	18333,3	180	1,1	0,3	2	2	1
F3	4	2	420	10500	180	0,9	0,4	3	2	1,25
F4	10	2	1350	13500	180	2,8	1,0	7	2	2,25
F5	10	-	1100	11000	180	2,3	0,0	-	-	-
F6	35	1	5600	16000	180	11,7	3,5	20	3	5,75

A_{imp}	= impervious area	NG	= notion group
PE	= population equivalent	PD	= population density
w_s	= specific daily water consumption per population equivalent		
Q_{pe}	= mean daily quantity of wastewater produced per population equivalent		
Q_i	= mean daily quantity of infiltration water		
t_f	= flow time in the completely filled main collector		
t_a	= flow time on the surface	k	= retention constant

In the simplified representation of the drainage network in this case study only three main collectors (S1, S3 and S6) are retained. Figure 14 shows the different system elements and how they are linked. This system is used to simulate the scenarios 1 to 4.

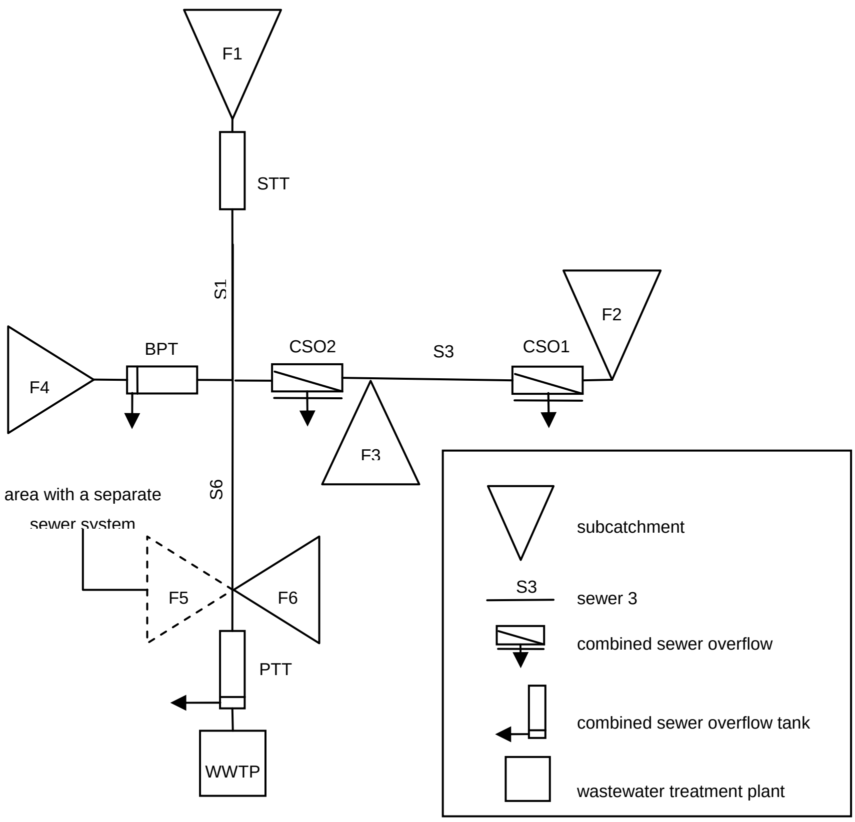


Figure 14: System plan of the roughly subdivided drainage network (System 1)

Table 4 specifies the characteristic values of the main collectors for this simplified system. The columns number of tanks n and retention constant k are needed to build the model in KOSIM-WEST® and are determined in an external Excel-sheet.

Table 4: Characteristic values of the main collectors for the simplified system

No.	L [m]	s [‰]	D [mm]	Q_{full} [l/s]	n [-]	k [s]
S1	500	8	300	88	33	10
S3	140	6,3	300	78	7	14,8
S6	1870	3	900	977	15	66,2

L = length
s = slope
D = diameter
Q_{full} = outflow of the completely filled pipe
n = number of tanks
k = retention constant

With this simplified system it is not possible to test backwater effects, even if the backwater models in SMUSI and KOSIM-WEST® are enabled. The resolution of the drainage area is not detailed enough for this and it is necessary to place a pipe above all rain retention basins. Hence, the pipes S2, S4 and S5 were added to the simplified model. Furthermore, following each point of discharge of all subcatchments a pipe has to be added. Hence, the pipes S3 and S6 were each split into two pipes. This more detailed representation of the drainage network is required for the conceptual backwater models in SMUSI and KOSIM-WEST® to be evaluated in the scenarios 5 and 6. It is illustrated in Figure 15.

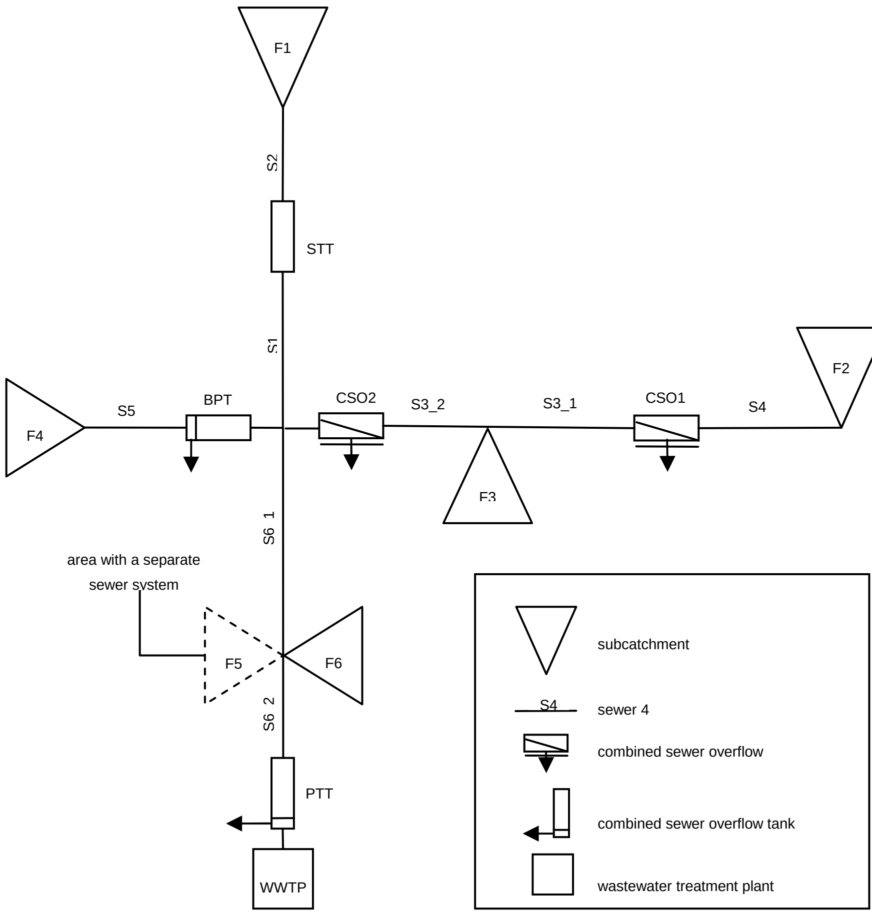


Figure 15: More detailed system plan of the drainage network to test backwater models (System 2)

Table 5 lists the characteristics of all main collectors for this representation of the drainage area of the case study. For each pipe the input parameter Q_{max} is obtained by taking the highest value of the flow from the simulation results of the hydrodynamic model SWMM. This value takes into account that, if a backwater effect occurs, a higher flow inside the pipe than in a completely filled one is possible, because water gets stored in the manhole above the pipe, which increases the water head. This parameter Q_{max} is needed in KOSIM-WEST®.

Table 5: Characteristic values of the main collectors for the more detailed system

No.	L [m]	s [‰]	D [mm]	Q_{full} [l/s]	n [-]	k [s]	Q_{max} [l/s]
S1	500	8	300	88	33	10	160
S2	120	5	800	925	2	26,5	1798
S3_1	100	6	300	76	5	15,2	194
S3_2	40	7	500	317	2	10,1	979
S4	100	5	400	148	3	23	397
S5	50	7	700	771	1	20,3	1123
S6_1	1290	3	800	716	12	61,5	-
S6_2	580	3	1200	2081	3	85,6	3082

L = length
s = slope
D = diameter
Q_{full} = flow in the completely filled pipe
n = number of tanks
k = retention constant
Q_{max} = maximum flow in the pipe

As described in section 4.2.1 in SMUSI the retention volume that can be activated in the pipes above a rain retention basin is determined. To this end, characteristic curves for the inflow-outflow-overflow relationships are needed for these structures and these were determined by using the SWMM simulation results of the water level, the inflow, the outflow and the overflow for every basin. In Table 6 they are listed separately according to the combined sewer overflows (CSO) and the combined sewer overflow tanks (STT, BPT, PTT) in Table 6.

Table 6: Characteristic curves of the rain retention basins

Structure	h [m]	Q_{out} [l/s]	V [m³]	Q_{bov} [l/s]	Q_{cov} [l/s]
STT	4	100	2000	0	0
	4,05	100	2025	226	0
	4,09	100	2045	602	0
	4,1	100	2050	722	0
BPT	3	12,3	180	0	0
	3,05	12,3	183	204	0
	3,09	12,3	185,4	565	0
	3,11	12,3	186,6	955	0
PTT	3	98	1200	0	0
	3,1	98	1240	0	1120
	3,2	98	1280	0	1574
	3,32	98	1328	997	1974

Structure	h [m]	Q_{out} [l/s]	Q_{in} [l/s]
CSO 1	0,3	50	50
	0,38	50	56
	0,87	50	397
	0,95	50	397,2
CSO 2	0,4	105,5	105,5
	0,62	105,5	169
	1,15	105,5	730
	1,31	105,5	975
	1,35	105,5	946

STT = storage tank

PTT = pass-through tank

h = water level inside the structure

V = volume of the structure

Q_{cov} = clarified overflow

BPT = by-pass tank

CSO = combined sewer overflow

Q_{out} = outflow

Q_{bov} = basin overflow

Q_{in} = inflow

5.2. Results

For the presentation of the results all hydrographs are structured as follows: The x-axis shows the simulation time in hours while the flow in the unit m³/s is displayed on the y-axis. The results of the hydrodynamic model SWMM are illustrated in blue with a drawn through line. The curve of the SMUSI results calculated with the Kalinin-Miljukov method is dashed and coloured in dark red. Marked with a dotted olive green line are the KOSIM-WEST® results and the results of the non-linear transport model in SMUSI are pointed out with a dash dotted line in orange. In the legend of the graphs the abbreviation KM refers to Kalinin-Miljukov. The presented rain retention basins with their abbreviations in brackets are a storage tank (STT), two combined sewer overflows (CSO 1 and CSO 2), a by-pass tank (BPT) and a pass-through tank (PTT).

