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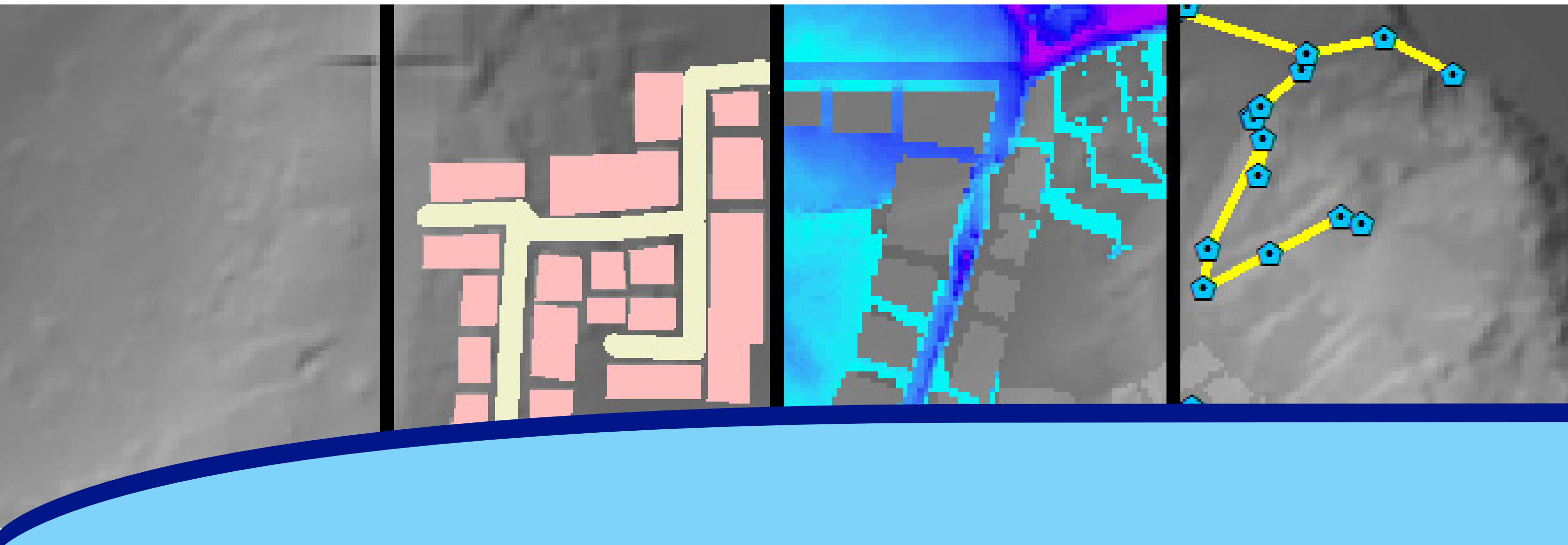

453.810 Analysis and Modelling
Summer Term 2012

Dynamic Modelling of Urban Rainfall Runoff and Drainage
Coupling DHI MIKE URBAN and MIKE FLOOD

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1. Introduction

Urban flooding and inundation are a serious and inescapable problem for many cities worldwide. However, both the spatial scale of these processes and the underlying causes differ significantly. Most cities in the industrialised part of the world are generally confronted with small scale issues, often caused by insufficient capacities of drainage systems during intense rain. On the other hand, cities in the developing world experience more severe problems. These can usually be traced back to lower sewer standards and much larger amount of rainfall. Such conditions illustrate that accurate simulations of urban hydrological processes are required in order to predict potential flood threats and to efficiently improve the designs of sewer infrastructure.

It is therefore the purpose of the present paper to dynamically model rainfall runoff in an urban area. This is done by coupling a 2D hydrologic overland model with a 1D hydraulic flow model, using the two software packages of DHI - MIKE URBAN and MIKE FLOOD. The overall work is split into three chapters, comprising the concept, the method and the actual application. The concept block deals with urban storm events, urban drainage infrastructure and gives an introduction into urban runoff and drainage modelling. Thus, different rainfall characteristics and their recordings are presented, the process of runoff generation is explained and drainage components as well as different system types are described. Besides, the concept of modelling, different approaches of model classification and advantages and limitations of models are highlighted. The third chapter on the method focuses on the various flows and elements of urban runoff modelling, required input data, 2D- and 1D modelling and the coupling possibilities between the two models. Chapter four is concerned with the application of the software and the creation of the urban drainage models. Here, the software packages are specified, an overview on the model, the study area and data input is given and the procedure of the model setup is outlined. Furthermore, possible usages of the model outcomes are listed, potential software couplings are identified and a critical reflection on the work is given. Last, a conclusion rounds off the work by summarising the relevant findings and outcomes.

2. Concept

The following chapter provides an introduction into the basic thematic of this work. Consisting of three parts, the chapter deals with rainfall and runoff in urban areas, describes different urban drainage systems as a response to rainfall runoff and introduces into the principles of urban flood models.

2.1. Urban Rainfall Runoff

This abstract brings into focus different characteristics and categories of rainfall and specifies important terms regarding measurement, prediction and rainfall design. Further, the emergence of runoff in urban areas is described, including the basic processes and flows that occur, starting from the rainfall hyetograph up to the runoff hydrograph.

2.1.1. Rainfall

Runoff in urban areas is generally caused by rainfall. Although there are many different forms of precipitation, such as snow for example, rainfall is the most significant contributor to storm water runoff in most areas (Butler and Davies 2011:77). Thereby the amount of rainfall changes, depending on time and space. When looking at short periods of time and small distances, these changes may be small. However, they become bigger with increasing time and distance. This is due to the fact that space and time are related to each other (Loucks et al. 2005:437). Intensive rainfalls tend to originate from small rain cells (approx. one kilometre in diameter) that either last for a short period of time or pass by the catchment area rapidly. Because of their small size, the intensity of these types of storms varies considerably in space. On the other hand, rainfall events that last for longer typically arise from larger rainfall cells of large weather systems (Loucks et al. 2005:437). Thus, the spatial difference in intensity is smaller. Depending on the area of the catchment and the number of rainfall measurement stations, it is possible to account for local differences.

Moreover, rainfall events show an intensity-duration-frequency relationship (IDF) where intensity and duration are inversely related (Butler and Davies 2011:82). As Figure 1 illustrates, the intensity of an event decreases with an increasing duration of rainfall. Besides, it can be seen from the figure that rainfall intensity and the frequency of an event are backwards related: less frequent events tend to be more intense.

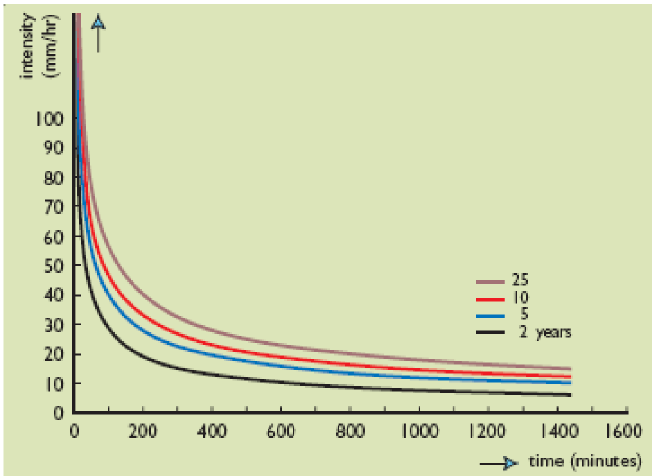


Fig. 1 : Rainfall intensity-duration-frequency diagram (Loucks et al. 2005:439).

At this point it needs to be mentioned that storm events do not just comprise one fixed rainfall depth for a given duration. Rather its intensity alters with time throughout an event (Butler and Davies 2011:90). This can be characterised by the peakedness and the skew of the storm in a hyetograph, as displayed in Figure 2. Thus a hyetograph can be defined as a storm profile, where rainfall intensity is plotted against time.

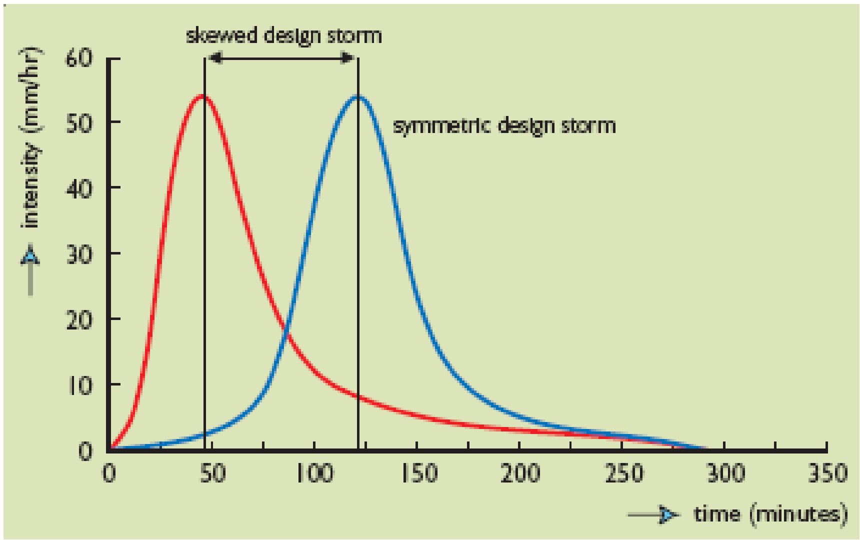


Fig. 2 : Storm peakedness and skew of a storm in a hyetograph (Loucks et al. 2005:439).

The representation and prediction of rainfall events is a fundamental step in order to take measures against the effects of rainfall (e. g. the design and operation of urban drainage). Reference to Butler and Davies (2011:77) reveals, that this is primarily done by making use of observations - "(...) the origin of all knowledge about rainfall". These provide historical records and facilitate the derivation of relationships between rainfall characteristics. A common way of making use of rainfall data is the analysis of long rainfall records and the definition of their statistical properties. Thereby three parameters are generally utilised to define the statistics of rainfall depth (Locks et al. 2005:438):

- the rainfall intensity (in mm/ h) or rain depth (in mm) of an event
- the duration/ time period over which the intensity occurs (in min)
- the probability of the event to take place in any particular year

For expressing the return period of an event, i. e. the frequency with which an event is likely to occur, it is common practice in the urban context to calculate the inverse of probability (Butler and Davies 2011:81). To exemplify this, an event with a chance of 0.1 of being matched or even exceeded in any particular year is expected to return every 1/0.1 or 10 years.

Locks et al. (2005:437) have drawn attention to the fact that rainfall events can be split into two categories, namely recorded (real) events and synthetic (not-real) designs. The latter can be further subdivided into stochastically generated rainfall data and design storms. Design storms are developed from actual rainfall data (intensity–duration–frequency data) and consist of an idealized hyetograph and a specific return period. Their purpose is to reduce the number of runs that are necessary to analyse the performance of urban drainage systems under specific flow conditions. In contrast, historical/ real events do neither have an idealized hyetograph nor an attached return period (Butler and Davies 2011:92).

Consequently, these events are mainly used to verify flow simulation models by providing measured hyetographs and simultaneous flow observations.
Beside these described single events, there are also multiple events for both rainfall categories, namely historical time-series and synthetic time-series. While historical series represent all measured rainfall at a certain location and for a specific time period, synthetic series can be based on conventional rainfall parameters (e. g. storm depth, catchment wetness and storm peakedness) (Butler and Davies 2011:92ff). Thus the challenge of integrating numerous different events is avoided by taking only few synthetic storms into account.
There is a lot of expert discussion going on concerning the question which of the rainfall events is most suitable to represent design rainfall (Locks et al. 2005:437, Butler and Davies 2011:93). The advantage of real rainfall time-series is that they take into account many different conditions. Thus they are almost certain to contain the conditions that are critical for the catchment. On the other hand, a lot of recorded data and analyses are required and it is difficult to ascertain how suitable the particular portion of history actually is. The argument in favour of using synthetic storms is that they require only few events to assess the performance of a system and thereforee are easy to utilise. However, the two methods do not exclude each other: Some syntheses are involved in the use of real rainfall, when choosing the storms to be used in time-series (Locks et al. 2005:437).

2.1.2. Rainfall Runoff

Based on the precipitation data that is available for a catchment area, runoff volumes need to be predicted in order to obtain data for various analyses. The relation between precipitation and runoff depends on a number of rainfall and catchment characteristics (Viessmann and Lewis 1989:149). Thus the lack of detailed basin information such as site-specific land cover, imperviousness, slope data, soil condition, etc. may deteriorate the accuracy of the predictions.
The actual runoff process includes numerous sub-processes, parameters and events as indicated in Figure 3. Starting with precipitation, the rain water falling on the land surface is exposed to various initial and continuous losses (Zoppou 2001:200). Continuous processes are assumed to continue throughout and beyond the storm event (as long as water is available on the surface). Initial losses encompass interception, wetting losses and depression storage, whereas evapo-transpiration and infiltration represent continuous losses (Butler and Davies 2011:108). In the following, the whole process is described more precisely.