5.2.1. System 1 ("Normal rainfall")

In this section System 1 is used as representation of the drainage network and is loaded with the "normal" rain event shown in Figure 11. This equals the scenarios 1 and 2 contained in Table 1.

Due to the fact that in the representation of the drainage network in System 1 no pipe is located upstream of the STT, the CSO1 and the BPT, the hydrographs of these basins are exactly the same for the linear and non-linear transport model in SMUSI. Also for the CSO 2 and the PTT the results of these two different models differ only marginally, so that their curves overlap in the following graphs.

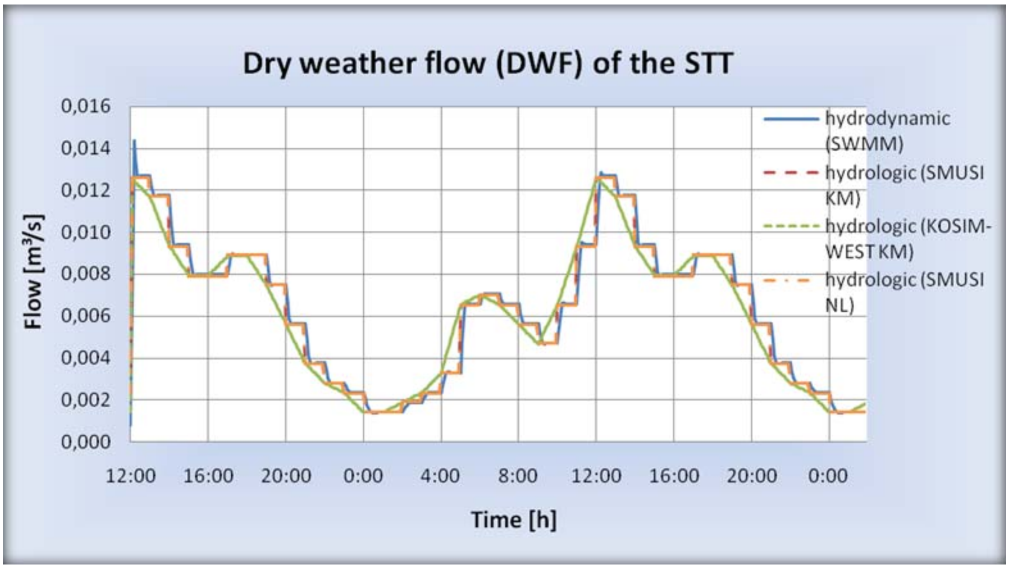


Figure 16: Comparison of the dry weather flow at the entrance of the storage tank (STT)

As illustrated in Figure 16 the dry weather flow is generated with the same values in the three models SWMM, SMUSI and KOSIM-WEST®. An interpolation for these values in KOSIM-WEST® provides a continuous evolution of the curve contrary to the discrete values used in SWMM and SMUSI. This issue is the same for all other basins and is only exemplified for the inflow of the STT during a dry weather period.

The normal rainfall does not cause the emergency overflow of the storage tank and for this rain event also no combined sewer overflow is induced. For this reason only the inflow of the STT and the two CSOs are illustrated in Figure 17.

SMUSI and KOSIM-WEST® deliver almost the same results. Only in the beginning of the simulation the flows in SMUSI are higher than in KOSIM-WEST®. A reason for this can be the use of different initial conditions, e.g. a different initial filling degree of the depressions. The small differences in the results can also be caused by the different solving methods of the softwares: SMUSI works with fixed time steps while the continuous solvers in WEST® vary the time steps.

Compared to SWMM the flow maxima are overestimated over the whole simulation period in the hydrologic models but the dynamics and the overall trajectory of the curve is rendered relatively well.

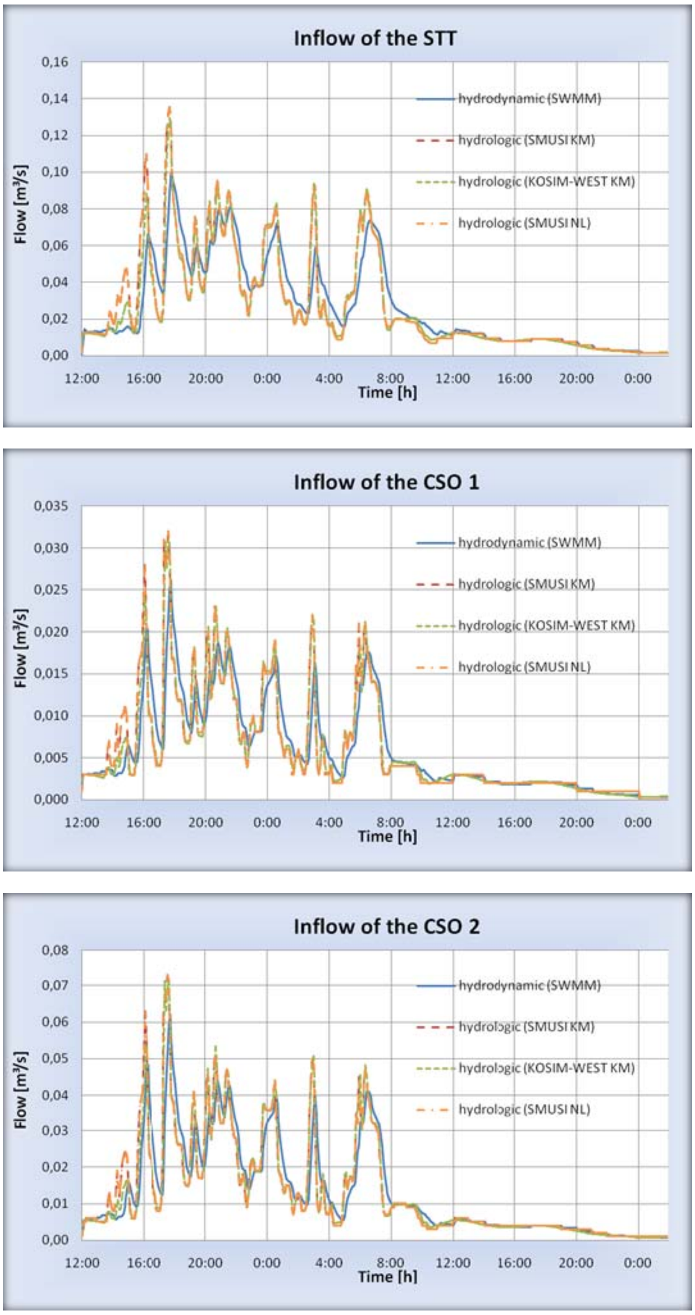


Figure 17: Comparison of the inflow for the "normal" rain event in the STT, CSO 1 and CSO 2 in System 1

The overflow of the BPT and the PTT, which is illustrated in Figure 18, is evaluated too high in the hydrological models compared to SWMM. In the beginning of the rain event SMUSI and KOSIM-WEST® record an overflow while in SWMM the volume in the tanks is not yet completely filled and no overflow occurs. The reason for this is that the inflow to the two tanks is overestimated in the hydrological models, because the retention behaviour of the pipes is not represented sufficiently. On account of this a wrong overflow event is recorded for a longer period in the beginning of the simulation at the PTT than at the BPT, because of the long pipe S6 upstream of the PTT. After this wrongly detected overflow event the overflow is assessed with nearly the same values compared to the SWMM results.

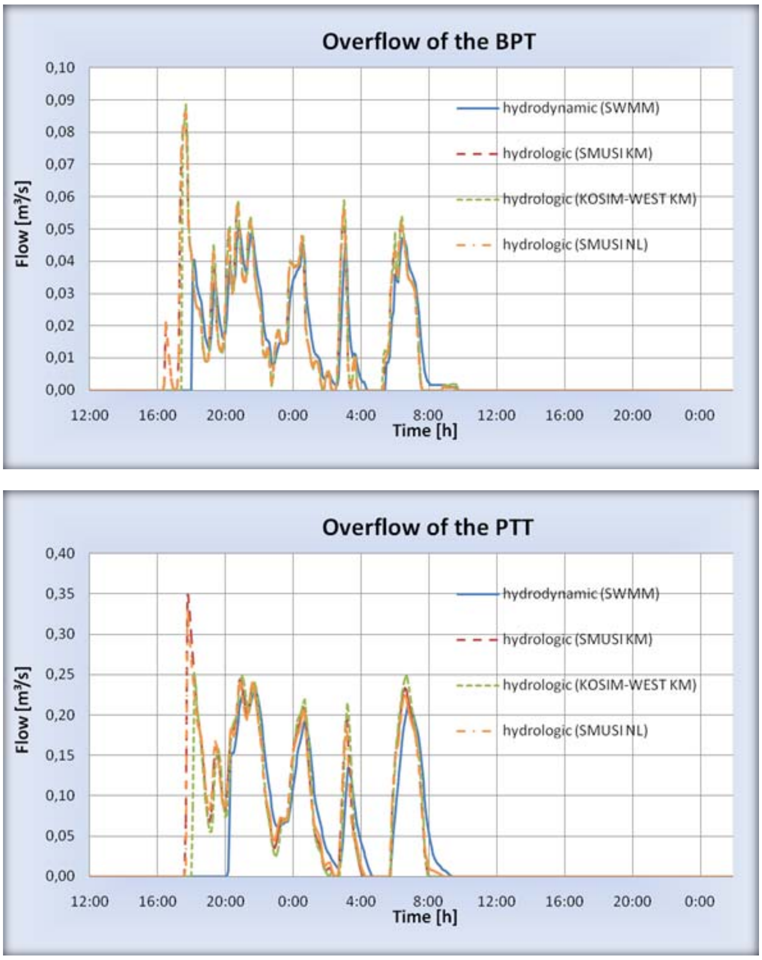


Figure 18: Comparison of the overflow of the BPT and PTT for the "normal" rain event in System 1

5.2.2. System 1 ("Heavy rainfall")

In this section the results of the scenarios 3 and 4 (see Table 1) are presented, i.e. System 1 is used as representation of the drainage network and is loaded with the "heavy" rain event shown in Figure 12. In this case the backwater-models in SMUSI and KOSIM-WEST® are disabled.

First of all, as mentioned in the previous section, the results in SMUSI, whether the non-linear or the Kalinin-Miljukov method is applied for the water transport, are exactly the same for the STT, the CSO 1 and the BPT. This is due to the too rough representation of the drainage network in System 1. Also for the CSO 2 and the PTT the results of the two different calculation methods in SMUSI differ insignificantly. The reason for this are the small flow rates in the pipes (S3 and S6) upstream of these basins (CSO 2 and PTT) compared to the high flow rates which are discharged from the subcatchments (F3 and F6). Thus, the different calculation methods are irrelevant in System 1.

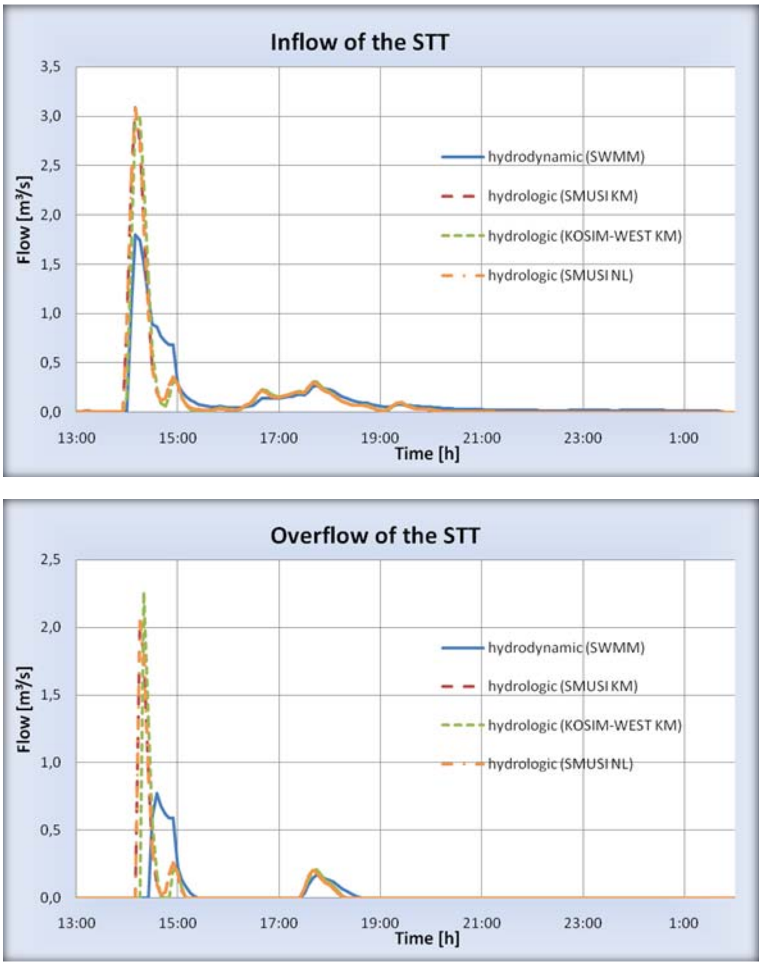


Figure 19: Comparison of the inflow and overflow of the STT for the "heavy" rain event in System 1

To illustrate the results, the hydrographs of the inflow and overflow of the STT are used (see Figure 19). The inflow and therewith the overflow is strongly overrated by the hydrological models SMUSI and KOSIM-WEST® compared to SWMM. The reason for this are backwater effects that occur as result of the heavy rain peak intensity around 14 h, which leads to a capacity overload in the sewer network. This situation is not identified in the hydrological models, so that the flow rate in the pipes is not limited and the water does not get stored inside the sewer system. For the smaller rain intensities that occur later the flows are assessed in the right range and the second overflow event between 17 h and 19 h is detected correctly.

This finding that an overestimated inflow leads to a too high overflow is qualitatively the same for the other basins. For the sake of completeness their overflow curves are shown in Figure 20. It also clarifies the fact that to take backwater effects into account the representation of the sewer network has to be more detailed than it is in System 1. To be able to represent the retention behaviour of the pipes more accurately and to limit the inflow to the rain retention basins System 2 was defined, as illustrated in Figure 15 and described in section 5.1.

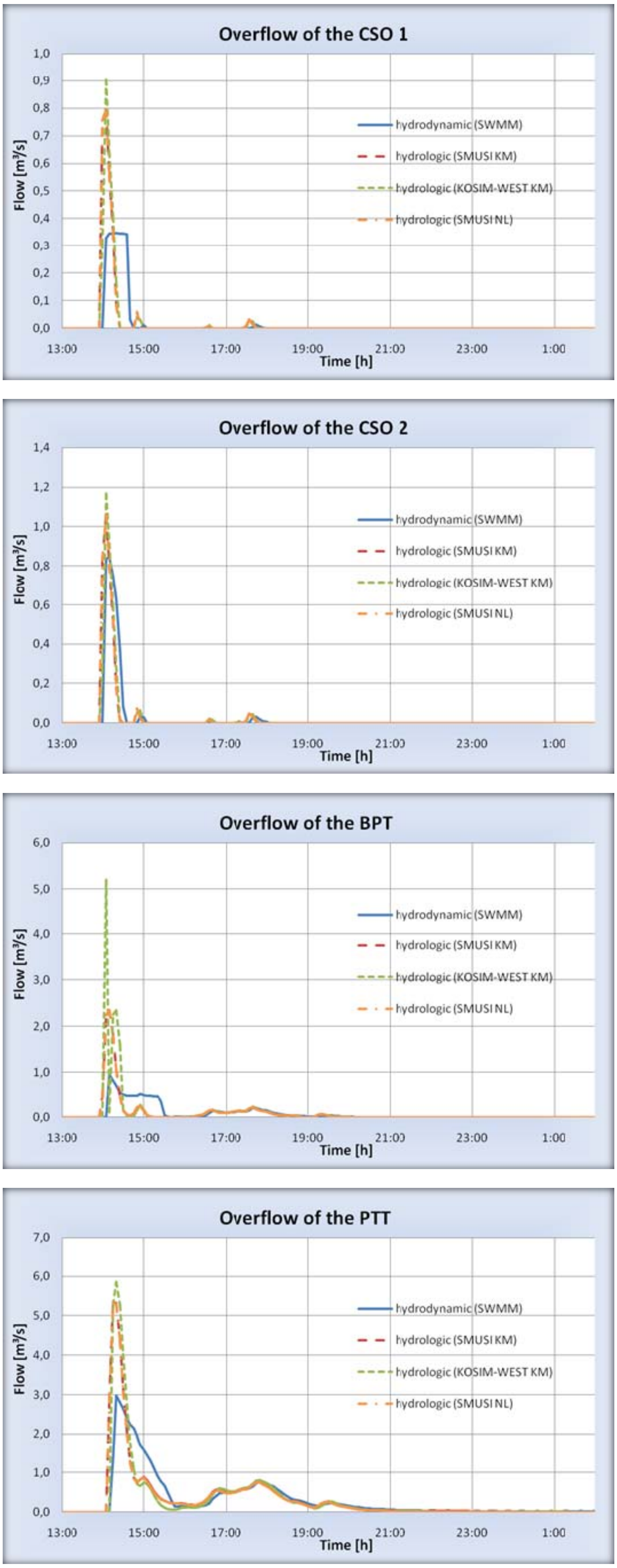


Figure 20: Comparison of the overflow of the CSOs, BPT and PTT for the “heavy” rain event in System 1

5.2.3. System 2

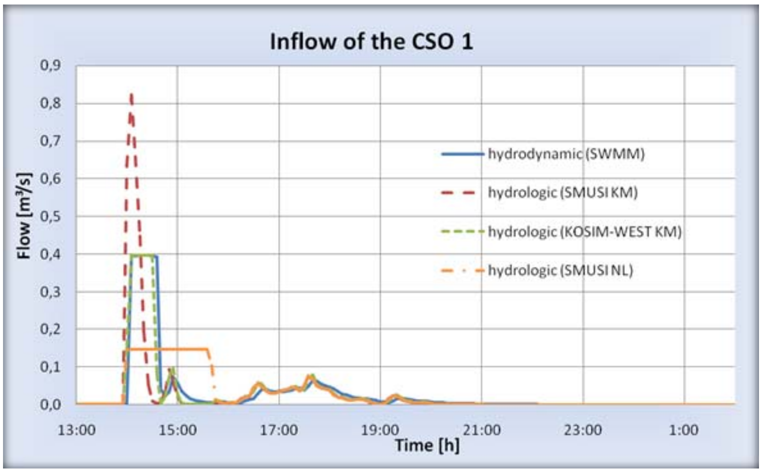
As described in the previous section the drainage network is represented in more detail in System 2 to take backwater effects into account. This section presents the results of the scenarios 5 and 6 (see Table 1), i.e. the backwater-models in SMUSI and KOSIM-WEST® are now enabled. In SMUSI the activatable retention volume of a pipe lying upstream of a rain retention basin which provokes a backwater effect is calculated, while the KOSIM-WEST® backwater model is a combiner-splitter combination. The backwater models are described in more detail in section 4.2.1.

The effects of the backwater-models will be explained by using the inflow and overflow curve of the CSO 1 as example. These hydrographs are illustrated in Figure 21.

The calculation of the activatable retention volume in SMUSI with the Kalinin-Miljukov transport model leads to a dampening of the maximum of the overflow curve compared to the scenarios with the disabled backwater-model. So, the retention behaviour of the pipe upstream of the CSO 1 is represented better, but nevertheless the maximum of the discharge wave is still twice as high as in the SWMM results. To improve the performance it would be necessary to extend the dimensions of the pipe, so that their retention behaviour can be simulated closer to reality.

The maximum pipe flow in the non-linear transport model in SMUSI is limited to the flow through the completely filled pipe S4 with Q_{full} = 148 l/s. For this reason the inflow and also the overflow of the CSO 1 is lower than in SWMM, where pressurized flow occur. The excess water gets virtually stored in the pipe until the flow rate in the pipe is again lower than Q_{full}. Then this stored water is released. The volume of the discharged water is in the same range as in SWMM, but the timing and the maximum value strongly differ.

The best fit of the inflow as well as the overflow curve to the SWMM-results without changing the pipe dimensions can be reached with the combiner-splitter backflow model in KOSIM-WEST®. The maximum flow through pipe S4 was directly taken from the SWMM-results and limits the flow rate in the pipe S4 to this maximum flow rate. The virtual storage effect of the excess water in the pipe is the same as in SMUSI with the non-linear transport model, with the difference that the threshold for the beginning of the storage lies higher. Hence, not only the discharged water volume, but also the temporal appearance and the maximum value of the discharge wave are assessed in the same range as in SWMM.