The first encounters of rainfall water are with intercepting surfaces, namely grass, plants, trees and other structures that collect and retain the rainwater. The magnitude of interception loss depends on the particular surface type. In a second step, the remaining water amount may become trapped in numerous small surface depressions or - contingent on the slope and the type of surface – may simply build up over the ground surface as surface retention (Viessmann and Lewis 1989:44). Continuing losses, such as infiltration, evaporation or leakage may eventually release the retained water again. Infiltration in this context describes the process of water passing through the ground surface into the upper layer of the soil, which is generally unsaturated. Penetrating deeper into the soil, it may even reach the groundwater/ saturated zone. Water that has seeped into the soil through the unsaturated zone and afterwards becomes surface water again, is named as inter-flow (Zoppou 2001:200). The magnitude of

infiltration is reliant on the soil type, its structure, the compaction, moisture content, the surface cover and on the water depth on the soil. It can range from 100 % for highly permeable surfaces to 0 % for completely impermeable areas. Further, the infiltration capacity of a particular soil describes the rate at which the water seeps into it. It is usually high at the beginning but tends to diminish exponentially, once the soil becomes saturated (Butler and Davies 2011:109). In contrast to infiltration, evapo-transpiration, the second process of continuing loss, refers to the vaporisation of water, directly from the vegetation and open water bodies. Finally, the residuary amount of water which is not affected by any of the losses, is called the 'excess rainfall'. As described, it builds up over the ground surface and once the water film has accumulated a sufficient depth, overland flow sets in (Figure 3). A precondition for this surface flow to happen is that the rate of surface supply is higher than the rate at which water infiltrates into the ground (Viessmann and Lewis 1989:44). Moreover, initial-loss depths refer to the minimum quantity of rainfall that is necessary to cause surface runoff (Loucks et al. 2005:439). By the time overland flow reaches defined channels, such as roads or natural streams for example, the flow changes to a gutter flow; it becomes sewer flow once it enters the underground drainage system.
The relative proportions of the different losses and the excess rainfall is not the same for all catchments. Rather, they vary both with the characteristics of the surface and with time during the rain event. For instance, overland flow usually rises with an increasingly saturated ground (Butler and Davies 2011:3). In terms of surface properties, urbanisation has a crucial influence on runoff processes: Rain falling over a catchment falls on either an impervious or a pervious surface. In the latter case, some part of the rainwater seeps into the ground, while the rest becomes surface runoff. However, in case the water falls on an impervious area, almost the whole rainwater turns into runoff (Zoppou 2001:197). Thus, due to the fact that urban areas are typically characterised by extensive impervious surfaces (such as roofs, roads, soil compaction and other modifications), they tend to result in increased total surface volume (Elliott and Trowsdale 2005:394). Besides, as surface runoff flows faster over solid surfaces than it does over natural areas, the runoff flow occurs and disappears much faster, resulting in a higher peak flow. This is also displayed in Figure 4.

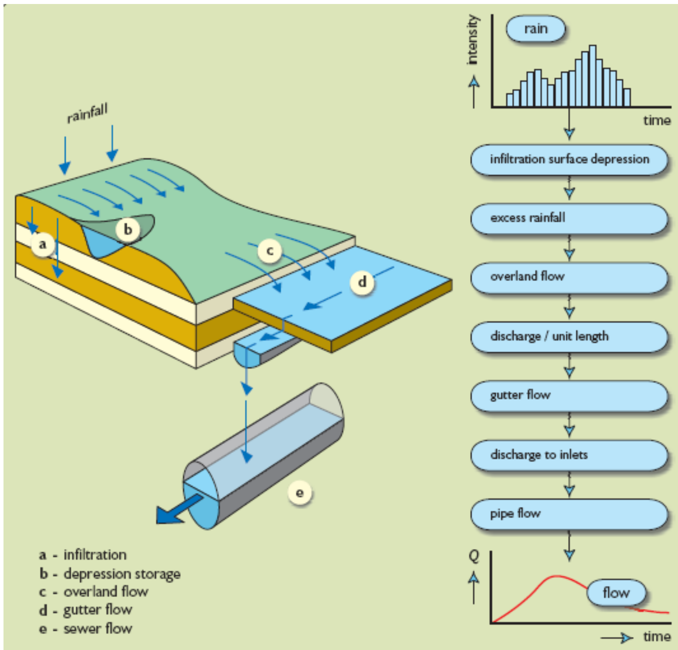


Fig. 3 : Schematic description of runoff emergence (Loucks et al. 2005:440).

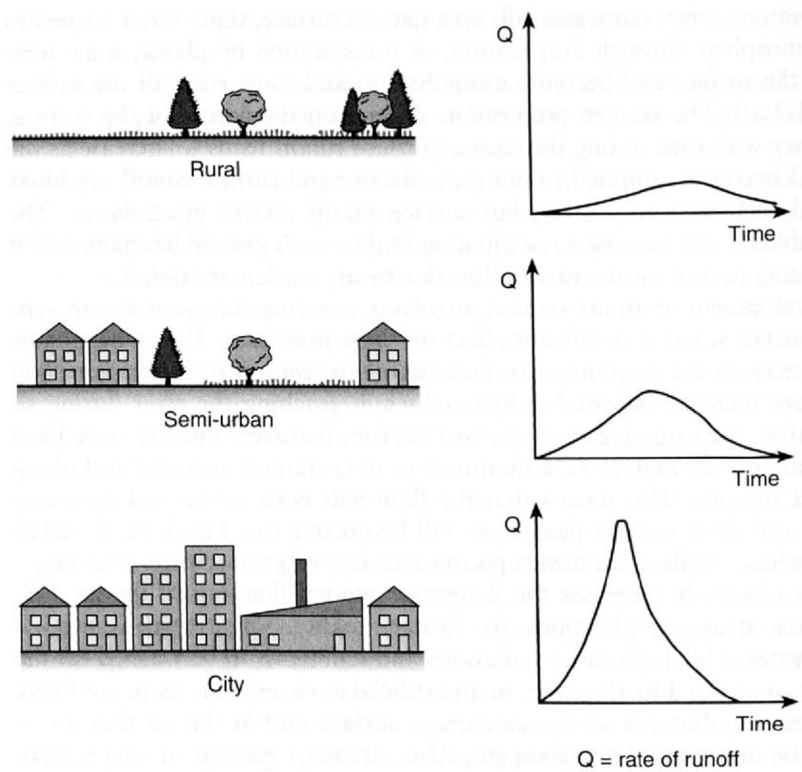


Fig. 4 : Impact of urbanisation on runoff emergence (Butler and Davies 2011:4).

2.2. Urban Drainage

In this part, measures to deal with urban runoff are presented. Thereforee reasons are given that explain why urban drainage systems are necessary. Moreover, fundamental terms and components of urban drainage are defined and a classification schema of drainage systems is outlined.

2.2.1. Need for Urban Drainage

The effective disposition of rainwater is crucial to protect properties and lives from flooding. As demonstrated, this is particularly true for urban areas, which are more susceptible to flooding than rural regions. Thus, precipitation and the need to collect the resulting runoff are the main purposes of urban drainage systems (Locks et al. 2005:437). However, there is a second reason, also leading back to the interaction between human and the natural water cycle: the need to abstract water and to provide it for human life (Butler and Davies 2011:1). Thereforee there are two types of human interaction that necessitate two types of water to be drained. The first one is stormwater, i. e. rainwater that felt on an impervious, built-up area. The second type of water is wastewater, which is supplied water to support and maintain a standard of living. In both cases, the lack of a drainage system would most likely cause inconvenience, damage, inundation and health threat. For this reason, urban drainage deals with both types of water, aiming at a reduction of problems affecting human life and nature.

2.2.2. Terminology

The work of Viessmann and Lewis (1989:307) indicates that drainage systems have been developed from rather simple ditches to complex systems consisting of curbs, gutters, surface and underground conduits. Besides, Zoppou (2001:200) has drawn attention to the fact that urban drainage systems tend to be more complicated than rural ones as they need to account for numerous additional system features, namely roof top storage, open and natural watercourses, etc. Together with the rising complexity of these networks comes the need for a more specific knowledge about basic hydrologic and hydraulic terminology and the processes that take place.

In general, all storm sewer networks link stormwater inlet points (e. g. gullies, manholes, catch pits and roof downpipes) to either a discharge point or an outfall (Butler and Davies 2011:242). This usually happens through continuous pipes. The pipes can be differentiated based on their location in the network: While drains convey flows from individual properties, sewers are used to transport water from groups of properties and larger areas (Butler and Davies 2011:18). In contrast to sewers, the term 'sewerage' refers to all system infrastructure components, including manholes, pipes, channels, flow controls, pumps, retarding basins and outlets, to name a few.

As mentioned before, flow in the drainage system comes from the random input of rainfall runoff over space and time. Usually, the flows occur periodically and are hydraulically unsteady (Butler and Davies 2011:242). According to the magnitude of the storm and the characteristics of the system, the network capacity is loaded to a varying extent: in times of low rainfall, the runoff amount may be well below the system capacity, whereas during strong rainfall the system capacity may be exceeded. This may cause surcharge, pressure flow and even inundation (Chen et al. 2005:221). Surcharge in this respect means that a closed pipe or conduit that usually would behave as an open channel, runs full and consequently starts acting as a pipe under pressure. However, sometimes this is even desirable as it may enhance the capacity of the drain (Zoppou 2001:200).

Figure 5 presents a street that is linked to a drainage pipe via a manhole. Here, the capacity of the pipe is suitable to cope with the runoff volume. However, at this stage it needs to be emphasized that in case of limited intake capacity through the manhole, only a certain part of the runoff will enter the drainage system, despite sufficient pipe capacity (Mark et al. 2004:286). A situation of insufficient pipe capacity is shown in Figure 6. In this case, the rainwater flows from the drainage system to the street network, causing inundation. The duration of the inundation is determined by the intake capacity of the manholes, the pipe capacity, as well as by continuous losses, namely infiltration and evaporation in the area (Mark et al. 2004:286).

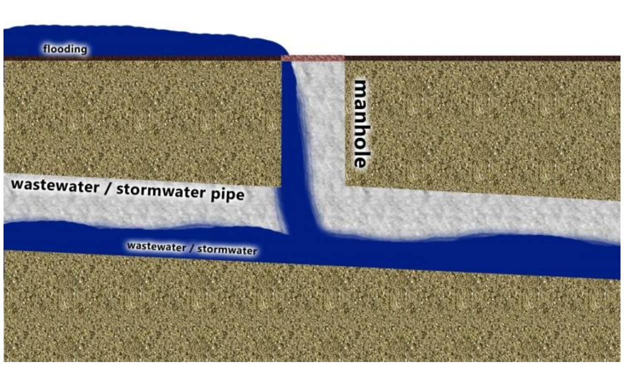


Fig. 5: Flooding from the street into a part-full pipe of a sewer system (authors' design, based on DHI 2007:17).

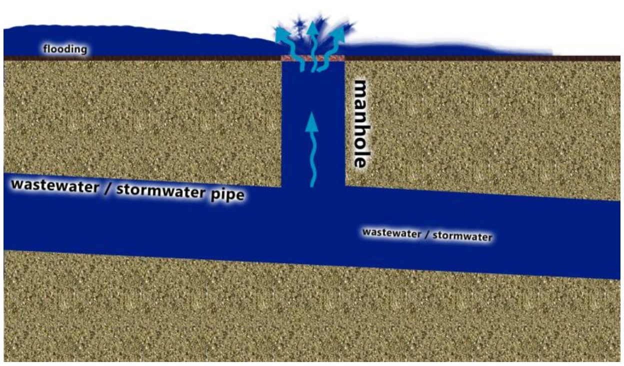


Fig. 6: Flooding from a surcharged drainage pipe to the street (authors' design, based on DHI 2007:18).

The district on the surface that actually adds to the water flow is called the 'contributing area'. Important characteristics of the contributing area are its size, shape, slope, land-use, roughness, soil type, wetness and storage. The two properties that impact most on the resulting water volume are catchment area and land-use (Butler and Davies 2011:245).

2.2.3. Types of Sewerage Systems

As described, urban sewerage systems need to cope with two different kinds of flows, namely stormwater and wastewater. As Butler and Davies (2011:18) perceptively state, despite all the progress that has been achieved, the relationship between the carrying of both flows is rather difficult to handle. Thus, according to the authors, there are very few systems with an ideal solution. The most common types of drainage systems encompass the combined, separate and hybrid sewer network as well as the dual system (Butler and Davies 2011:18, GEMETEC Limited 2008:4).

The combined sewer network is the most widespread system in Europe (Butler and Davies 2011:18). Thereby both stormwater and wastewater are conveyed together in one singe pipe. A schematic graph illustrating the flow path and interrelationships of this system type is given in Figure 7. As described in the legend, solid errors stand for intentional flows while dotted ones indicate unintentional flows. Moreover, thick boxes show flow sources and dashed thick boxes represent sinks. Two particular elements are the combined sewer overflows (CSOs) and the storage which the water runs through before it is drained to the water treatment plant (WTP). The purpose of these features is the following: While in dry weather, the pipes transport wastewater only, the amount of flow significantly increases during rainfall (up to hundred times the usual amount of wastewater) (Butler and Davies 2011:19). As it is not cost effective to provide drainage capacity for such conditions, the idea is to divert water above a certain level out of the drainage and into a natural watercourse. This is achieved by means of the CSOs: during heavy rainfall, an appropriate amount of the storm- and wastewater is conveyed to the WTP (continuation flow) whereas the excess is transported to a water course (spill flow) (Butler and Davies 2011:19ff). In case storage is available, some water flow may be retained for some time before it is discharged.

The second kind of sewerage system is the separate system. As the name implies, stormwater and wastewater are conveyed separately in two pipes, usually laid next to each other (Butler and Davies 2011:20). A schematic graphic is shown in Figure 8. It can be seen that, although the wastewater flow alters significantly during the day, the pipes are prepared to transport the maximum water volume all the way to the WTP. Also, due to the fact that the stormwater is not mixed with wastewater, the whole amount can be directly drained to a watercourse. On the other hand, the wastewater sewers transport the sewage directly to the WTP. While the size of the stormwater pipe is approximately the same as the size of the combined sewer pipe, the wastewater pipe is usually smaller (Butler and Davies 2011:20). One major problem of this kind of system is that wrong connections and cross-connections at various points may lead to the undesirable mixing of both flows (Butler and Davies 2011:26).

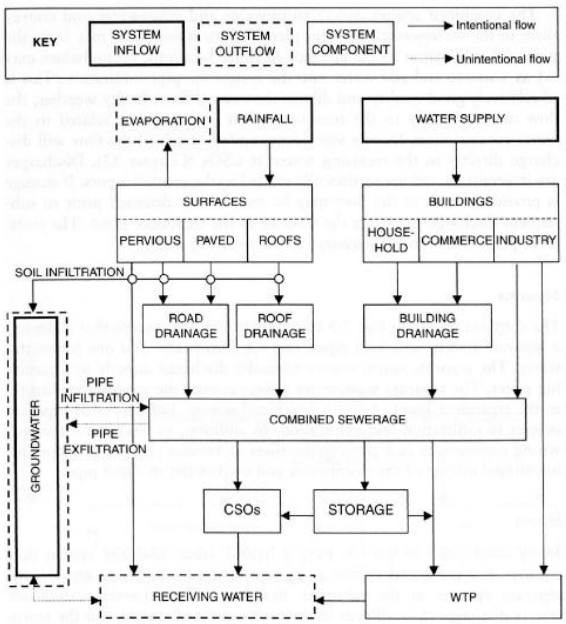


Fig. 7: Combined sewerage system (Butler and Davies 2011:25).

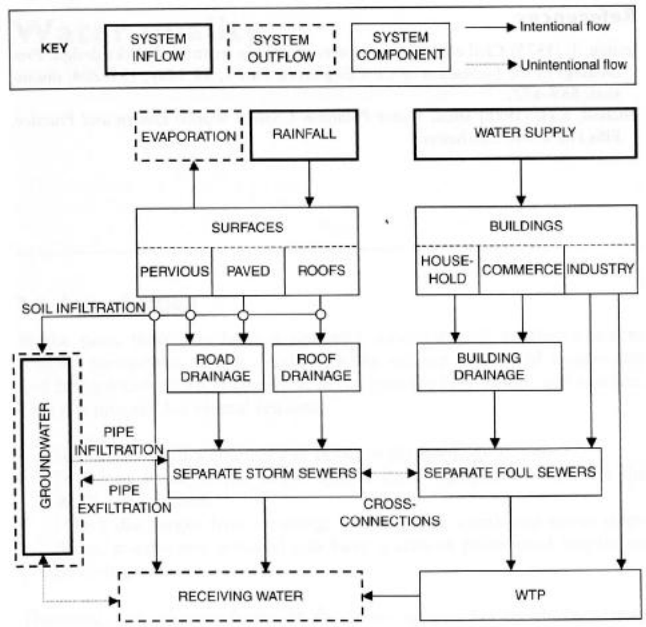


Fig. 8: Separate sewerage system (Butler and Davies 2011:27).

The third, so-called hybrid system is usually composed of a combined sewerage system (often the oldest parts at the core of the system) and a separate system (generally at the suburban periphery) (Butler and Davies 2011:26). In this case, the separate wastewater pipes drain into the combined system whereas the separate storm sewers discharge to receiving waters.

Last, some cities make use of a dual drainage system, consisting of a separate 'Major Drainage System' and a 'Minor Drainage System' (GEMETEC Limited 2008:4). The latter is designed to carry flows from minor storms, i. e. storms with low rainfall intensity and a more frequent return period of two to ten years. On the other hand, the major system is suitable to cope with runoff from major rainfall events. The resulting amount of these events would exceed the capacity of the minor system. While the minor system is built up of ditches, backyard swales, foundation drains and roof leaders, major systems usually comprise ditches, open drainage channels and roadways (GEMETEC Limited 2008:4).

2.3. Introduction to Urban Runoff and Drainage Modelling

In this section the idea of modelling urban stormwater is introduced. First, the term 'model' is specified and their purpose as well as their requirements and field of application in the urban context are explained. Second, major categories of models are outlined, facilitating a better comparison and understanding of different models. Last, important benefits of models are enumerated, followed by a couple of limitations that come with them.

2.3.1. Definition and Purpose of Models

Viessmann and Lewis (1989:493) found that despite the fact that numerous streams have been equipped with rain gauges to continuously measure flow characteristics, there are still many situations where planners are faced with no or insufficient streamflow information. As a consequence of this, mathematical approaches are necessary to create artificial flow sequences that help to investigate the area. Simple rules of thumb and rough formulas are usually not appropriate for this purpose (Viessmann and Lewis 1989:307). Thereforee, simulation models provide powerful tools to design and operate large-scale systems of flood control. The term ‘simulation’ in this context refers to a mathematical description of a system response to a given storm event. Thus, as the two authors have indicated, a simulation model can be specified as “(...) a set of equations and algorithms hat describe the real system and imitate the behaviour of the system” (Viessmann and Lewis 1989:498). These mathematical models consist of algebraic equations with known variables (parameters) and unknown variables (decision variables) (Loucks et al. 2005:60).

Generally, the purpose of sewer system models is to represent a sewerage system and to simulate its response to altering conditions. This helps to answer certain questions such as ‘What if...’ scenarios. Butler and Davies (2011:470) discovered that there have always been two main application domains for urban drainage models, namely the design of new sewers and the analysis of existing ones. When dealing with the design of a new system, the challenge is to determine appropriate physical characteristics and details to make the model responds satisfactorily in certain situations. On the other hand, when analysing existing sewers, the physical characteristics of the system have already been determined (Butler and Davies 2011:470). Hence, the planner wants to find out about how the system behaves under certain conditions concerning water depth, flow-rate and surface flooding. Based on the result, decisions can be made on whether the sewer system needs to be improved, and, if so, how.

Given these purposes, the model needs to meet a number of particular requirements. Zoppou (2001:200f) correctly points out that urban catchments react much faster to storm events than catchments in rural areas. Consequently, urban models have to be able to capture and deal with that rapid behaviour. Furthermore, a sewer flow model must be able to represent different hydrologic inputs (rainfall, runoff, sewer flow) and transform them into relevant information (flow-rate, depth, pressure). Thus the main physical processes that occur need to be represented. Butler and Davies (2011:471) conclude that the model has to be reasonably comprehensive: accurate results can only be derived if no important process is missed out. This also leads to the fact that substantial scientific knowledge about both hydrologic and hydraulic processes is required. Having said that, every model is also subject to simplification as the various interrelationships and feedbacks of reality are too complex to be represented. Based on this, Butler and Davies (2011:471) correctly argue that - at a general level - there are three factors that have a considerable impact on both the accuracy and the usefulness of a model:

- its comprehensiveness
- its completeness of scientific knowledge
- its appropriateness of simplification.

2.3.2. Model Classifications

The considerable amount of various simulation models that have been developed and applied provoked a range of different classification attempts. Some of these categorisations are outlined in the following. Turning to Viessmann and Lewis (1989:493), one finds that the terms stochastic and deterministic are commonly used to describe simulation models. The differentiation between these two types is rather straightforward: In case one or more of the model variables can be identified as being random and thus selected from a probability distribution, the model is of stochastic nature. Thereby model procedures are based on statistical properties of existing records and estimates, always producing a different model response. Apart from that, a model may be deterministic. These models can be considered as stochastic models where all random variables are replaced by their mean values (Zoppou 2001:199). Due to the lack of random parameters, deterministic methods will always deliver the same results for an identical set of input parameters. Viessmann and Lewis (1989:497) note that especially deterministic models have become popular for representing hydrologic behaviour of watersheds.

Research by Loucks et al. (2005:69) suggests that simulation methods can be classified into statistical or process-oriented models or even into a mixture of both. Viessmann and Lewis (1989:496) use for the same categorisation the terms 'descriptive' and 'conceptual'. Zoppou (2001:199) has expressed a similar view by differentiating between 'empirical' and 'conceptual' methods. According to the authors, statistical, descriptive and empirical models build up on data/field measurements; i. e. observed phenomena are taken into account. Because of this and due to their usage of basic fundamentals, including continuity or momentum conservation assumptions, these model types are of particular interest to hydrologists (Viessmann and Lewis 1989:496). On the other hand, process-oriented or conceptual models make use of knowledge on fundamental procedures that are happening. Thus, these models rely on theory to interpret certain phenomena, rather than representing physical processes. However, Loucks et al. (2005:70) observed that a number of models seem to combine elements of both categories.

Simulation models are also classified as physical and mathematical models. This in one of the earliest classification types (Viessmann and Lewis 1989:494). Consisting of analogous technologies and basic principles of similitude, physical methods are usually used for small-scale models. In opposition to this, mathematical approaches use mathematical statements to represent a system.

A fourth classification is achieved, considering temporal resolution. Thereby temporal resolution alludes to the smallest computational time step that is used in the model (Elliott and Trowsdale 2005:395). The time step may range from annual average to sub-hourly. Hence, models can be intended for single storm events only or for long term, continuous processes. Based on these differentiations, other authors distinguish between continuous and discrete models (Viessmann and Lewis 1989:494) and accordingly event and continuous process driven (Zoppou 2001:199). Thereforee it could be concluded that event models are short-term models, suitable for representing individual storms, whereas continuous simulations model the overall water balance of an area over a long period of time.

Moreover, catchment models can be classified as either dynamic or static. Dynamic models are suitable to simulate processes including changes over time or even time-varying interactions (Viessmann and Lewis 1989:496). Most hydrologic models belong to this category. Besides, simulation methods that deal with time-independent processes are referred to as static.

As a sixth and last classification method that is presented in this work, models can be separated into lumped and distributed. All three types describe how a particular model deals with spatial variability (Zoppou 2001:199). Models that ignore variations in parameters throughout an entire study area are called lumped parameter models (Viessmann and Lewis 1989:496). In contrast, distributed parameter simulations make allowance for behaviour variations from one location to another. As Elliott and Trowsdale (2005:395) have indicated, distributed simulations can be further split into quasi-distributed (model is divided into several elements such as sub-catchments) and fully distributed (often grid-based) parameter models. The majority of urban runoff simulations belong to the distributed category (Zoppou 2001:199).

2.3.3. Advantages and Limitations of Models

It is in the nature of modelling that some system behaviour may be misrepresented to a certain degree. This is mainly due to the mathematical abstraction of real-world phenomena (Viessmann and Lewis 1989:497). The extent of deviation, i. e. how much the outputs of the model and the physical system differ, depends on various factors. In order to identify differences and to verify the consistency of the model, testing needs to be performed. However, even verified models tend to have limitations which the user needs to account for during the analysis. Viessmann and Lewis (1989:497) claim that especially when applying models to determine optimal development and operation plans, simulation models may encounter limitations. This is because models cannot be used efficiently to develop options for defined objectives. Moreover, the authors argue that models tend to be very inflexible regarding changes in the operating procedure. Hence, laborious and time demanding programming might be necessary to investigate different operating techniques. Last, models carry a certain risk of over-reliance on their output. This is particularly a problem, as the results may contain considerable distortions and errors. The fact that models make use of simplifications and ignore random effects makes that clear (Butler and Davies 2011:470). Besides, there might be uncertainties associated with the input data or improperly chosen model parameters which cause biases.

On the other hand, simulations of hydrologic processes bring numerous important advantages and possibilities that need to be considered. Especially when dealing with complex water flows that include feedback loops, interacting components and relations, simulations may be the most suitable and most feasible tool (Viessmann and Lewis 1989:498). Once set up, they bring the advantage of time saving and non-destructive testing. Also, proposed modifications in the system design and alternatives can easily be tested, compared and evaluated, offering fast decision support. Butler and Davies (2011:470) have drawn attention to the fact that particularly stochastic models can be of great value in urban drainage modelling. This is due to the fact that randomness in physical phenomena allows for the representation of uncertainty, or the description of environments that are too complex to understand. Stochastic models account for this matter and provide an indication of uncertainty with their output.

2.4. Brief Summary

This part dealt with the general concept of urban rainfall and runoff, urban drainage infrastructure and various modelling categorisations. It was found that models and simulations in the field of urban hydrology imply the use of mathematical methods to achieve two things: Either imitate historical rainfall events or make analyses on possible future responses of the drainage system according to a specific condition. However, as was pointed out, the results of such models do not necessarily have to be correct. Rather, they should be considered useful and taken with a pinch of salt.

3. METHOD

This chapter is concerned with methodological approaches of modelling. It forms the theoretical base for the third chapter, which puts theory into practice. Starting with an introduction to the basic procedure in urban stormwater modelling, both the hydrological and the hydraulic model, as well as their relations and components are elucidated. In addition to this, basic input data and parameters for both models are illustrated. Subsequent to this, the two models are analysed in more detail. Therefore specific terms are explicated and the crucial procedure is outlined respectively. The last section is dedicated to the combination of both models by means of different coupling techniques.

3.1. Basic Procedure in Urban Flood Modelling

The following abstract describes the relation and derivation of the different elements in an urban runoff/ drainage model. However, it does not deal with the basic development and organisation of a simulation model, which addresses the phases of system identification, model conceptualisation and model implementation (including the model programming). A detailed description of these steps is provided by the work of Viessmann and Lewis (1989:498f).

Most urban stormwater models account for all pertinent phenomena and processes that occur in the physical runoff/ drainage system from the input (rainfall) to the output (outflow). Thus, on a very general level, these models consist of two basic components (Zoppou 2001:199): On the one hand there is rainfall runoff modelling, which is about the generation of surface runoff resulting from precipitation excess. On the other hand transport modelling is involved, including the routing of flows through the rainwater infrastructure. Figure 9 illustrates the general procedure and interstage products that are calculated by both models. As an addition to that, Figure 10 displays the elements and components of each model as well as their interactions. Besides, the hydraulic model is further divided into a pipe flow and a street flow model.

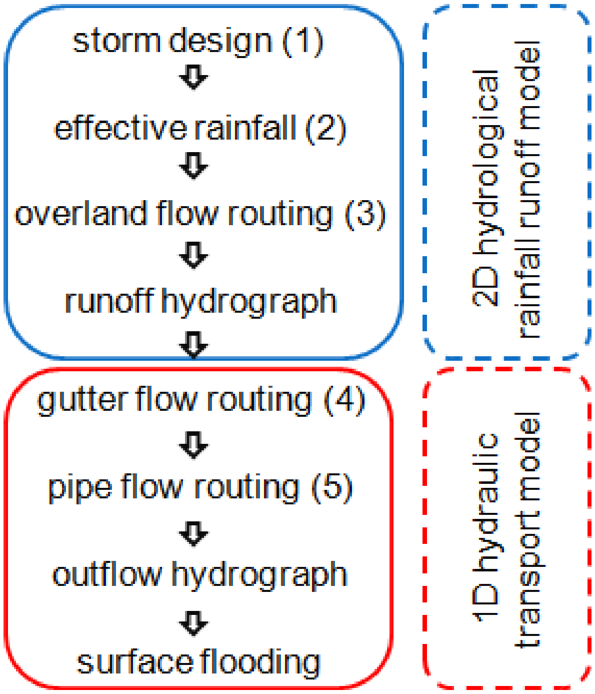


Fig. 9: General procedure of a 2D-1D runoff model (authors' design).