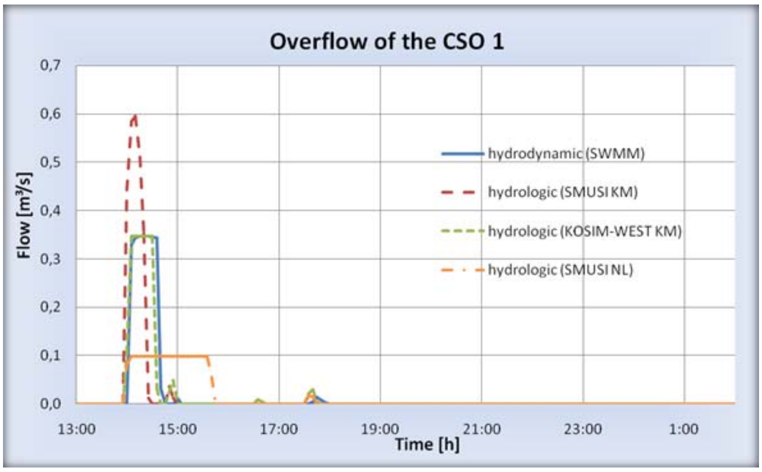


Figure 21: Comparison of the inflow and overflow of the CSO 1 for the "heavy" rain event in System 2

The inflow and overflow curves of the other rain retention basins show the same characteristics as described above. The hydrographs are shown in Figure 22 and Figure 23.

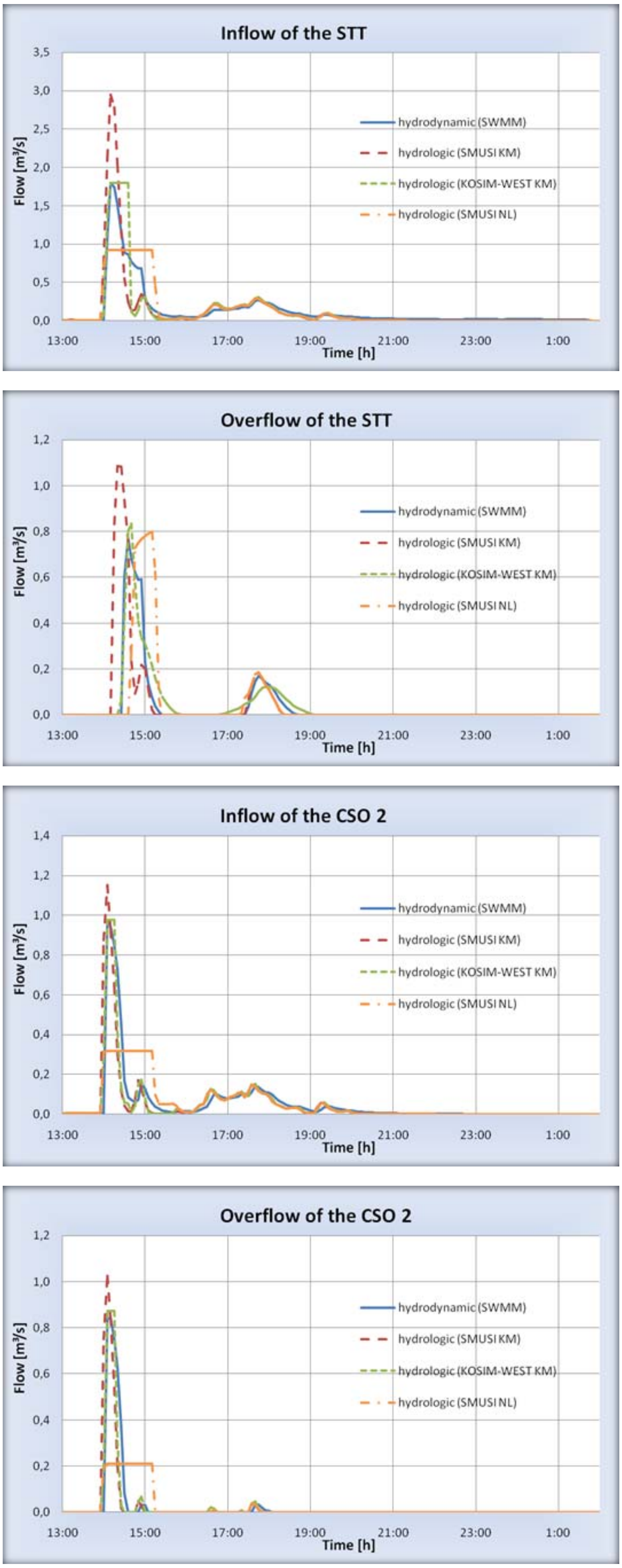


Figure 22: Comparison of the inflow and overflow of the STT and CSO2 for the “heavy” rain event in System 2

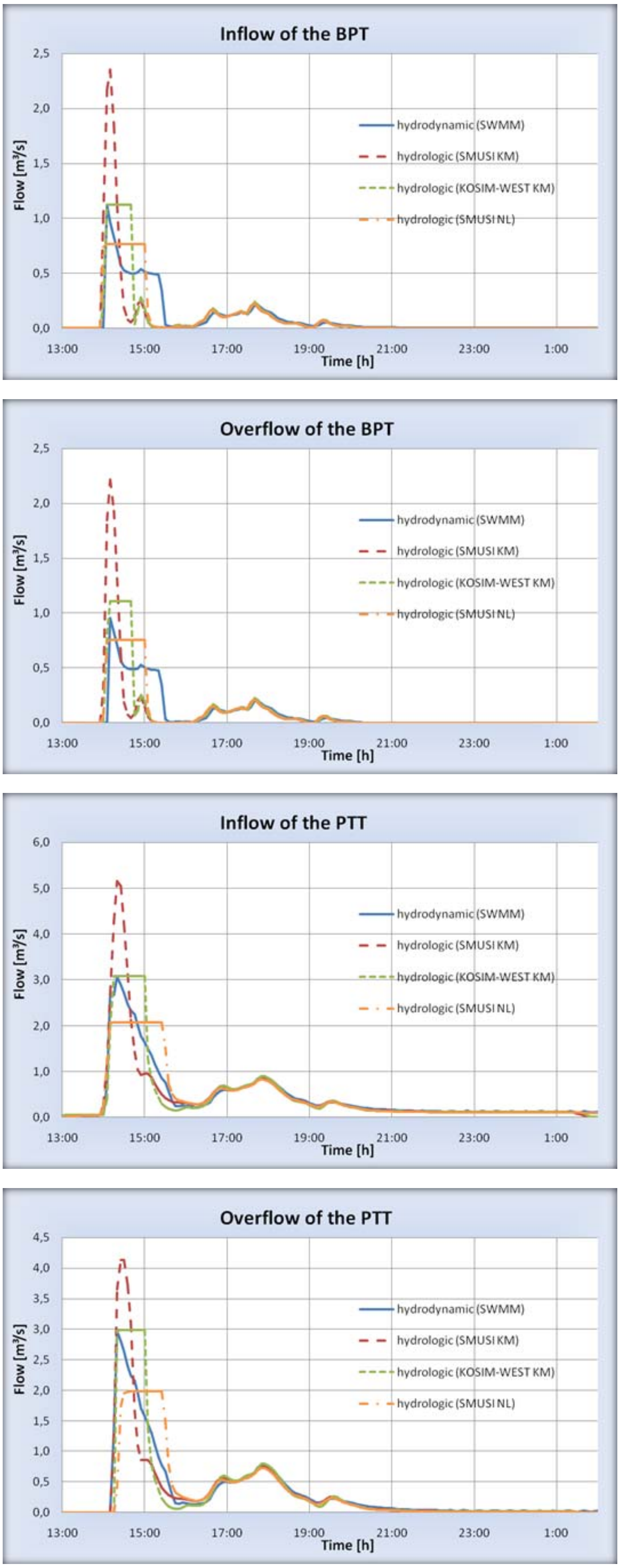


Figure 23: Comparison of the inflow and overflow of the BPT and PTT for the “heavy” rain event in System 2

5.3. Conclusion

Regarding the simulation results of SMUSI and KOSIM-WEST® it can be said that both modelling tools deliver almost the same results with the standard approaches, i.e. when the backwater-models are disabled. This was expected given the theoretical approaches behind the two sewer models compared in chapter 4. The small differences are due to the different solving methods in the softwares. Hence, the implementation of the KOSIM model in WEST® was successfully tested against the similar hydrological rainfall-runoff modelling software SMUSI.

Concerning the representation of the drainage network in System 1 it can be stated that without calibration the flows in the sewer network are assessed too high and overflow events are wrongly predicted. This problem could be solved by changing the actual characteristics of the pipes, e.g. varying the length and the diameter, so that the results of the hydrological models SMUSI and KOSIM-WEST® come within reach of the results of the hydrodynamic model SWMM.

For high rain intensities the overflows are strongly overestimated by the hydrological models due to the non-consideration of backwater effects. It could be shown that backwater effects have a significant influence on the performance of the sewer network and that they have to be taken into account.

This result leads to System 2, a more detailed representation of the drainage network, as described in section 5.1. The conceptual backwater-models in SMUSI and KOSIM-WEST® dampen the flow maxima, though in a different extent.

The flow maxima simulated with the activatable retention volume in SMUSI are still too high compared to the SWMM-results, which makes it necessary to change the actual pipe dimensions in a way that the retention behaviour approximates reality better.

Even though in SMUSI the volume of discharged water is assessed in the right range with the non-linear transport model, the dynamics and the peak of the discharge wave differ from the SWMM-results.

In this context the combiner-splitter backflow-model in KOSIM-WEST® delivers the best results regarding the discharged water volume, the dynamics and the peak of the discharge wave in comparison to the SWMM-results. The physical pipe dimensions can be used and only the maximum flow obtained in the SWMM model needs to be specified.

In summary, it could be shown that with conceptual backwater models it is possible to take backwater-effects into account and reach reasonable results without raising the calculation time.

6. Implementation of the non-linear approach in KOSIM-WEST®

In this chapter, the outflow-volume relationship for pipes with circular cross-sections is derived. Following this, the basis of the non-linear approach for the water transport process and its implementation in KOSIM-WEST® is described. Finally, the implemented non-linear approach in KOSIM-WEST is tested with reference to the non-linear approach in SMUSI.

6.1. Derivation of the flow-relationship for pipes with partially filled circular cross-sections

Figure 24 illustrates the geometric relations for a partially filled circular cross-section. Due to the geometrical similarity of all circular cross-sections the geometrical characteristic values of a circular cross-section can be expressed in relative values, i.e. the ratio of actual value to the value of the completely filled cross-section.