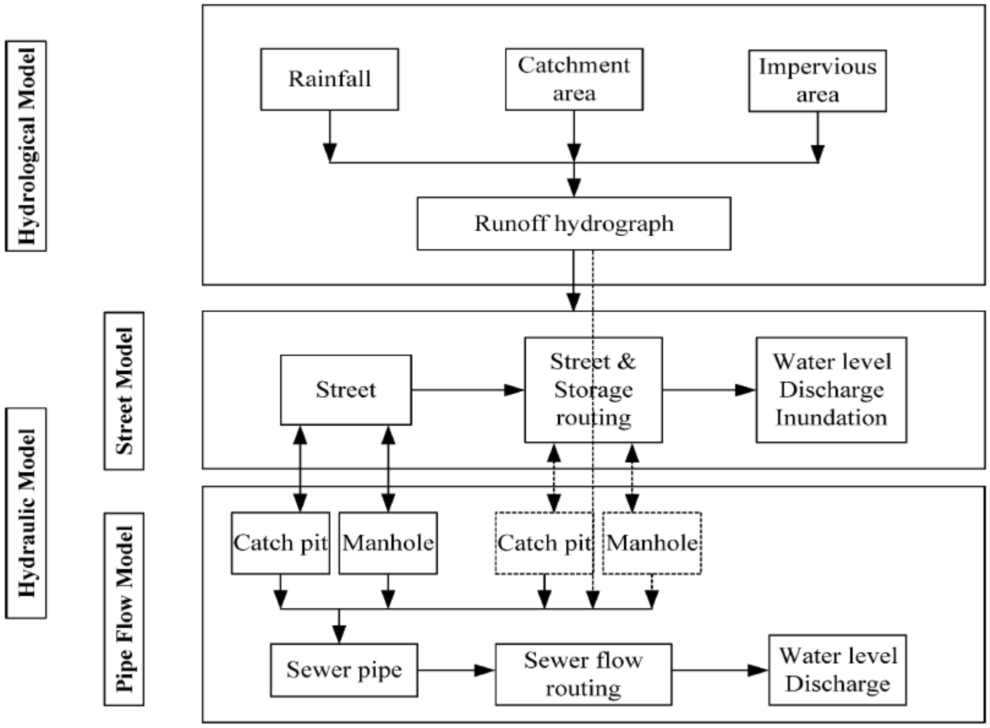


Fig. 10: Interactions between model components of an urban drainage system (Marks et al. 2004: 287).

The procedure is as follows: In a first step, a design storm needs to be determined. Simple examples of this would be constant rainfall, or, being more realistic, a rainfall event that has a certain storm profile, i. e. rain intensity variation over time (see chapter 1). This profile is created together with a particular return period by making use of IDF relationships. Second, losses need to be deducted from the design rainfall in order to calculate the excess rainfall rate. As described earlier, this is a complex process, because there are many aspects to be taken into account. However, the opinion of Mark et al. (2004:293) is that these losses must only be taken into account, if they affect the simulation. Hence, for evaporation the authors propose to compare accumulated evaporation to accumulated rainfall in order to decide, whether it should be included. Third, the remaining runoff (also known as overland flow or the initial flow of runoff) is routed to the gutter by making use of overland flow equations. The physical processes behind this procedure are very complex, including several irregularities on the surface that impact on the flows and thereforee need to be taken into account. In an article by GHKSAR (2000:50), flow routing is described as "(...) a procedure to determine the flow hydrograph at a point in a drainage system (...)". Further, the article notes that two methods can be distinguished, namely hydrologic and hydraulic routing. While in hydrologic routing the flow is estimated as a function of time based on upstream inflows and storage attenuation, hydraulic routing rates the flow as a function of time and space throughout the system (GHKSAR 2000:50). At the end, the described processes result in the transformation of the rainfall hyetograph into a surface runoff hydrograph. This hydrograph is the output of the 2D hydrological model and is usually available for every sub-catchment. It serves as input data for the 1D hydraulic model (Figures 9 and 10).

The 1D transport model is concerned with the gutter and sewer/ pipe flow routing. Thus - going back to the described process - as a fourth step, the gutter flow needs to be routed to the sewer inlet points.

Important at this stage is not so much the volume of water that will enter the pipe drainage system, but rather, how long it will take the flow to arrive at the inlet point (Butler and Davies 2011:472). In step five, pipe flow routing in the sewer system is operated. According to Butler and Davies (2011:472), many software packages represent the sewer system as a collection of 'links' and 'nodes'. In this case links refer to the sewer pipes, holding important hydraulic properties, such as the diameter, roughness, gradient, depth and flow-rate. In contrast, nodes stand for manholes that may cause head losses or changes of level.

The output of the hydraulic model is an outflow hydrograph. It generally consists of simulated changes in water depth and flow-rate for a certain time period at particular points in the sewer system and at its outlet points (Butler and Davies 2011:473). However, in case the capacity of the sewer system is exceeded by the amount of pipe flow, surface flooding becomes an additional component that needs to be included (Butler and Davies 2011:473). This encompasses surcharge modelling in the pipe system and inundation simulation on the surface. To do this, the model has to be capable of representing 2D-1D and 1D-2D interactions (compare to Fig. 5 and 6). The accuracy of the final model output strongly depends on the validity of simplifying assumptions that were taken as a basis (Viessmann and Lewis 1989:614).

3.2. Model Input Data

Depending on the complexity and elaborateness of the stormwater model, different kind and a varying amount of input data is required. Butler and Davies (2011:490) discovered that a conventional system generally makes use of the data presented in Figure 11. Otherwise, in any particular software package, the input data is that which exists for the respective model.

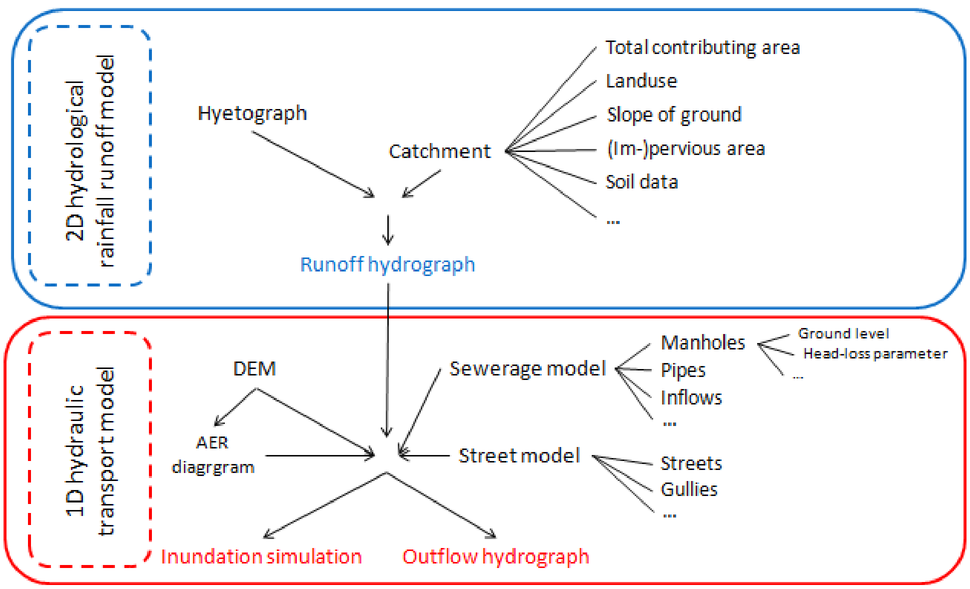


Fig. 11: Input data and parameters of a conventional stormwater model (authors' design).

As indicated in the figure, a hydrological rainfall runoff model may necessitate a hyetograph, defining the rainfall intensity over time, and catchment characteristics. Latter includes - amongst other things - information on the total contributing area, the portion of different land-uses, soil data and information on slope ground and pervious/ impervious areas. The hydraulic drainage model in turn utilises the generated runoff hydrograph, a digital elevation model (DEM) and a derived area-elevation-relation (AER) diagram, as well as a physical definition of the sewerage and the street network. An ordinary sewerage system, where manholes are connected through pipes, there are generally parameters to be set for each element. As Figure 11 exemplifies, for manholes, the parameters could comprise ground level information or a head-loss parameter.

In regard to the DEM, Butler and Davies (2011:481) perceptively state that there is often confusion on the difference between a DEM and a digital terrain model (DTM). Beyond that, Nielsen et al. (2008:2) refer in their work even to a further term - the digital surface model (DSM). In urban flood modelling, a DTM represents a topographic model of the ground surface only (Butler and Davies 2011:481). It forms the foundation for any flood model. In opposition to that, a DEM - equal to a DSM - builds up on the underlying DTM but additionally includes elevation information on buildings, trees, cars, and other prominent features (Nielsen et al. 2008:2). The purpose of the DEM is to represent land elevation data. This is in need of the estimation of flood volumes on the surface (Mark et al. 2004:288). Moreover, the resulting inundation map represents water levels that are usually based on the DEM. After all, a DEM is also used to develop a AER diagram. This diagram is required for surface topography, to allow for the definition of water store capacities during surface flooding (Mark et al. 2004:291). Hence, it becomes obvious that the accuracy of the final outcomes pretty much depend on the quality of the DEM. In regard to this, a study by Mark et al. (2004:288) shows that for urban flood analysis, a horizontal resolution of the DEM of 1x1 - 5x5 meters is suitable to represent important details, including the width of sidewalks, roads and houses. Though, as found by the authors, using a more detailed resolution (e. g. 1x1m) may only provide a better visual presentation, but does not necessarily bring more accurate results regarding the flood levels. In terms of elevation details (vertical resolution) Mark et al. (2004:288) claim that the interval should be in the range of 10-40cm. However, the work of Gourbesville and Savioli (2002:311) asserts that the DEM must even be able to represent variations of less than 10cm to be realistic. Anyhow, the resolution must be sufficient to cover crucial differences, particularly the distance between the road (= crest level of the manhole) and the curb level, as shown in Figure 12. If the vertical DEM resolution is not appropriate to account for these differences, the streets must be 'burned' into the elevation model (Nielsen et al. 2008:2). It is important that the street corresponds to the height of the curb, as streets and major roads act as drains for the surface runoff (see "gutter flow" in Figure 3). Thus, in case the water on the street swells above the curb level, water will run to adjacent areas and eventually cause flooding.

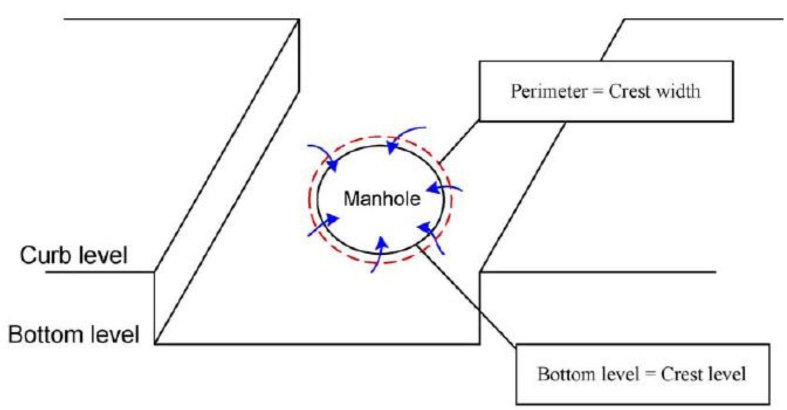


Fig. 12: Distance between curb and crest level (Marks et al. 2004:292).

3.3. 2D Hydrological Modelling in more Detail

As already mentioned, a hydrological model is used to transform a rainfall hyetograph into a surface runoff hydrograph. On a general basis, hydrological models can be differentiated into two distinct classes: surface runoff models and continuous hydrological models. While latter account for both the overland and the sub-surface runoff, surface runoff models only deal with runoff that appears on the surface (DHI 2011:59). Due to the fact that in urban runoff analysis the surface runoff model is the most commonly used model type, it will be subject to investigation in this abstract.

The computation of surface runoff resulting from precipitation excess encompass two main parts that need to be addressed by the surface runoff model: the deduction of initial and continuous losses and the surface flow routing, i. e. the reconversion of the effective rainfall into an overland flow and its passing to enter the drainage network (Loucks et al. 2005:439). For this purpose, a number of different methods can be applied; most of them make use of general catchment data (e. g. catchment area size, ground slope), model-specific catchment data and model-specific parameters (DHI 2010c:9). Thus, in order to be able to interpret the different outcomes, it is important to understand the background and functional principle of each model as well as the meaning of the main input parameters. In the following, some general facts on the deduction of losses are presented and main runoff model parameters and terms will be explained. Subsequent to this, most commonly used routing techniques will be outlined.

3.3.1. Deduction of Losses

The opinion of Loucks et al. (2005:439) is that the majority of models accounts for initial losses due to surface wetting and the filling of depression storage. Depression storage d (mm) may be described by the following formula:

$$d = \frac{k_1}{\sqrt{s}}$$

Formula 1: Representation of depression storage d (in mm) (authors' design based on Butler and Davies 2011:108).

Thereby k_1 represents the (im-)perviousness coefficient of the surface type (ranging from 0.28 for pervious to 0.07 for impervious surfaces); *s* stands for the ground slope. This means that depression storage is high for rather pervious areas and small ground slope. Concerning interception loss, Butler and Davies (2011:107) feel that in impervious areas - such as urban environments - the interception loss tends to be very small in magnitude (<1 mm) and thus is normally omitted or added to the depression storage. Then again, this also depends on the time of the year, the particular event and the degree of imperviousness: Initial losses are relatively unimportant during intense summer storms, but should not be neglected in less severe events or in less urbanised areas (Butler and Davies 2011:108). In the modelling process, it is common practice to subtract the sum of all initial losses right at the beginning of the storm to end up with the net rainfall.

Turning to continuous losses, these processes generally play an important role in urban catchments, especially in areas with large open spaces. Nevertheless, the effect of evapo-transpiration during short rainfall events tends to be marginal (Butler and Davies 2011:108). Hence, in most cases it is omitted or simply lumped into the initial losses. To exemplify this, in case of a heavy event (approx. 25 mm rainfall depth), when rain is falling on hot pavement (approx. 60°C), a maximum loss of 1 mm may occur (Loucks et al. 2005:439). In terms of infiltration, the loss rate is usually high at the beginning of an event but lowers exponentially to a final steady rate once the upper soil is saturated (Butler and Davies 2011:109). This is also expressed in a common formula that is used to represent infiltration: Horton's equation (Formula 2).

$$f_t = f_c + (f_o - f_c)e^{-k_{2t}}$$

Formula 2: Horton's equation to account for infiltration (authors' design based on Butler and Davies 2011:109).

In this formula, the infiltration rate at time t f_t (in mm/ h), the final infiltration rate f_c (in mm/ h), the initial rate f_0 (in mm/ h) and the decay constant k_2 (in h^{-1}) are included. The respective parameters mostly rely on the soil/ surface type as well as the initial moisture content of the soil.

3.3.2. Important Terms

There are several terms and parameters that are frequently used in the context of hydrological modelling and routing processes. Thus, to facilitate a better understanding of the techniques described in the following abstract, some definitions are given below.

When setting up a surface runoff model, an important parameter is the catchment delineation that needs to be defined. This means that the boundaries of the complete catchment (as well as of each sub-catchment) have to be determined. This needs to be done in a way that any rain falling within the determined area will be directed (by reason of gravity) to a particular point of the local drainage channel. Thereforee both topography and the drainage network need to be understood. Butler and Davies (2011:246) state that this is feasible with appropriate accuracy either by means of field work or the usage of contour maps. However, as Mark et al. (2004:290) rightly point out, difficulties arise in flat areas, where the boundaries are blurry. In this case the authors propose to compare the results of the model with those in real life and to make according adjustments. Beside this manual procedure, there is also a way to automatically delineate the catchment. Three common methods are available (Mark et al. 2004:290):

- Distance-based approach. Areas are allocated based on their distance to the drainage network
- DEM-based approach. Algorithms are used to calculate the most probable flow paths depending on terrain aspects and slopes in the DEM
- DEM plus cover image. Equal to the 'DEM-based' approach, but with additional information on land-use from the digital image.

A further important parameter - as described earlier - is the type of land-use accompanied by different degrees of impermeability. This factor impacts a lot on the amount of loss, respectively runoff. Thus, once the total catchment area has been determined, the extent and type of the surfaces that drain into the system need to be enquired. A measuring unit for quantifying impervious surfaces is the percentage imperviousness (PIMP) (Butler and Davies 2011:247). It can be derived manually from maps or automatically from aerial pictures. Another way to estimate the PIMP is to make use of the density of housing in the area. Thereforee the following formula may be used, where J represents the housing density (dwellings/ha) (Formula 3).

$$PIMP = 6.4\sqrt{J} \qquad 10 < J < 40$$

Formula 3: Percentage imperviousness derivation (authors' design based on Butler and Davies 2011:247).

In addition, the runoff coefficient *C* is a parameter that is widely used in surface runoff models. The dimensionless parameter represents the proportion of rainfall that actually adds to the surface runoff (Butler and Davies 2011:247). Its value is dependent on the imperviousness, the slope and other retention properties of the ground. Further, the work of GHKSAR (2000:42) shows that *C* is influenced by soil characteristics and conditions, vegetation cover and the rainfall intensity, too. Based on the assumption that impervious surfaces contribute 100 % of their rainwater to the runoff and pervious surfaces add 0 %, the coefficient can be defined as $C = PIMP/100$ (Butler and Davies 2011:247). Although *C* has to be somehow related to PIMP, it does not necessarily have to be the same, as some runoff may originate from pervious surfaces as well. Commonly used runoff coefficient values are displayed in Table 1. When adopting the given values, it is important to investigate and ascertain the respective conditions on the ground beforehand. This is particularly necessary when dealing with unpaved surfaced, as both uncertainties and variability properties tend to be large in this case (GHKSAR 2000:42).

Surface Characteristics	Runoff coefficient, *C*
Asphalt	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Grassland (heavy soil)	
Flat	0.13 - 0.25
Steep	0.25 - 0.35
Grassland (sandy soil)	
Flat	0.05 - 0.15
Steep	0.15 - 0.20

Table 1: Runoff coefficient values for particular surface types (author's design, based on GHKSAR 2000:42).

Last, the expression - time of concentration t_c - often appears in the hydrological modelling context. It describes the time for surface runoff to flow from the remotest point in the catchment to a point being under consideration, e. g. the outlet (GHKSAR 2000:42). Thereforee each location in the catchment has a different time of concentration. Butler and Davies (2011:249) further differentiate between two components of t_c: the overland flow time, also called the time of entry t_e and the sewer flow time, known as the time of flow t_f. T_e may differ considerably, depending on slope, surface roughness, length of the flow and rainfall characteristics (e. g. return period: rarer, stronger storms result in more water in the catchment, thus faster t_e).

3.3.3. Routing Techniques

Research by Butler and Davies (2011:115) suggests that there are currently two types of overland flow routing techniques that are widely applied, namely the unit hydrograph method and the kinematic wave model. The former approach is implemented in numerous different ways and belongs to the group of linear reservoir models; the kinematic wave technique is part of the non-linear reservoir methods. Both techniques and some particular parameters are specified in the following, whereat particular emphasis is put on the unit hydrograph technique.

The unit hydrograph refers to the relation between net rainfall and direct runoff (GHKSAR 2000: 44). It is founded on the assumption that effective rain falling over a certain area produces a unique and time-invariant hydrograph. More precisely, the method describes the out-flow hydrograph that results from a unit depth (usually 10 mm) of effective rainwater that falls equally over a catchment area (Butler and Davies 2011:115). This happens at a constant rainfall rate *i* for a certain duration *D*. For this reason, the system is considered linear and time-invariant. The respective response is the *D-h* hydrograph that is illustrated in Figure 13. The Y-axis of the *D-h* unit hydrograph is given as *u(D, t)* - the volume of effective rain at any time *t*. The duration *D* is usually 1 hour but can be changed to any time period. Using the unit hydrograph as a base, it is possible to develop a hydrograph response to any rainfall event. However, to do so, the following conditions need to be fulfilled (Butler and Davies 2011:115f):

- the time base of the unit hydrograph is constant, no matter how the rain intensity changes
- the ordinates of the runoff hydrograph are proportional to the amount of effective
- the response to successive blocks of effective rainfall, each starting at particular times, may be obtained by summing the individual runoff hydrographs starting at the corresponding times.

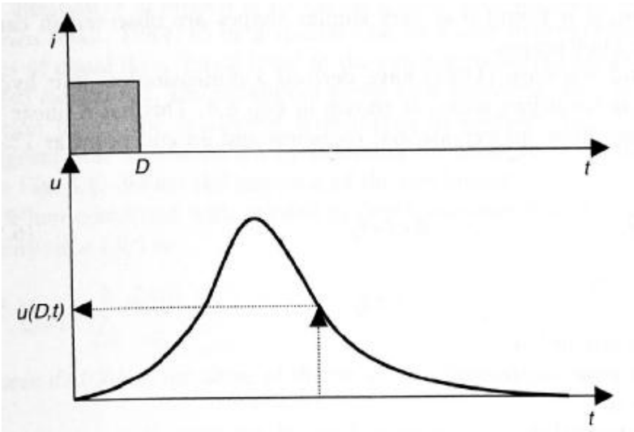


Fig. 13: The unit hydrograph (Butler and Davies 2011:115).

Consequently, the direct runoff caused by any net rainfall with different depths for successive increments of a certain duration may be retrieved by a linear superposition of the responses of the different net rainfall depths (GHKSAR 2000:44). This approximate approach is known as convolution and can be described by the following Formula 4:

$$Q(t)=\sum_{w=1}^{N} u(D,j)\, I_{\omega}$$

$$Q(t)=u(D,t)\,I_1+u(D,t-D)\,I_2+\ldots.+(D,t-(N-1)D)\,I_n$$

Formula 4: Equation to process convolution (authors' design based on Butler and Davies 2011:116).

While $Q(t)$ is the runoff hydrograph ordinate at the time t (in m³/ s), the term $u(D, j)$ represents the *D-h* unit hydrograph ordinate at time j (in m³/ s). Further, I_w stands for the rainfall depth in the *wth* of N blocks of the duration D (in m). The variable j (in s) can be written as $t-(w-1) D$. Thus, the formula allows calculating the runoff $Q(t)$ at a certain time t of a rainfall event that has n blocks of rainfall with a duration D.

As GHKSAR (2000:44) has indicated, the usage of the unit hydrograph method requires a loss model and a unit hydrograph. In regard to the losses to infiltration, the unit hydrograph model assumes that they can be either described as a fixed initial and constant loss (by the Ø-index), as a constant proportional loss (by the runoff coefficient), or as a fixed initial and continuous loss (by the SCS curve). Turning to the unit hydrograph, this can be derived for a particular catchment through rainfall-runoff monitoring. However, if there are no gauges for measurement, it may be predicted, based on catchments with similar characteristics. Butler and Davies (2011:116) propose three methods to do this: a synthetic unit hydrography, reservoir models and the time-area method. The latter method will be specified in the following.

In case of the time-area method, lines are delineated that define equal time of flow travel (isochrones) from a catchment's outfall point. Thus, referring to the abstract that dealt with important terms in the previous chapter, the maximum flow travel time, i. e. the most remote line from the outfall represents the time of concentration of the catchment. By adding up the areas between the different isochrones, a time-area diagram can be created, as presented in Figure 14 (lower graphic). This diagram defines the catchment's runoff response. Describing the whole process from a conceptional point of view, the runoff process in the catchment is split into several cells in the form of concentric circles, as shown in the upper graphic of Figure 14 (DHI 2010c:13f.). The area of each cell depends on the respective time-area curve that is used (e. g. rectangular, divergent, convergent); it should be chosen to suitably represent the shape characteristics of the catchment. At each time step after the runoff started, the accumulated water amount from every cell moves to the neighbouring cell in downstream direction (DHI 2010c:14). Hence, the water volume of each cell is derived as a balance of inflow, outflow and the current rainfall on the cell (multiplied with cell area). The outflow of the last cell in the catchment (outflow) represents the runoff hydrograph of the catchment.

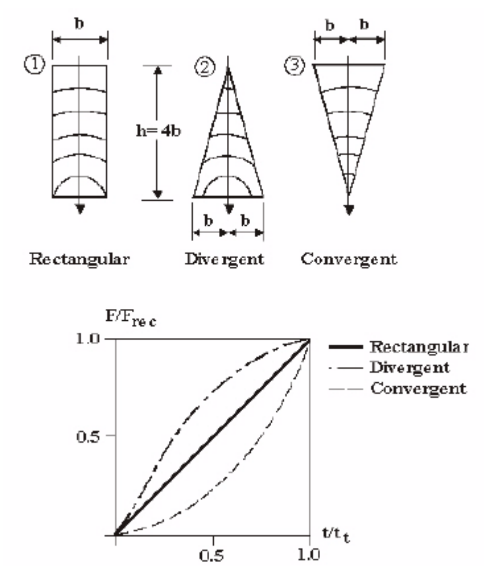


Fig. 14: Three examples of a time-area curve (DHI 2010c:14).

Turning to the kinematic wave method, a more physically-based approach is presented that aims at simplifying and solving equations of motion (Butler and Davies 2011:129). Using this method, the surface runoff is calculated as an open, uniform channel flow, being subject to gravitational and friction forces only. The amount of runoff thereby depends on numerous hydrological losses as well as on the size of the contributing area (DHI 2010c:17). Thus, the shape of the runoff hydrograph is influenced by surface parameters of the catchment (including length, roughness, and slope) which are part of the kinematic wave computation. A widely used formula of the kinematic wave method is the Manning equation in Formula 5 (Loucks et al. 2005:432). It can be used to estimate the average velocity *V* (in m/ s) and the amount of flow *Q* (in m^3/s) for a given cross-sectional area *A* (in m^2). As illustrated, the velocity is influenced by on the hydraulic radius *R* (in m) and the slope bed *S* of the channel as well as by a friction factor n (dimensionless). Typical values for the friction factor according to particular channel material are given in Table 2.

$$V = (R^{2/3} S^{1/2})/n$$
$$Q = AV$$

Formula 5: Manning's equation to predict open-channel flow velocity (authors' design based on Loucks et al. 2005:432).

Channel material	*n* range
Glass	0.009 - 0.013
Cement	0.010 - 0.015
Concrete	0.010 - 0.020
Brickwork	0.011 - 0.018

Table 2: Typical values of Manning's *n* (authors' design, based on Butler and Davies 2011: 172).

3.4. 1D Hydraulic Modelling in more Detail

The following abstract covers some basic hydraulic principles that need to be taken into account when dealing with hydraulic questions in the context of sewer flow modelling. Further, different categories and types of drainage flows as well as their specific properties are characterised and common approaches for the hydraulic modelling process are described.

3.4.1. Basic Hydraulic Principles and Terms

A fundamental condition in various matters of urban sewerage is the continuity of flow. This means that in a section of conduit with a constant diameter and no cross-sections, the mass of liquid that flows into the conduit at a point one must be equal to the mass that discharges at a point two (Butler and Davies 2011:147). Based on the assumption that the density, i. e. mass per unit volume stays the same, the volume entering must be the same as the volume flowing out. A second condition that builds up on the continuity of flow is the continuity of flow-rate: the flow-rate (in m^3/s or l/s) $Q_1 = Q_2$. However, when looking at the velocity of the liquid in the pipe, there is a difference across the flow cross-section. The maximum velocity can be detected in the centre of a pipe (Butler and Davies 2011:148). To make it easier to cope with this situation, the mean velocity *v* is defined as the flow-rate per unit area *A* that is passed by the flow: $v = Q/A$.