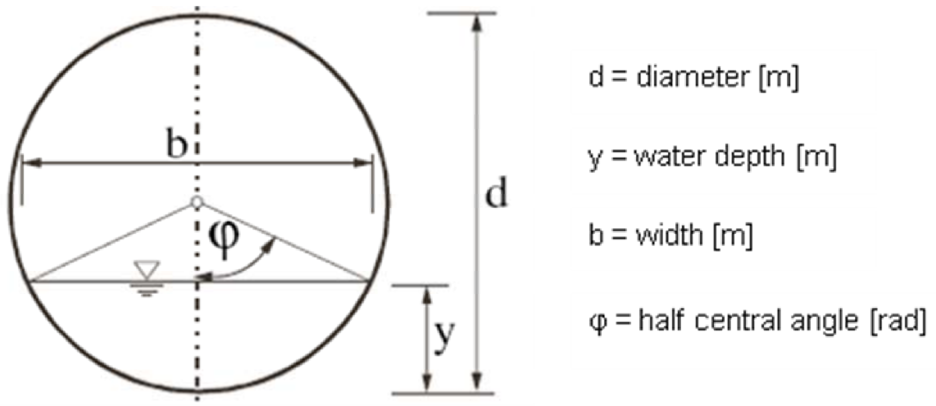


Figure 24: Geometrical characteristic values of a partially filled cirular cross-section (Manhart, 2007)

In the following the derivation of the relative geometric characteristic values of a circular cross-section is described:

$$y_{full} = d$$
$$\tau = \frac{y}{y_{full}} = \frac{y}{d}$$

y = water depth

y_{full} = water depth of the completely filled pipe (6)

τ = filling level

$$b = d \cdot \sin\varphi$$
$$b_{full} = d$$
$$\beta = \frac{b}{b_{full}} = \sin\varphi$$

b = width of the flow

b_{full} = width of the flow of the completely filled pipe (7)

β = relative width of the flow

The actual cross-sectional area of flow A results from the area of the sector of a circle minus the area of the triangle:

$$A = A_{\square} - A_{\square} = \left[\varphi \cdot \frac{d^2}{4}\right]_{\square} - \left[\left(\frac{d}{2} \cdot \cos\varphi\right) \cdot \frac{d}{2} \cdot \sin\varphi\right]_{\square} = \frac{d^2}{4} \cdot (\varphi - \sin\varphi \cdot \cos\varphi) \qquad (8)$$

with $\sin\varphi \cdot \cos\varphi = \frac{\sin(2\varphi)}{2}$ follows:

$$A = \frac{d^2}{4} \cdot \left(\varphi - \frac{\sin(2\varphi)}{2}\right)$$

$$A_{full} = \pi \cdot \frac{d^2}{4} \qquad (9)$$

$$\xi = \frac{A}{A_{full}} = \frac{1}{\pi} \cdot \left(\varphi - \frac{\sin(2\varphi)}{2}\right)$$

A	= cross-sectional area of flow
A_{full}	= cross-sectional area of flow of the completely filled pipe
ξ	= relative cross-sectional area of flow

Dependent on φ the wetted perimeter can be determined:

$$P = \varphi \cdot d$$

$$P_{full} = \pi \cdot d \qquad (10)$$

$$\eta = \frac{P}{P_{full}} = \frac{\varphi}{\pi}$$

P	= wetted perimeter
P_{full}	= wetted perimeter of the completely filled pipe
η	= relative wetted perimeter

The hydraulic radius R_{hy} is defined as the cross-sectional area of flow divided by the wetted perimeter. It is needed to determine the discharge in relation to the filling level τ.

$$R_{hy} = \frac{A}{P}$$

$$R_{hy,full} = \frac{A_{full}}{P_{full}} = \frac{\pi \cdot \frac{d^2}{4}}{\pi \cdot d} = \frac{d}{4} \qquad (11)$$

$$\zeta = \frac{R_{hy}}{R_{hy,full}} = \frac{\xi}{\eta}$$

R_{hy}	= hydraulic radius
$R_{hy,full}$	= hydraulic radius of the completely filled pipe
ζ	= relative hydraulic radius

To describe all characteristic values dependent on the filling level τ a relation between β and τ is needed:

$$\left(\frac{d}{2}\right)^2 = \left(\frac{b}{2}\right)^2 + \left(\frac{d}{2} - y\right)^2 \Leftrightarrow \frac{d^2}{4} = \frac{b^2}{4} + \frac{d^2}{4} - d \cdot y + y^2 \Leftrightarrow \frac{b^2}{4 \cdot d^2} = \frac{y}{d} - \frac{y^2}{d^2}$$
$$\Leftrightarrow \beta^2 = 4\left(\tau - \tau^2\right) \tag{12}$$

Insertion of equation (12) in equation (7) gives:

$$\varphi = \arcsin\beta = \arcsin\left(\sqrt{4\left(\tau - \tau^2\right)}\right) = 2 \cdot \arcsin\left(\sqrt{\tau}\right) \tag{13}$$

According to equations (7) to (11) the other characteristic values can be calculated dependent on the filling level τ. The course of these values is illustrated in Figure 25.

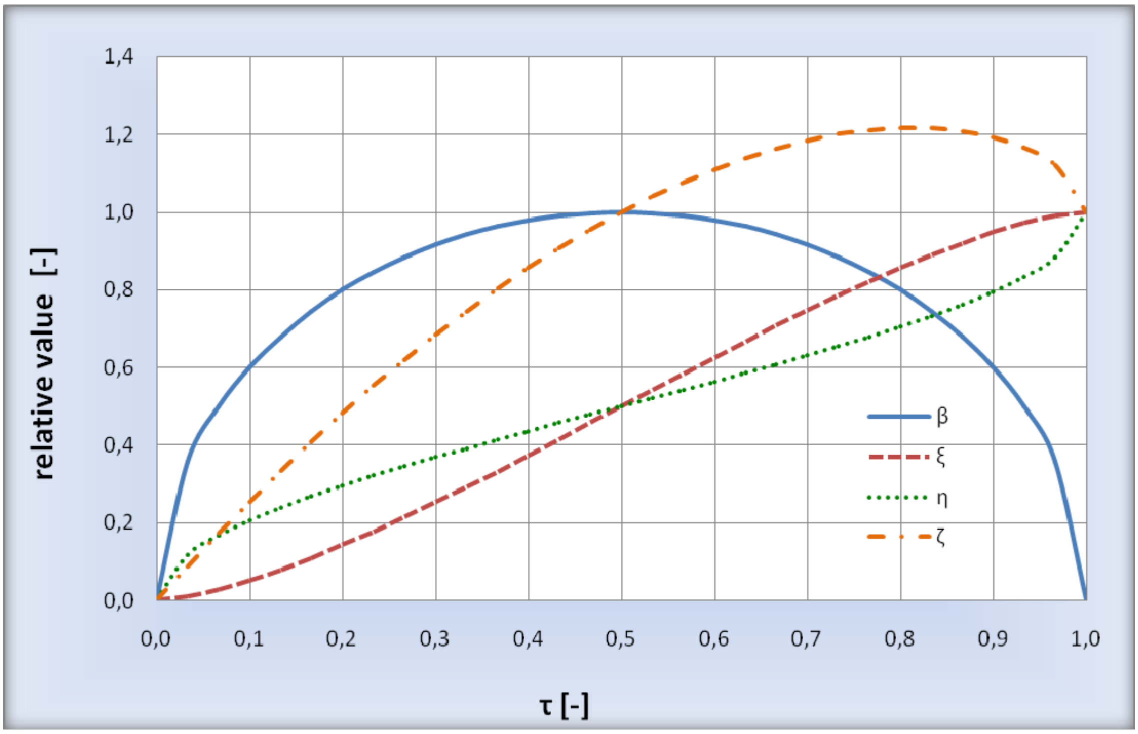


Figure 25: Curves of the characteristic geometrical values for a partially filled circular cross-section

The relative hydraulic radius ζ has its maximum value at a filling degree of about 81 % and the value decreases after this until the pipe is completely filled.

The discharge Q in a pipe with steady flow can now be determined by using the de Chézy-equation:

$$Q = C \cdot \sqrt{R_{hy} \cdot s} \cdot A \tag{14}$$

C	= Chézy-coefficient [$m^{0,5}/s$]	R_{hy}	= hydraulic radius [m]
s	= slope [-]	A	= cross-sectional area of flow [m²]

According to Manhart (2007) the following relationship between the coefficients and the hydraulic radius could be found in experiments:

$$\frac{C}{C_{full}} = \zeta^{\frac{1}{8}} \tag{15}$$

Herewith and the equations (8), (11) and (14) the ratio between the discharge of a partially filled and a completely filled cross-section follows to:

$$\frac{Q}{Q_{full}} = \frac{C \cdot \sqrt{R_{hy} \cdot s} \cdot A}{C_{full} \cdot \sqrt{R_{hy,full} \cdot s} \cdot A_{full}} = \zeta^{\frac{1}{8}} \cdot \zeta^{\frac{1}{2}} \cdot \xi = \zeta^{\frac{5}{8}} \cdot \xi \tag{16}$$

With equation (16) it is possible to determine the discharge for every water level inside a pipe with any cross-section. However this equation is only valid under steady and normal flow conditions. The evolution of the relative discharge for a circular cross-section is shown in Figure 26. Due to the increase of the hydraulic radius the discharge of a partially filled circular cross-section is higher than the discharge of a completely filled pipe for a filling level between 81 and 100 %.

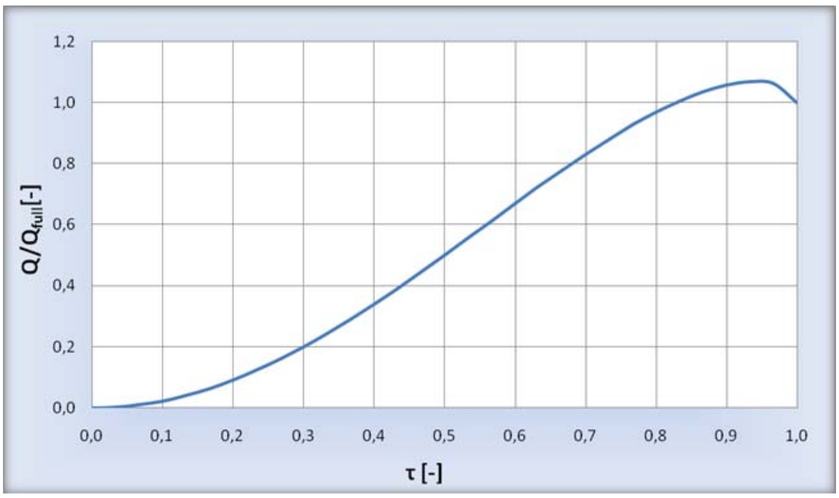


Figure 26: Evolution of the relative discharge for a partially filled circular cross-section

6.3. Description of the non-linear approach

The approach behind the linear tank (see Figure 27) method is that the relationship between the volume of the tank and the outflow is linear (18). The change of the volume in time is described by a mass balance (17).