Important parameters that affect the flow in the sewage system are the roughness and the shape of the pipes. Depending on the material, condition and age of the pipe, there are different values of roughness k_s. Common k_s values for specific pipe materials are given in Table 3. Thereby the condition 'new' refers to new, clean and well-aligned conduits. In contrast, 'old' stands for pipes that are influenced by biological slime and thus a higher degree of roughness.

Pipe material	**k_s range (in mm)**	
	new	old
Clay	0.03 - 0.15	0.3 - 3.0
PVC-U	0.03 - 0.06	0.15 - 1.50
Concrete	0.06 - 1.50	1.5 - 6.0
Fibre cement	0.015 - 0.030	0.6 - 6.0
Brickwork - good condition	0.6 - 6.0	3.0 - 15
Brickwork-poor condition	-	15 - 30

Table 3: Common values of roughness (authors' design, based on Butler and Davies 2011:158).

The shape of a pipe may vary, depending on different conditions such as country, ground properties and other things. Figure 15 displays some common shapes (including a circular, egg-shaped, horseshoe and U-shaped form) as well as their respective geometric and hydraulic properties. In most sewerage systems, circular-shaped pipes are used (Butler and Davies 2011:167). In terms of geometry, this shape offers the shortest circumference per unit of cross-sectional area. For this reason, circular shaped pipes require least wall material to stand both internal and external pressure.

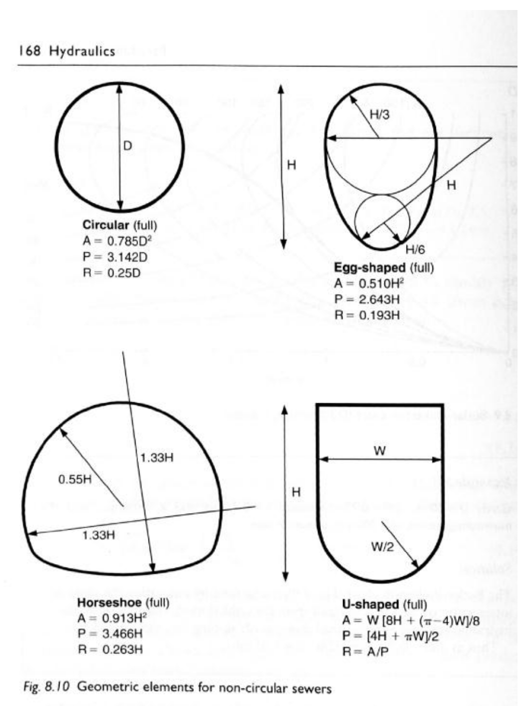


Fig. 15: Shapes and geometric properties of sewers. (Butler and Davies 2011:168).

Depending on the type of drainage conduit as well as on the amount of water that flows within it, some pipes are exposed to water pressure. In general, pressure may be defined as force per unit area. It is expressed in (kN/ m²) or (bar), at which 1 (bar) is equal to 100 (kN/ m²). In the hydraulic context, pressure needs to be further differentiated into absolute pressure, which refers to pressure relative to vacuum, and gauge pressure, i. e. pressure relative to the pressure in the atmosphere (Butler and Davies 2011:147). In hydraulic equations and calculations the latter type is usually used. Pressure in a liquid grows with vertical depth according to the following Formula 6:

$$\Delta p = pg \Delta y$$

Formula 6: Components of pressure growth (authors' design, based on Butler and Davies 2011: 147).

Thus, an increase in pressure *Δp* (in N/ m²) depends on the product of the liquid density *p* (in kg/ m³), the gravitational acceleration *g* (9.81 m/ s²) and the increase in depth *Δy* (in m). In order to express the energy level of a flowing liquid at any point within the network, three components need to be taken into account (Loucks et al. 2005:432):

- Pressure
- Velocity
- Potential.

These three aspects are commonly represented by a so-called energy head - energy per unit weight - as indicated in Formula 7. Accordingly, the pressure head is pressure per specific weight (N/ m²) of the liquid, the velocity head is the quotient of the squared velocity (in m/ s) and twice the gravitational acceleration and the potential head is the elevation above some base elevation (in m). The total head *H* consists of the sum of all three head components.

$$pressure\ head\ \frac{p}{pg} \qquad velocity\ head\ \frac{v^2}{2g} \qquad potential\ head\ z$$

Formula 7: Representation of pressure head, velocity head and potential head (authors' design, based on Butler and Davies 2011:150).

The energy level of any flow constantly changes. It may increase e. g. through pumps or decrease, e. g. due to friction. In terms of the energy head, this is described through head gains HG and head losses HL (Loucks et al. 2005: 432). Figure 16 and 17 point up the energy components for open channel flow and along a pressure pipe respectively. As indicated by the figures, the hydraulic grade line HGL represents the sum of pressure head and elevation head. In open-channel flow the HGL coincidences with the water surface slope as the pressure head at the surface is zero (Figure 16). However, for a pressure pipe, the hydraulic head can be expressed as the height to which a water column would build up in a so-called piezometer - a tube rising from the conduit (Figure 17) (Loucks et al. 2005:432). Turning to the second line, the energy grade line EGL, the sum of the hydraulic grade and the velocity head is described. Loucks et al. (2005:432) make clear that the EGL represents the height up to which a water column would rise in a Pitot tube, accounting for the velocity of the fluid.

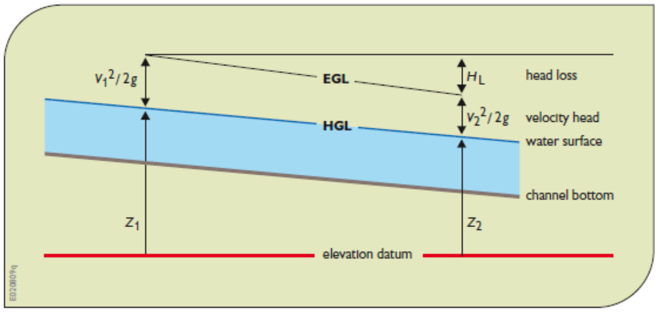


Fig. 16: EGL and HGL for an open channel (Loucks et al. 2005:431).

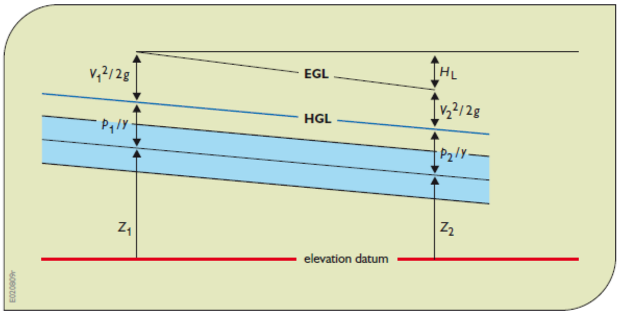


Fig. 17: EGL and HGL for a pipe flowing full (Loucks et al. 2005:432).

Another aspect that is represented in the last two graphics is the head energy loss. It consists of friction losses and local losses. Butler and Davies (2011:151) report that friction losses are the result of forces between the liquid and the wall of the pipe. Hence, this type of loss occurs along the whole length of the pipe. In contrast to this, local losses evolve from local disruptions to the flow e. g. due to features such as bends or altering in the cross-section. The sum of both components forms the head loss h_L.

3.4.2. Types of Drainage Flows

In the field of hydraulic modelling there are two main types of flow which engineering hydraulics focus on, namely open-channel flow and flow under pressure (Butler and Davies 2011: 146). Besides, there is a hybrid type, i. e. a combination of both types which is referred to as part-full pipe flow. This is the most common type of flow in sewer systems (Butler and Davies 2011:146). Each type of flow has special characteristics that need to be accounted for in the modelling process.

In order to consider flow to be under pressure, the liquid flowing in the pipe has to fill the whole cross-section of the conduit, i. e. there is no free surface for the whole length of the conduit. In such cases, the flow is also described as surcharged flow. As Butler and Davies (2011:479) point out, that it is quite common for conduits in a drainage system to be surcharged. As an example, surcharge may occur if flood volumes exceed the design capacity of a pipe. Figure 18 presents a longitudinal vertical part of a conduit that experiences surcharge and therefore carries the maximum flow-rate. In case of an increase of water entering the sewer system, the capacity of the conduit cannot be raised by an increase in flow depth. As the capacity of a conduit depends on its diameter, roughness and the hydraulic gradient, the only way to further increase it is through a rise of the hydraulic gradient (Butler and Davies 2011:469). Thus, once the hydraulic gradient rises above ground level, manhole surcharge and surface flooding may occur.

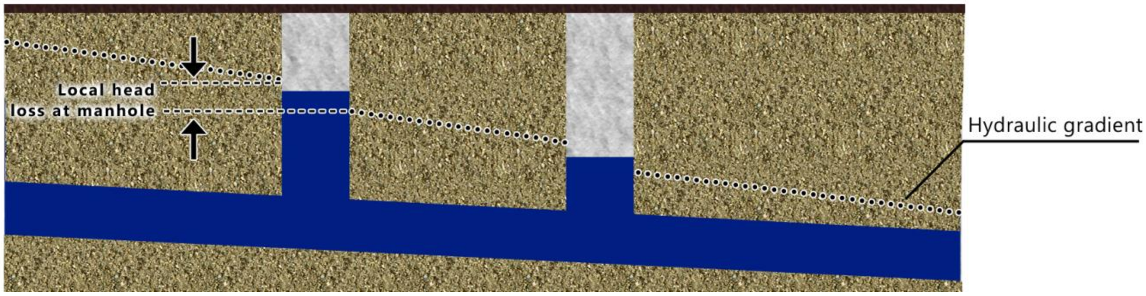


Fig. 18: Profile of a pipe flow with surcharge (author's design, based on Butler and Davies 2011:169).

Looking at the second type of flow - open-channel flow - liquid carried in a channel flows by gravity forces with a free surface at atmospheric pressure condition. In this case, the hydraulic cross-section alters with the flow (GHKSAR 2000:45). As already explained in the last chapter, the velocity of open-channel flow can be estimated by means of the Manning's equation (Formula 5). Open-channel flow can be divided into uniform and non-uniform flow. The former type of flow consists of a normal flow depth, meaning that the total energy line, the hydraulic grade line and the channel bed are all parallel to each other (Butler and Davies 2011:173). However, this is rarely the case, as there usually exist alteration in the pipe slope, the pipe diameter, or the roughness of the pipe, preventing the energy lines from being parallel. Typically, in an urban sewerage, there is a combination of uniform and non-uniform flow.

Part-full pipe flow, as a combination of both main flow types, is defined as liquid in a conduit that flows by gravity, with a free surface (Butler and Davies 2011:146). It is presented in Figure 19. The liquid may even fill the whole cross-section of the pipe if its flow-rate is equal or higher than the designed pipe capacity. Butler and Davies (2011:172) found that for this reason, theories on part-full pipe flow are traditionally more related to those for pipes flowing full, though it is rather a special case of open-channel flow.

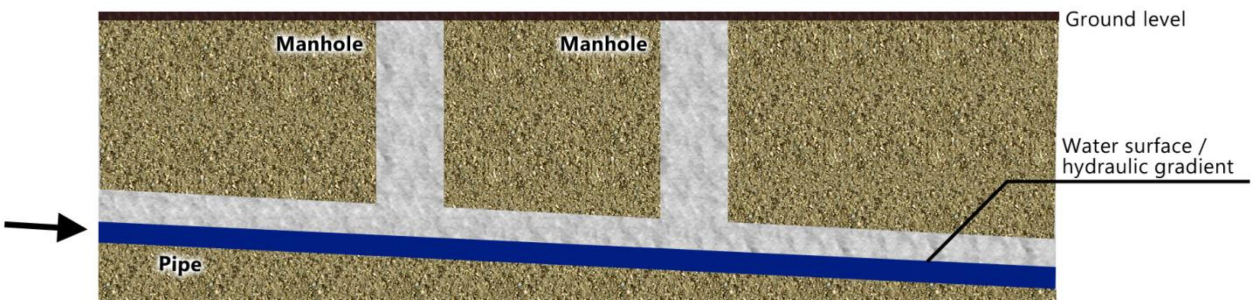


Fig. 19: Profile of a part-full pipe flow (author's design, based on Butler and Davies 2011:169).

In addition to the different types of pipe flow, the flow in a sewerage can be classified into steady and constant/ uniform, and accordingly unsteady and non-uniform. Steady in this case means constant with time, while uniform refers to constant with distance (GHKSAR 2000:46). Respectively, unsteady means not constant with time and non-uniform means not constant with distance. Thus, the following hydraulic conditions exist:

- uniform and steady
- uniform and unsteady
- non-uniform and steady
- non-uniform and unsteady.

The flow in the sewers generally tends to be unsteady: stormwater alters during a storm event, whereas wastewater changes with the time of day. However, the opinion of Butler and Davies (2011:149) is that it seems to be unnecessary to account for this altering in a lot of calculations and for reasons of simplicity, conditions can be treated as steady.
As another property of the pipe flow, the motion can be distinguished into laminar and turbulent. The viscosity of a liquid, i. e. the property that prevents motion, is caused by interactions of fluid molecules that generate friction forces amongst the different layers of fluid flowing at different speed (Butler and Davies 2011:149). If the velocities are high, there is an erratic motion of fluid particles, leading to turbulent flow (GHKSAR 2000:45). On the other hand, low velocities result in laminar flow.

3.4.3. Modelling Sewerage Behaviour

As it was stated above, flow in the sewerage is generally unsteady, i. e. it varies with the time of day. Besides, the flow tends to be non-uniform due to friction and different head losses along the conduit. For this reason, the representation of unsteady, non-uniform flow is an important matter when modelling sewerage behaviour. As Butler and Davies (2011:474) have indicated, there are numerous methods available to model and analyse unsteady, non-uniform flows in a sewerage system. Thereby some methods are founded on approximations, whereas others pursue a complete theoretical investigation of the physics of the flow. A very common theoretical method that can be used for gradually-varied unsteady flow in open channels or part-full pipes is presented by the Saint-Venant equations. In order to make use of the method, the following conditions must apply (Butler and Davies 2011:475):

- there is hydrostatic pressure
- the bed slope of the sewer is very small such that flow depth measured vertically tends to be the same as that normal to the bed
- there is a uniform velocity distribution at the cross-section of a channel
- the channel is prismatic
- the friction losses computed by a steady flow equation is also valid for unsteady flow
- lateral flow is marginal.

The Saint-Venant equations consist of a dynamic and a continuity equation and can be used to represent the development of water depth *h,* discharge *Q* and the mean flow velocity *v* (Vojinovic and Tutulic 2008:184). The dynamic equation is defined in Formula 8:

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{v}{g} \cdot \frac{\partial y}{\partial x} - \frac{1}{g} \cdot \frac{\partial y}{\partial t}$$

1

2

3

Formula 8: Dynamic equation part of the Saint-Venant equations
(author's design, based on Butler and Davies 2011: 474).

Thereby y represents the flow depth (in m), v the velocity (in m/ s), x the distance (in m), t the time (in s), S_o the bed slope and S_f the friction slope, respectively without any unit. The formula illustrates the three components that form the equation. The first part does not account for changes with distance or time and thus is valid for steady uniform conditions. By including the second term, alteration with distance is factored in. Finally, the whole equation takes account of variation with time and non-uniform conditions. Formula 9 represents the same equation but transformed according to flow-rate rather than velocity. Besides, the Continuity equation is given. Q stands for the discharge/ flow-rate (in m³/ s), the variable A defines the area of flow cross-section (in m²) and the term B represents the water surface width (in m).

$$\underbrace{\frac{\partial Q}{\partial t}}_{\text{Local acceleration term}} + \underbrace{\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right)}_{\text{Convective acceleration term}} + \underbrace{gA\frac{\partial y}{\partial x}}_{\text{Pressure force term}} - gA(\underbrace{S_o}_{\text{Gravity force term}} - \underbrace{S_f}_{\text{Friction force term}})=0$$

$$B\frac{\partial y}{\partial t}+\frac{\partial Q}{\partial x}=0$$

Formula 9: Momentum and Continuity equation (author's design, based on GHKSAR 2000: 51 and Butler and Davies 2011: 475).

In short, the work of WeiFeng et al. (2009:1615) reveals that the numerical solution above is derived from a finite differential formulation of the equations according to changing flow-rate and water depths.

3.5. Types of Model Coupling

In regard to the coupling of a hydrological and a hydraulic model, there are different ways to define the relationship between the two models. To exemplify this, the two models can be coupled in a loose or tight manner, in a sequential or a simultaneous mode and it is feasible to either couple a 1D hydrological with a 1D hydraulic model (1D-1D) or a 2D hydrological with a 1D hydraulic model (2D-1D). These possibilities are outlined in the following.

Sui and Maggio (1999:36) have drawn attention to the fact that there is a difference between loose and tight coupling. On a general basis, the terms 'loose' and 'tight' are a qualitative description of the coupling. Processes in models are said to be tightly coupled, if there is a high degree of non-linearity. This means that the interaction between the two models cannot be split into distinct processes which can be calculated independently from each other and again combined afterwards. Thereforee tightly coupled models may be described as a single unit. In contrast to this, when defining loosely coupled models, each model makes only little use of the calculations and outcomes of the other one. Thus, there is only a small dependency amongst them and the two models may even be derived from two different software packages (Butler and Davies 2011:484). As a consequence, the models can be run independently from each other and outcomes can be combined afterwards in a final solution. Taking this into consideration, the decision for a sequential or a simultaneous coupling mode depends to some degree on the strength of coupling. Turning to WeiFeng et al. (2009:1615), one finds that in a sequential coupling mode, the hydrological runoff model can be seen as a pre-processor, to create input data for the hydraulic drainage model. Opposed to that, the simultaneous mode coupling implements real-time interactions between the models. As a consequence of this, it may be inferred that a tightly coupled model is necessary in order to run the models in a simultaneous way.

As a last method that is described in this work, there is 1D-1D and 2D-1D coupling. For each approach both loose and tight coupling is possible. Mark et al. (2004:291) note that in traditional modelling approaches, surcharge water that flew from the sewerage system onto the streets was virtually stored in a reservoir and returned back to the pipe system once there was sufficient capacity again. Butler and Davies (2011:484) observed that the next level of complexity is a loose coupling of a 1D hydrological pipe model with a 1D flow representation on the urban surface. Thus, the surface is represented as a series of interconnected open channels and ponds where the processes are calculated in a similar way as for the pipe hydraulics. Interactions between flows on the ground and those under the ground are disregarded. Applying a more tightly coupled approach, the 1D-1D coupling is considered to result in an appropriate presentation of surface inundation, provided that the flow does not spill over the streets (Butler and Davies 2011:484f.).

In order to simulate a surcharge-induced flood in an urban area, a 1D-2D coupling approach is required (Hsu et al. 2000:22). This allows for a more accurate prediction of the development of flooded zones and the estimation of flood depths. Especially when making use of tightly coupled 1D-2D models, the most accurate results can currently be achieved (Butler and Davies 2011:486). However, this is achieved at the expense of computational power, calculation time and data requirements.

3.6. Brief Summary

In can be concluded form this chapter that both the hydrologic overland flow model and the 2D hydraulic flow model respectively conist of holistic formulae. This enables the user of these models to account for various kind of detaild information concerning the rainfall event, the catchment characteristics and the drainage system. At the same time, this requires a lot of specific knowledge and experience in order to find the right constellations of parameters. Besides, the great number of adjustment possibilities should not mislead over the fact that models are incapable to include every detail.

4. APPLICATION

The following chapter presents the practical part of the work. It describes the software which is used as well as the various parameters which are chosen for the exemplary study area. In the end a short summary and a critical reflection of the software are provided.

4.1. DHI Software

The decision about the software was made in favour of DHI which stands formerly for "Danish Hydraulic Institute". DHI offers for urban flood simulation two specific software packages that may be used. The first one and likewise the major one is called MIKE URBAN, the second one is named MIKE FLOOD.

Generally, DHI has well-developed software packages with different objectives respectively field, such as environment, water and health. Furthermore, the company offers a range of consulting services. DHI describe itself as an independent and international consulting and research organisation with leading edge technologies, software tools, laboratories (chemical and biological) and physical model test facilities (DHI 2012b).

4.1.1. MIKE URBAN

MIKE URBAN is used for modelling the urban water (drainage) network which covers the sewers, the water distribution system and the storm water drainage system, which further encompasses the 2D overland flow. Since DHI is cooperating with ESRI, the GIS-integration is ensued with components of ESRI ArcGIS. The basic modular structure of MIKE URBAN is shown in Figure 20:

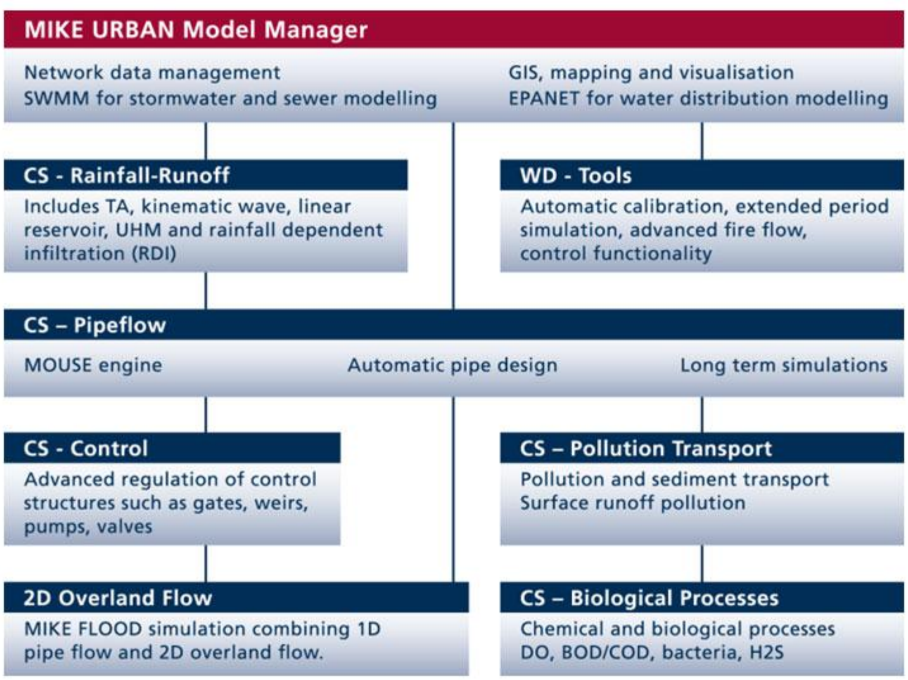


Fig. 20: Overview of the modular structure (DHI 2012c).

The foundation of MIKE URBAN forms the Model Manager, which is included with all license types of MIKE URBAN. The Model Manager is used for the input, the manipulation as well as the visualization of the model data. It is also used for illustrating and analysing the results of the simulations. As a part of the Model Manager, the SWMM5 engine allows a hydrodynamic computation of the sewer network. Built on top of the Model Manager, there are different add-on modules which can be licensed. In general it can be distinguished between the CS (Collection System) -modules for urban dewatering and WD (Water Distribution) –modules for water supply. For an urban flood simulation merely CS-modules are necessary, which are described as follows:

Like most CS-modules the “Rainfall-Runoff” uses the so called MOUSE engine from DHI. The CS-Rainfall-Runoff includes different hydrological methods such as the time area method, the kinematic wave or the Unit Hydrograph Method (UHM). This rainfall runoff module includes also additional flows as well as the RDI (rainfall dependent infiltration). These hydrological methods are given within a specified catchment area, whereby in MIKE URBAN a catchment is defined as a geographical feature which represents the hydrological urban catchments or wastewater drainage areas (AECOM n.d.:1). Finally, also the precipitation plays a role in this module.

The “CS-Pipeflow” module includes tools for hydrodynamic computation of the discharge. The user can manage the simulation of specific events as well as hydrodynamic long term series-simulations, which embrace statistical evaluations as well. This module includes also the automatic and iterative assessment of sewer sections (Telegdy 2011).

The “CS-Control” module serves for the regulation and modification of control structures, like pumps and weirs, from a sewer system. During a simulation it is possible to intervene this control structures. In the end the “2D Overland Flow” module is a major and important part for an urban flood simulation. This module enables MIKE URBAN to use a MIKE FLOOD simulation in the scope of the software. Thus, it is not necessary to use these software packages separately. Following the MIKE FLOOD software package and thereforee this module are described in more detail. (Telegdy 2011)

4.1.2. MIKE FLOOD

MIKE FLOOD is used for simulating flood plains, river flooding, storm surge and urban flood studies. This software includes three core elements:

- MIKE 11, which is a one-dimensional model especially designed for river flooding
- MIKE 21, which is used as a two-dimensional overland model
- MIKE URBAN model, which includes the urban drainage network.

With MIKE FLOOD it is possible to couple at least two of these elements respectively models together. The model linkage which has to be used depends on the study area. For the urban flood simulation, the coupling of a MIKE 21 model with a MIKE URBAN model might be the best choice. The integration of MIKE FLOOD into MIKE URBAN is only possible for this coupling method, and thereforee merely available for urban flood simulations. While this linking method is called “Urban Link” in MIKE FLOOD, it is called “2D Overland Flow” in MIKE URBAN. It is fully integrated into the GIS based GUI or MIKE URBAN. For instance the user can define different inputs like eddy viscosity, flooding and drying depth and bed resistance (For more details see 4.2.3.3). (DHI 2007:15ff)

4.2. Model Application

The subsequent part describes the model used, as well as the study area. Therefore all parameters respectively settings and attributes which are necessary for the simulation are described. Furthermore, some important (but not used in the model) parameters are described as well. For example, this paper describes the functions of weirs and pumps, although they are not given in the study area, but they could play an important role in some other urban flood simulations.

4.2.1. Model Description

A flood event can be caused by different events, starting from a heavy rainfall season up to a crevasse. In any case a number of factors, like topography, exchange with groundwater or land cover influence this flooding. By using software from DHI it is possible to include plenty of these factors to the GIS simulation. Especially an urban flood simulation requires complex modelling for an adequate result. Therefore a dynamic linkage of different models for the description of all important hydrological processes is necessary.

The foundation of model coupling in urban areas is the 2D digital elevation model. For the calculation of the water movement in the overland, detailed knowledge about the terrain character is relevant (Engels 2007:73). Thus the DEM should have a rather high resolution. MIKE URBAN uses the DEM to model physical processes, thereforee slope, stream barriers and other terrain characteristics are taken into account.

The coupling of 1-dimensional models, for flow modelling and pipe system modelling, with the 2-dimensional model takes place over a spatial allocation of the coupling spots to the raster cells. (Engels 76) The shafts (nodes) are linked dynamically to the corresponding raster cells, whereby all relevant hydraulic parameters are committed.

Meterological circumstances, like rainfall and air temperature can be applied as boundary conditions for diverse catchments and sub-catchments of the study area. Thereby MIKE URBAN can handle continuous data as well as time-series data. Hence it is for example possible to implicit the rainfall intensity for each minute for a specific catchment. By involving these inputs, for each catchment a hydrograph is calculated. For the hydraulic module the physical characteristics as well as the connectivity of the 1D model (pipes, manholes…) must be described. To calculate the 1D, gradually varied and unsteady flow, MIKE URBAN is using the Saint-Venant equations (for more information see chapter 3.4.3). (AECOM n.d.:1ff.)

The so called MOUSE engine of MIKE URBAN interacts with the 1D and 2D model as well as with the boundary conditions temporal and spatial dynamically. Based on the model categories outlined in chapter 2.3.2, the following model can be described as a, dynamic, deterministic and distributed model. Since a single storm event is simulated this model can described as an event model.