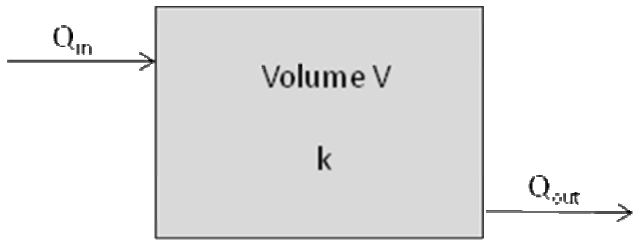


Figure 27: Linear tank

$$\frac{dV}{dt} = Q_{in}(t) - Q_{out}(t) \tag{17}$$

$$Q_{out}(t) = \frac{1}{k} \cdot V(t) \tag{18}$$

Q_{in}	= Inflow [m³/s]	Q_{out}	= Outflow [m³/s]
V	= Volume in the tank [m³]	k	= Retention constant [s]

Figure 28 shows that the actual outflow-volume relationship is not linear; in this case a circular cross-section is used to illustrate this fact. Due to this non-linearity the linear approach is a strong simplification. The aim of this section is to find a way in which this non-linear relationship can be expressed and implemented in a model.

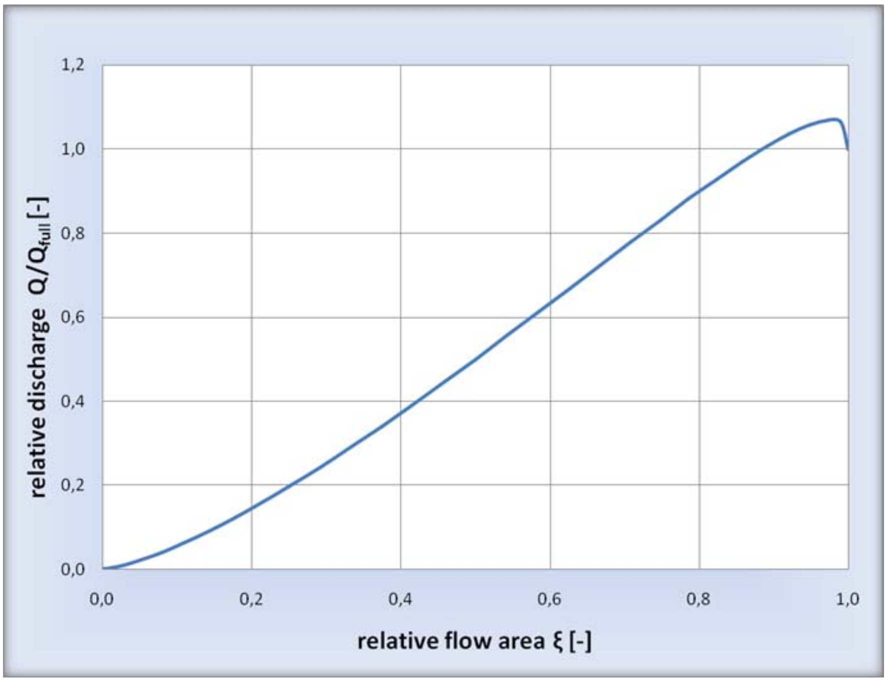
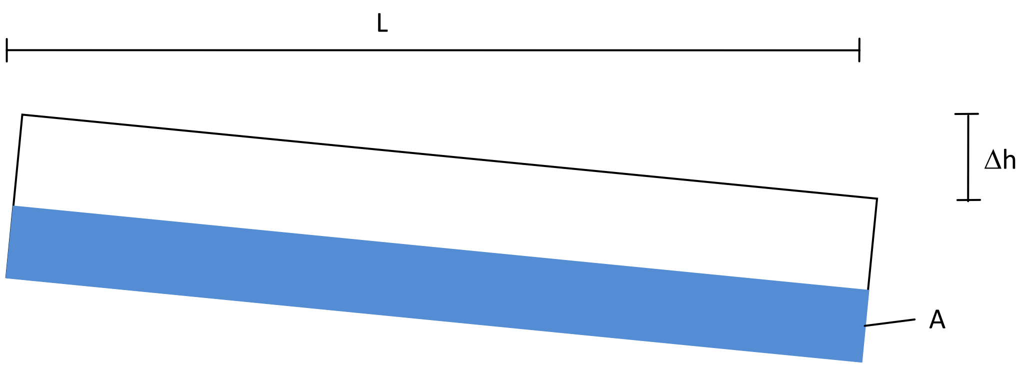


Figure 28: Q-V relationship for a circular cross-section

The longitudinal section of a pipe is illustrated in Figure 29. Thereby each pipe is characterized by its length, the height difference between its inlet and outlet and its cross-sectional area of flow. The following equations are derived only for pipes with a circular cross-section, for which the geometrical characteristic values are shown in Figure 24.



L = length	Δh = height difference
A = cross-sectional area of flow	s = slope = $\frac{\Delta h}{L}$

Figure 29: Longitudinal section of a pipe

The water volume V_{full} that can be contained in a completely filled pipe can be calculated like this:

$$V_{full} = A_{full} \cdot L = \frac{\pi \cdot d^2}{4} \cdot L \tag{19}$$

With the assumption that $\Delta h << L$ follows:

$$s \approx 0 \text{ and } \frac{V}{V_{full}} = \frac{A}{A_{full}} \tag{20}$$

Figure 25 shows that the relation between the filling level τ and the relative area of flow ξ is explicit, i.e. to each value of ξ corresponds exactly one value of τ. Combination of equations (9) and (13) leads to:

$$\xi = \frac{A}{A_{full}} = \frac{1}{\pi} \cdot \left(2 \cdot \arcsin\left(\sqrt{\tau}\right) - \frac{\sin\left(4 \cdot \arcsin\left(\sqrt{\tau}\right)\right)}{2} \right) = \frac{V}{V_{full}} \tag{21}$$

It is not possible to solve equation (21) explicitly for ξ. Therefore for a known value of ξ, τ has to be calculated iteratively, for example by using the Newton-method. Equations (22) to (26) describe the procedure for calculating the relative discharge for a circular cross-section. The schematic of this procedure is shown in Figure 30.

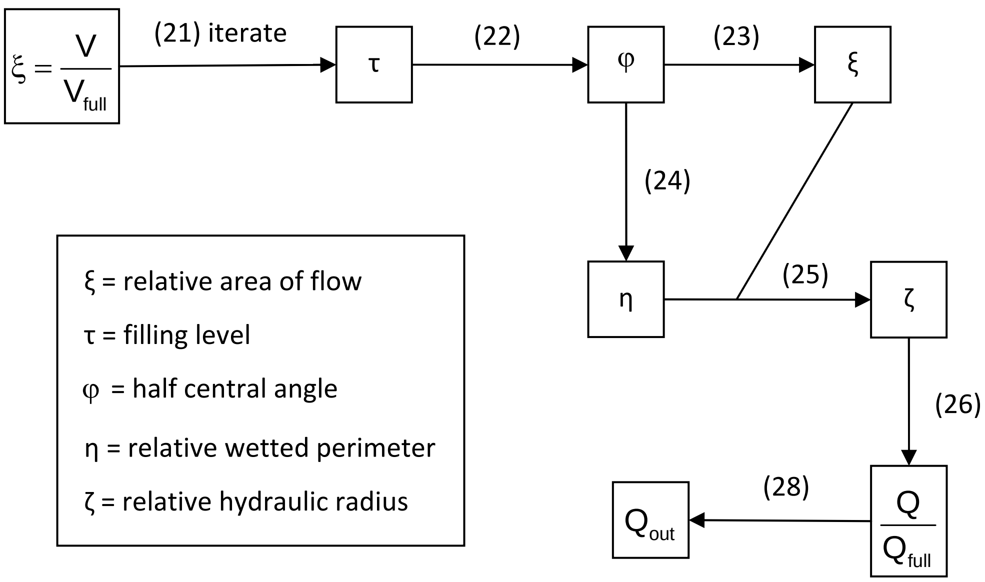


Figure 30: Schematic of the procedure for calculating the outflow of a circular cross-section

$$\varphi = 2 \cdot \arcsin\left(\sqrt{\tau}\right) \tag{22}$$

$$\xi = \frac{1}{\pi} \cdot \left(\varphi - \frac{\sin(2\varphi)}{2} \right) \tag{23}$$

$$\eta = \frac{\varphi}{\pi} \tag{24}$$

$$\zeta = \frac{\xi}{\eta} \tag{25}$$

$$\frac{Q}{Q_{full}} = \zeta^{\frac{5}{8}} \cdot \xi \tag{26}$$

The outflow of a completely filled pipe Q_{full} can be calculated using the Prandtl-Colebrook equation:

$$Q_{full} = A_{full} \cdot \left[-2 \cdot \log\left(\frac{2{,}51 \cdot \nu}{d \cdot \sqrt{2 \cdot g \cdot d \cdot s}} + \frac{k_s}{3{,}71 \cdot d} \right) \cdot \sqrt{2 \cdot g \cdot d \cdot s} \right] \tag{27}$$

d = diameter [m]	A_{full}	= cross-sectional area of the completely filled pipe [m²]
g = gravity [m/s²]	ν	= kinematic viscosity [m²/s]
s = slope [-]	k_s	= pipe roughness [m]

In the end the outflow of the pipe is calculated by multiplying the relative discharge with the value of Q_{full} :

$$Q_{out} = \frac{Q}{Q_{full}} \cdot Q_{full} \tag{28}$$

Instead of the linear outflow-volume relationship in equation (18) the outflow is now calculated non-linearly without the need of determining a retention constant k for the tank cascade. According to Engel (1994) the characteristic length in the Kalinin-Miljukov method, as described in section 4.2.1, is not a significant parameter if a non-linear volume-outflow relationship is used. Therefore the linear tank cascade which is used to simulate the pipe flow can be replaced by a simple tank and the volume of this tank corresponds with the volume of the pipe. This consideration is closer to reality and simplifies the model building process in KOSIM-WEST® a lot, because only one model needs to be included for one pipe. For other cross-sectional shapes than a circular cross-section only the equations (21) to (24) have to be replaced.

6.4. Implementation of the non-linear approach in WEST®

In WEST® the ordinary differential equation (equation (17)) of the volume of the tank with respect to time is solved numerically. Therefore only a relation in any form between the volume and the outflow is needed and no care has to be taken of the discretization of time in discrete time steps, as typically used in hydrological models, such as SMUSI. Based on an initial volume the outflow for the actual timestep is calculated with which again the volume of the next timestep is calculated in the end. This procedure is shown in Figure 31.

timestep t_0 (inflow is known):

Initial volume V_0:

⇨ Outflow = f (V_0)

⇨ $\frac{dV}{dt} = \text{Inflow - Outflow}$

⇨ V_1 determined numerically : $V_1 = V_0 + \frac{dV}{dt} \cdot \Delta t$

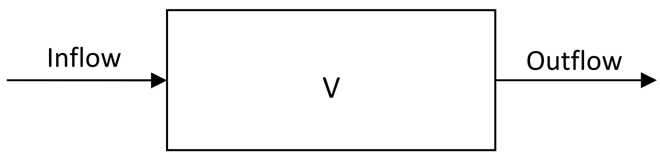


Figure 31: Procedure of calculating the outflow and volume of a tank in WEST®

In the prior section a way to express the non-linear relationship between the volume and the outflow of a pipe with a circular cross-section is described. For this non-linear approach it is only necessary to implement an algorithm which calculates the filling level τ based on the known relative area of flow ξ as input variable (see equation (21)). To solve this implicit relationship the Newton-method is applied to find the value of τ numerically. In this method a starting value of τ is needed and the next approximated value is calculated as follows:

$$\tau_{n+1} = \tau_n - \frac{f(\tau_n)}{f'(\tau_n)} \tag{29}$$

with: $$f(\tau) = \frac{1}{2 \cdot \pi} \cdot \left(4 \cdot \arcsin\left(\sqrt{\tau}\right) - \sin\left(4 \cdot \arcsin\left(\sqrt{\tau}\right)\right)\right) - \frac{V}{V_{max}} \tag{30}$$

and $$f'(\tau) = \frac{1}{\sqrt{\tau} \cdot \sqrt{1-\tau} \cdot \pi} \cdot \left(1 - \cos\left(4 \cdot \arcsin\left(\sqrt{\tau}\right)\right)\right) \tag{31}$$

It is important that the values of τ are between 0 and 1, so that the derivation of the function returns good values. The breakup-criterion, i.e. the accuracy to be reached before the algorithm is stopped and the results can be used to calculate the flow, can be set by the user. With an accuracy of 0,001 the hydrograph of the pipe outflow has a smooth course compared to larger values of the accuracy criterion. The C++-source code of the implemented algorithm is given in Figure 32.

```
double calCircularTau (double Arel, double prevTau, double breakupcriterion)
{
        double f;
        double derivf;
        double denominator;
        double tau;
        double fourasinsqrtau;

        if (Arel <= 0) { tau = 0; }
        else if (Arel >= 1) { tau = 1; }
        else
        {
                // tau has to be in the range of 0 and 1
                // otherwise the results of the functions f and derivf would be NaN!!
                if (prevTau <= 0)     {       tau = 0.5;     }
                else if (prevTau >= 1){       tau = 0.5;     }
                else {  tau = prevTau; }

                fourasinsqrtau = 4 * asin(sqrt(tau));
                f = 1 / (2 * pi) * (fourasinsqrtau - sin(fourasinsqrtau)) - Arel;

                // The newton-method is applied to determine a value for tau until
                // f (tau) < breakupcriterion
                while (fabs(f) > breakupcriterion)
                {
                        fourasinsqrtau = 4 * asin(sqrt(tau));
                        f = 1 / (2 * pi) * (fourasinsqrtau - sin(fourasinsqrtau)) - Arel;

                        denominator = sqrt(tau) * sqrt(1-tau);
                        derivf = 1 / (denominator * pi) * (1 - cos(fourasinsqrtau));

                        tau = tau - f / derivf;

                        // tau has to be in the range of 0 and 1
                        // otherwise the results of the functions f and derivf would be NaN!!
                        if (tau <= 0 || tau >= 1)     {       tau = 0.5;     }
                }
        }
        return tau;
}
```

Figure 32: Source code of the Newton-algorithm calculating τ

6.5. Backwater model

The outflow of the pipe calculated with the non-linear method, as described in the previous sections, is limited to the flow of the completely filled pipe Q_{full}. For the case that the inflow is much higher than Q_{full} the volume in the pipe increases above the volume of the completely filled pipe V_{full}, i.e. the water gets virtually stored in the pipe. However, in reality the flow increases when a pipe is surcharged as a result of the increased water level in the manhole above it. Therefore it is necessary to allow a higher outflow than Q_{full}. The simplest approach is that when the volume in the pipe increases above the volume of the completely filled pipe V_{full}, the outflow-volume relationship is allowed to increase linearly above Q_{full} with the parameter a as gradient:

$$Q_{out} = a \cdot \left(V - V_{full}\right) + Q_{full} \text{ for } V > V_{full} \tag{32}$$

Q_{out} = outflow of the pipe [m³/s] Q_{full} = outflow of the completely filled pipe [m³/s]

V = actual volume of the pipe [m³] V_{full} = volume of the completely filled pipe [m³]

a = gradient in the outflow-volume relationship [1/s]

In reality this outflow is of course limited to a maximum, i.e. when the manhole is completely filled with water. Therefore it makes sense to add a parameter Q_{max}, which represents this maximum outflow and can be set by the user. There are two possible ways to implement this limited outflow in the pipe-model.

The first method is to keep the one pipe-model and limit the outflow to Q_{max}. As the result the water gets virtually stored in the pipe when the inflow is higher than Q_{max}. In this case it only happens for a value higher than Q_{full}, as described above. The virtual storage is then emptied when the inflow is again lower than Q_{max}.

The second method is to combine the non-linear pipe model as described in section 6.2 with the backwater model developed by Solvi et al. (2005), as illustrated in Figure 9, a splitter is positioned after the pipe which leads only Q_{max} to the structure lying downstream. All excess water is returned as backwater to the upstream structure. Ahead of the pipe a combiner is placed which adds up the backwater from downstream and the inflow from upstream.
The advantage compared to the first method is that with this model it is possible to simulate water flowing back to upstream basins, by which for example an overflow can be caused.

In the ATV-example, described in chapter 5 both methods lead to the same results due to the fact that in the SWMM-model the outflow of the rain retention basins is represented by pumps and for this reason backwater in the downstream pipes has no influence on the upstream basins. This is exemplified on the inflow of the by-pass tank (BPT) for the heavy rain event (see Figure 12), which is illustrated in Figure 33. The small difference between the non-linear model and the linear method (Kalinin-Miljukov method) with backwater-model in the rising of the hydrograph is due to the different transport models implemented in KOSIM-WEST®.

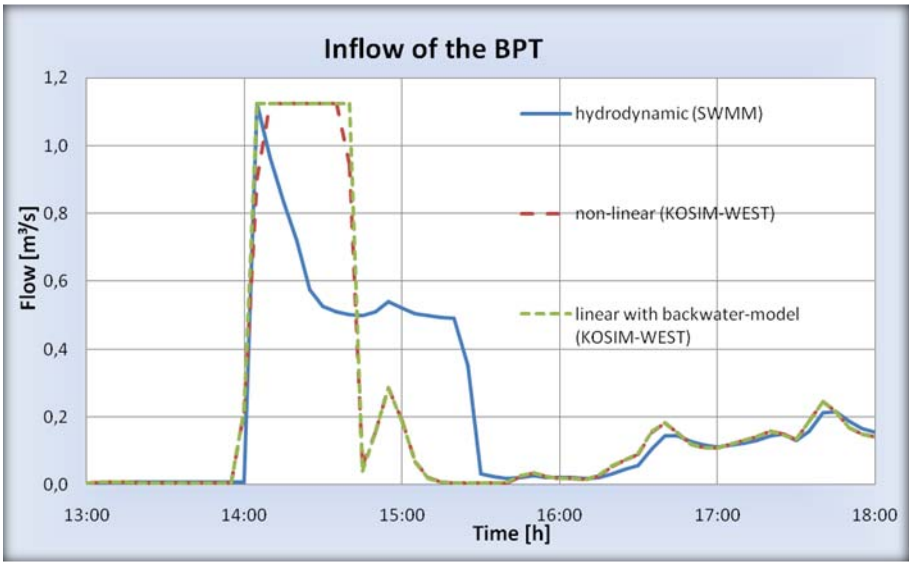


Figure 33: Comparison of the inflow of the BPT for the "heavy" rain event on 15.06.1968

To be more general and closer to reality it is recommended to use the non-linear-backwater-pipe-model. Since it is not possible to describe the flow behaviour in general in case of a backwater situation, the user will have to calibrate the model on real data sets or hydraulic model results with the parameters Q_{max} and the gradient of the outflow-volume relationship a.

6.6. Comparison of the non-linear method in SMUSI and KOSIM-WEST®

For a comparison between the non-linear pipe-model implemented in SMUSI and KOSIM-WEST® only the pipe S2 from the ATV-example in chapter 5, located upstream of the storage tank (see Figure 15), is regarded. The results of the hydrodynamic model SWMM are the reference for the flow and are used to determine the parameter Q_{max}, with which a pressurized flow can be taken into account. The pipe has a length of 120 m, a slope of 5 ‰, a diameter of 800 mm and a roughness of 1,5 mm. Figure 34 shows four different hydrograph curves of the flow rate of the pipe: The first is simulated with SWMM, the second with the non-linear method in SMUSI, the third with the non-linear method in KOSIM-WEST® without a maximum outflow Q_{max} above Q_{full} and the fourth with the same method like the previous but with a value for Q_{max}. The simulation date is the 15.06.1968 and the rainfall is the heavy rainfall shown in Figure 12. The hydrographs of the non-linear method in SMUSI and KOSIM-WEST® have nearly the same course and the correlation coefficient between these two simulations over the simulation period is 0,98. The maximum outflow in these models matches the outflow of the completely filled pipe Q_{full} of 925 l/s. According to SWMM the peak of the hydrograph is about 1798 l/s, thus nearly two times higher than Q_{full}. For that reason the water is virtually stored in SMUSI and KOSIM-WEST®, the outflow stays longer on the value of Q_{full} and approaches the course of the SWMM hydrograph not until about 2 hours after the rain event started. By using Q_{max} = 1798 l/s in the non-linear method in KOSIM-WEST® it is possible to approximate the SWMM hydrograph more accurately. So the course of this can be reached around 1 hour after the beginning of the rain event. At about 16 h all non-linear methods give the same results.

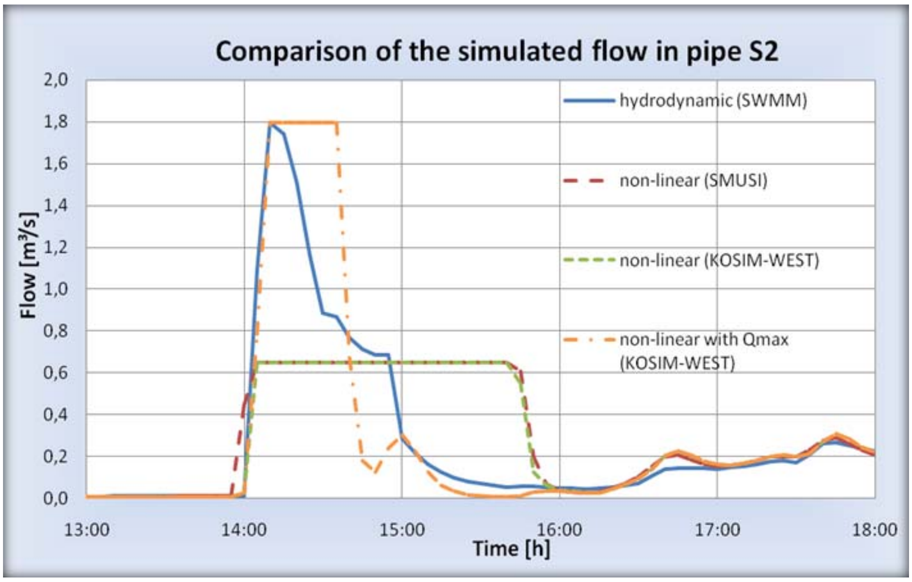


Figure 34: Comparison of the simulated flow in pipe S2 for a "heavy" rain event on 15.06.1968

With this example it is shown that, if no value for Q_{max} is set, the non-linear method which has been implemented in KOSIM-WEST® works nearly the same as in SMUSI. The small differences are due to the different implementations. By the extension of this model with a pressurized outflow Q_{max}, the outflow can approximate reality better.

6.6. Conclusion

The implemented non-linear approach for the water transport leads to several simplifications in building a model of a sewer system in WEST® compared to the linear Kalinin Miljukov method.

So far in WEST® the parameters needed for the Kalinin-Miljukov method have to be calculated in an external Excel-sheet. This is necessary because in the Configuration Builder of WEST® the user has to choose the submodel for the pipe-icon corresponding with the calculated number of tanks n in the Excel-sheet. After the configuration of the entire sewer network is finished, the model will be compiled automatically to a WEST®model library file (WML-file), with which simulations in the Experimentation Environment of WEST® can be run. In the Experimentation Environment only the parameters but not the submodels can be changed, so that as parameter for the water transport process only the retention constant k remains. If the user wants to change the pipe dimensions, he has to start again with the Excel-file, then choose a different submodel for the pipe-icon in the Configuration Builder and compile the model of the whole sewer system. So, useful functionalities available in WEST® such as automatic parameter estimation and sensitivity analysis cannot be applied for the water transport process. This matter complicates the fitting of the simulation results on real data sets.

In the non-linear approach the pipe is represented in the model by one tank. Thus, only different submodels for the pipe-icon in the Configuration Builder are necessary for different pipe shapes. The characteristics of the pipe such as diameter, slope, length and roughness can be directly set by the user after the compiling process in the Experimentation Environment. Moreover, the calibration procedure is facilitated and the parameter estimation as well as the sensitivity analysis feature in WEST® can be used on the water transport process.

The model building process in WEST® with the linear and non-linear water transport model is illustrated in Figure 35.

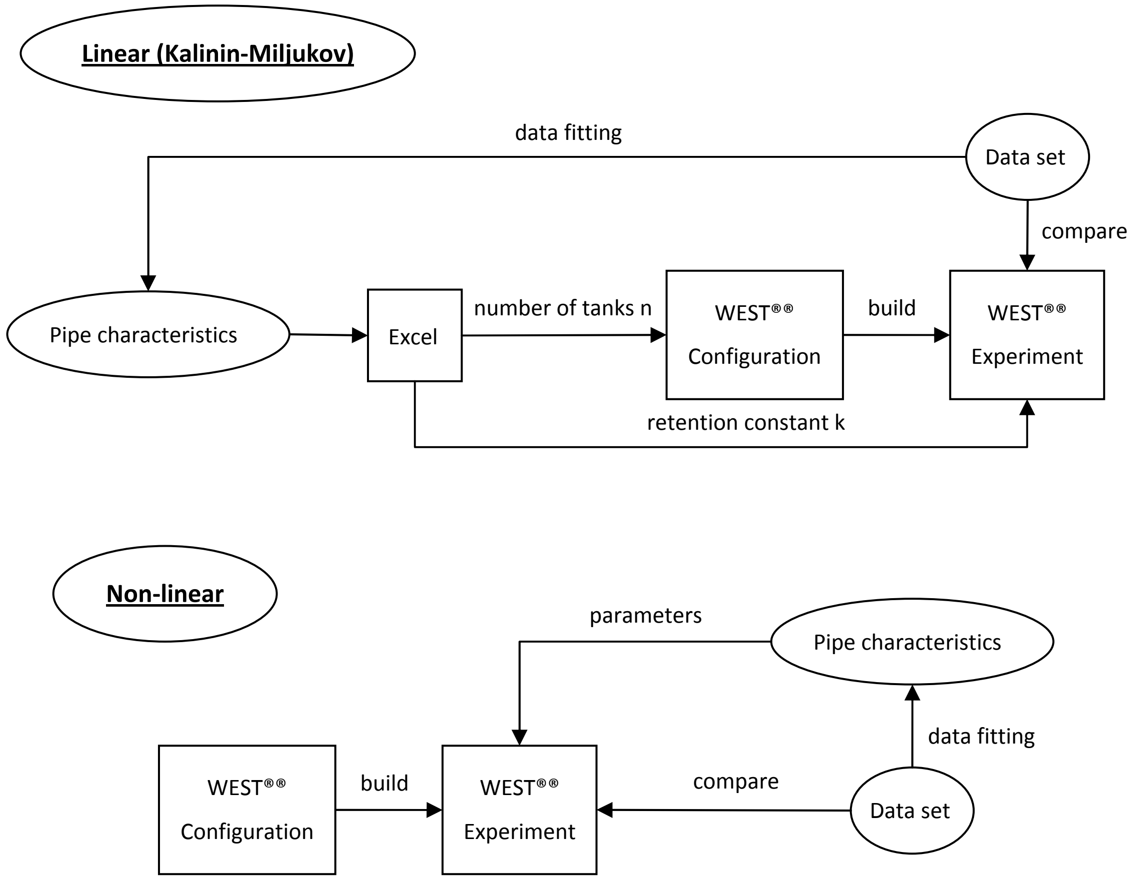


Figure 35: Schematic of the model building process in WEST® with the linear and non-linear water transport model

Another advantage of the non-linear approach is that the number of equations, which is coupled with the number of tanks, is reduced. For instance in the simple representation of the sewer network in System 1 of the ATV-example in chapter 5 the number of tanks is reduced from 55 to 3, meaning that the number of differential equations has changed from 55 to 3. The numerical solvers in WEST® work better with fewer equations, so that the calculation performance is improved. It has to be said that this improvement is somewhat reduced by the iterative procedure for the outflow-volume relationship. The non-linear approach also allows the user to model larger sewer systems, since the number of equations is reduced, facilitating the compilation process in WEST® (Vanhooren et al., 2003). Finally, due to the fact that the user has to specify initial conditions for every tank, the reduction of tanks speeds up the implementation of a model.

7. Final Conclusion

To improve the simulation accuracy of integrated urban wastewater system models the modelling approach used takes an increasing number of processes into account. The simulation of these processes is influenced by the state of the art knowledge about physical, chemical and biological processes as well as its measuring technology. This concerns the sewer system, the wastewater treatment plant and the river as elements of the urban wastewater system.

This thesis focused mainly on sewer system modelling and more particularly on the water transport process. The simulation results computed with different hydrological methods to describe the water transport process (Kalinin-Miljukov method and non-linear approach) were compared with reference to a hydrodynamic water transport model. Furthermore, the efficiency and the mode of operation of conceptual backwater models, which are necessary to take backwater effects into account within a hydrological sewer model, were also examined.

For these purposes the hydrological sewer models KOSIM-WEST® and SMUSI and the hydrodynamic modelling software SWMM were used. With the applied case study in chapter 5 it could be shown that SMUSI and KOSIM-WEST® deliver almost the same results with the standard hydrological approaches, when backwater-effects are not considered. Thus, the implementation of the KOSIM modelling tool in WEST® was successfully tested with reference to SMUSI.

For high rain intensities the overflows were strongly overestimated by the hydrological models due to the non-consideration of backwater effects occurring in the studied system. Hence, backwater effects have a significant influence on performance assessments regarding the sewer network and it is essential to take them into account.

With the combiner-splitter combination as backwater-model in KOSIM-WEST® the flow curves calculated with SWMM could be approximated the best. The discharge waves of the rain retention basins and the combined sewer overflows, which are mainly influencing the water quality in the river, could be assessed in the right range both in terms of the discharged water volume and the dynamics and peak of the discharge wave.

Furthermore, the non-linear approach for the water transport process was implemented in KOSIM-WEST®. This leads to a simplification of this process, because considering the pipe as one tank with a non-linear outflow-volume relationship is closer to reality. Also the non-linear water transport model eases the model building procedure of a sewer system in WEST® compared to the linear Kalinin-Miljukov method.

In summary, it could be shown that with the non-linear transport model and the combiner-splitter combination as backwater model in KOSIM-WEST®, backwater effects can be taken into account and good results can be reached without increasing the calculation time.

In this work only the water quantity (flow) was considered in the sewer models. Thus, KOSIM-WEST® could be improved only on that score. In future work processes concerning the water quality in the sewer system like settling and resuspension of pollutants in the pipe or their biodegradation on during transport to the wastewater treatment plant, have to be examined. Furthermore, non-linear outflow-volume relationships have to be derived for all types of cross-sections which can be found in a sewer system and should then be implemented in KOSIM-WEST®.

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