4.2.2. Study Area

For the practical part of this assignment it was tried to find a local respectively a regional appealing data set. However, since the models (especially the 1D- network model) require a lot of specific information, it was not possible to accomplish this task. Furthermore the contact person (Telegdy, Thomas) from DHI also inquired for a suitable data set. It was possible to get a dataset of Egebjerg/Denmark, however, it was only suitable for a network simulation and not for a combined 1D/2D simulation. Anyway, the final decision was to use the tutorial data set from DHI, which are specially designed for the 2D overland simulation. This data set includes the complete 1D network model with manholes, pipes and one outlet. All necessary parameters like the runoff method are provided. The DEM consists of a relative high resolution of 2 meter, but objects like streets and buildings are not included. Since a DSM offers in many cases a better result than a DTM, a small village with buildings and streets was created in ArcMap. The buildings were added to the DEM with a value of 10 meters while the streets get subtracted from the DEM with a factor of 20cm. Anyway effort was made to use a simulation with both DEMs to get a good comparison. The Figures 22 and 23 visualize the DEMs. Generally, the study area has a downslope from South-West to North-East with a coastline on the upper right. The altitude difference amounts from 0 (m) at the coast up to 70 (m) in the North.

Fig. 21: DEM without streets and buildings (author's design).

Fig. 22: DEM with streets and buildings (author's design).

In the 1D model all nodes are defined as manholes except one outlet at the coastline. The pipes have a circular shape and a diameter between 15 (cm) and over 1 (m). Figure 24 shows the manholes (blue), the outlet (yellow) and the pipes (orange) with the original DEM. As boundary condition only the rainfall in Figure 25 is included, which is given as a centenary event. Unfortunately, since the test data set is not georeferenced it is not possible to find out about the actual location of the study area.

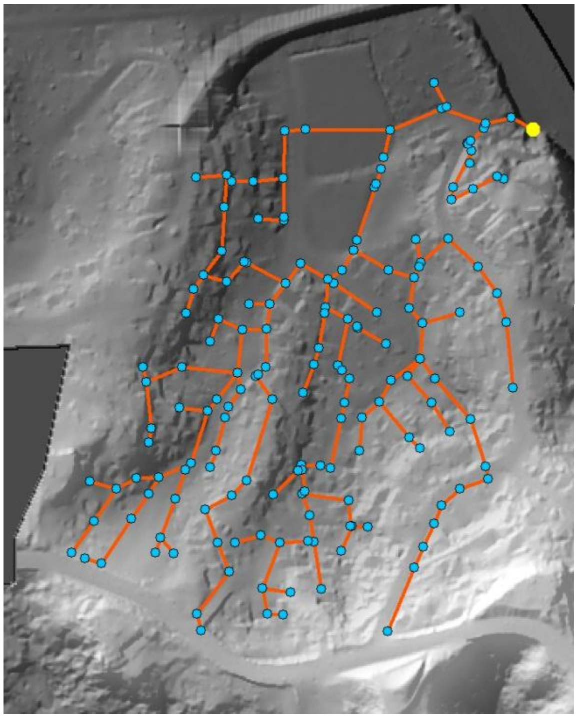


Fig. 23: 1D and 2D Model, without streets and buildings (author's design).

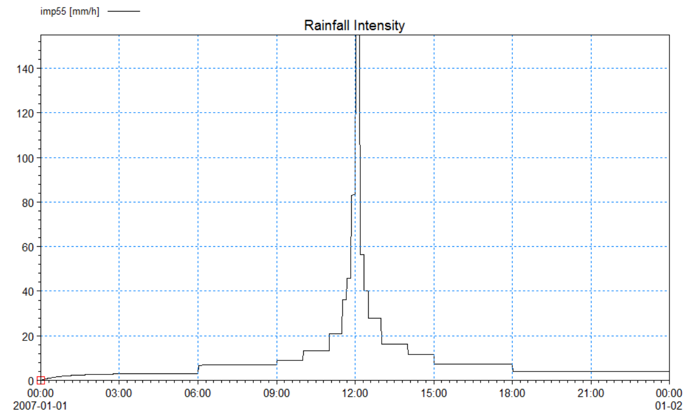


Fig. 24: Used rainfall intensity for the model (author's design).

4.2.3. Model Setup

When creating an urban flood simulation, it might be the best way to to start with the 1D network model. Therefore it is necessary to create and define the drainage system components, like manholes, basins, pumps and of course pipes. Furthermore it is necessary to define the catchment and each sub-catchment of the study area. If these components are not already given, they can be created easily and in a very similar way as the editor mode in ArcGIS. The second part of the model setup forms the 2D overland model. The major part of this model is probably the digital elevation model, which should have of course a very high resolution. Furthermore there are still some parameters which have to be defined. The following pages describe the major parameters and settings of both models which have a great impact to the urban flood simulation.

4.2.3.1. 1D Pipe Flow

As mentioned previously, the first step is to produce the drainage network, which could be done similar to the Editor Mode in ArcGIS. Anyway, after all network components are created, for each of them several settings have to be defined.

The following stage will describe the major parts of the pipe systems but with a special focus on nodes and structures as well as pipes and canals, since weirs, pumps and orifices are not used in our model or play just a minor part.

Nodes and Structures

Beside of some basic information like geometry (diameter), location (ground and bottom level) and of course the node type itself, which could be manhole (Figure 26), basin, outlet and so forth, there are some more parameters which must be given, especially for coupling with the 2D overland. According to this, the user has to define the “Max Flow”, the “Inlet Area” and the “Qdh factor” as well as the correct equation to setup the exchange flow between the 1D and 2D model.

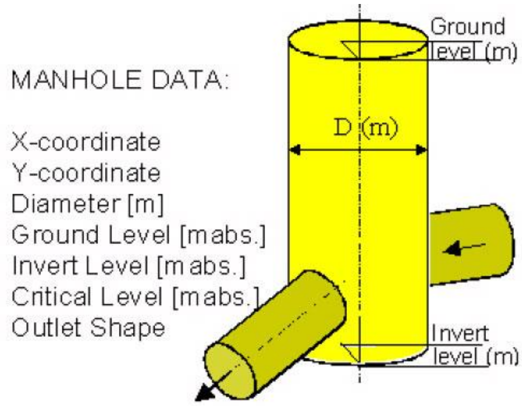


Fig. 25: Manhole in MIKE URBAN (DHI 2010c:10).

2D Overland Coupling Settings:

With the “Max Flow” the user can define an upper limit to the discharge, which is able to flow through the component. The discharge is reduced to this value, if the calculated discharge from the flow equation exceeds this value.

The “Inlet Area” plays merely a role, if the orifice equation is defined for describing the flow exchange between the 1D model elements and the 2D overland model. Anyway, the greater the cross section the greater the conveyance capacity of the coupling. The inlet area confirms physically to the area of the manhole cover. Acceding to this, the parameter is neglected when coupling to a basin. (DHI 2011:201)

The so called “Qdh factor” indicates the water level at which the (calculated) discharge should be oppressed. This “suppression is calculated as the water level difference divided by the Qhd factor to the power of 4” (DHI 2011:201). If instability is caused by a small difference in the water level in the two models, this factor can be used for stabilizing this situation. (DHI 2011:201)

Furthermore it is also necessary to define the exchange flow of the 2D Overland with the drainage network. Therefore MIKE URBAN offers three different methods/equations:

- Orifice equation
- Weir equation
- Exponential function.

The “Orifice equation” is more or less the standard used method. This equation is defined as follows (DHI 2007:74):

$$Q_{UM21} = sign(H_U - H_{M21})\, C Min(A_m, A_I)\sqrt{2g|H_U - H_{M21}|}$$

Formula 10: The orifice calculation (authors’ design based on DHI 2007:116).

for |QUM21|< Qmax , where:

QUM21 = the flow from the drainage network (sewer) to the overland (MIKE 21 grid point)

HU = the water level of the drainage system

HM21 = *the ground water level*
CD = *the discharge coefficient (this should be typically 1)*
Am = *the cross-sectional area of the manholes*
AI = *the cross-sectional area of the inlets.*

The corresponding water level gets overridden by the ground level if the nodes ground level is greater than the water level of the overland or the water level in the nodes. By using the orifice equation it is necessary to define a coefficient. This "orifice coefficient" is a non-dimensional factor which is used to scale the orifice flow.

The "Weir equation" is quite different to the orifice equation. This equation is depending on the "flooding situation". So, there are two different calculations for the weir equations, according if the surface is flooded or not. In the case that the overland is not flooded, the weir equation is calculated as a free flowing weir as follows (DHI 2007:117):

$$Q_{UM21} = (H_U - H_{M21}) W_{crest} \sqrt{2g|H_U - H_{M21}|}$$

Formula 11: Calculation as a free flowing weir (authors' design based on DHI 2007:117).

for |QUM21|< Qmax, where:
Wcrest = *the crest width which are used in the equation*
CD = *the discharge coefficient*

If the water level in the nodes is below the ground level of nodes, this equation is also used even the surface is flooded. Otherwise the following equation takes place to calculate the weir as a submerged weir: (DHI 2007:117):

$$Q_{UM21} = C(H_U - H_{M21}) W_{crest} \sqrt{2g|H_U - H_{M21}|} \left(\frac{max(H_{M21}, H_U) - H_g}{|H_U - H_{M21}|} \right)$$

Formula 12: Calculation as a submerged weir (authors' design based on DHI 2007:117).

The term Hg in these formulae is defined as the ground level at the coupling. (DHI 2007:117). The Wcrest "should be typically be the circumference of the manhole cover" (DHI 2011:201).

The third method to define the flow between the overland and the drainage network is the "Exponential function". This function is using the following formulae (DHI 2007:117):

$$Q_{UM21} = sign(H_U - H_{M21}) S (|max(H_{M21}, H_g) - max(H_U, H_g)|)^{\exp}$$

Formula 13: Exponential function (authors' design based on DHI 2007:117).

for |QUM21|< Qmax, where:
S = the scaling factor
Exp = the exponent
H is given in meters and Q in m^3/s.

The scaling factor has a linear effect to the discharge. Consequently, the higher this value the larger the flow for a certain water level difference between the overland and the sewer model. The exponent factor has a huge influence to the discharge. An increase of this value will affect a larger flow for a specific water level difference between the 1D and 2D model.

"There is an option to suppress oscillations by use of the QdH factor" (DHI 2007:117). The value correlates to the water level difference at which the suppression is referred. Especially in cases where the pressure level in drainage system is close to the water level in the overland model, this suppression is designed for. It may be applied individually at each urban coupling. By a given value of zero the suppression will not be applied. "The suppression is factor between zero and one, by which the discharge is multiplied." In the case that the water level difference between the nodes and the overland is greater than the before described Qdh then the factor will be set to one. Otherwise the factor varies as the water level difference to the power of four. (DHI 2007:117)

Settings for the study area:

For our study area all nodes got for the 2D overland coupling the same values, which are:

- Max flow: 0.10
- Inlet area: 0.16
- Qdh factor: 0.0
- Orifice equation with a coefficient of: 0.98

Q-H relation:

Irrespective of the simulation method (1D only or 1D/2D), the user has to define the Q-H relation for the nodes. By specifying a Q-H relation for manholes or even basins, this Q-H relation will control the infiltration to these nodes. By setting the Q-H relation the user can control the relation between the infiltration flow and the water level in the manholes. However, by defining of the Q-H relation for outlets the user can control the flow behaviour for an outlet. The flow, which is defined as Q, should be given as a positive value when water flows into the node and negative value when water gets lost from a drainage network. (DHI 2011:29)

Settings for the study area:

Q-H relation was not given for the manholes.

Pipes and Canals

Pipes and Canals in MIKE URBAN are defined as one-dimensional water conduits (links) between two nodes. As well as nodes also the pipes need some basic geometric and location information. An important parameter of links is the slope, which can be calculated automatically (distance and altitude difference between the two connected nodes) or typed in by the user. The latter can be important when the link bottom is not equal like the node bottom but rather overhead (Figure 27). However, the ending of a link cannot be below the node bottom.

Moreover the cross section as well as the material of the links is an important factor for the pipe flow runoff. For the link cross section MIKE URBAN offers four different pipe shapes:

- Circular
- Rectangular
- O-Shaped
- Egg-Shaped.

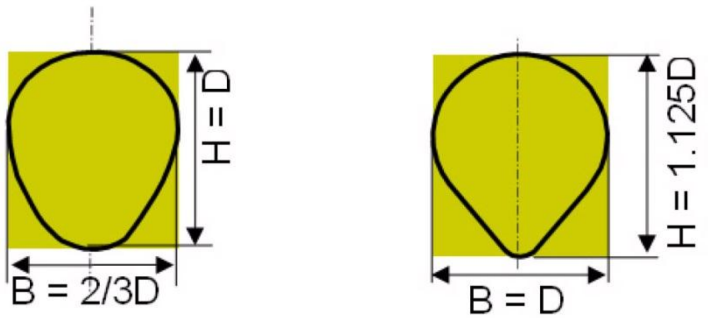


Fig. 26: Left: “Egg-Shaped” Pipe , Right: “O-Shaped” Pipe (DHI 2010b:16).

Furthermore it is possible to define own link types with the Cross-section Editor. This may be important when the study area includes special shaped links, like natural channels are often like to be. Generally it can be distinguished between opened and closed links. The user can therefore decide between a natural channel and a so called “CRS channel” (cross section shape). For natural channels the shape can be defined with the “Topography editor”. While a CRS channel is defined with just one shape, the natural channel topography is defined as a series of (many) CRSs. Figures 28 and 29 visualize a CRS channel, once as an opened link and once as closed link. This graphs show the geometry, the width, the conveyance, the hydraulic radius and the area of the CRS.

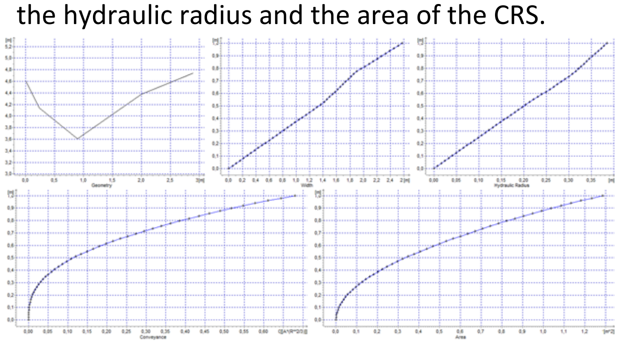


Fig. 27: Open CRS channel (screen-shot taken by the authors).

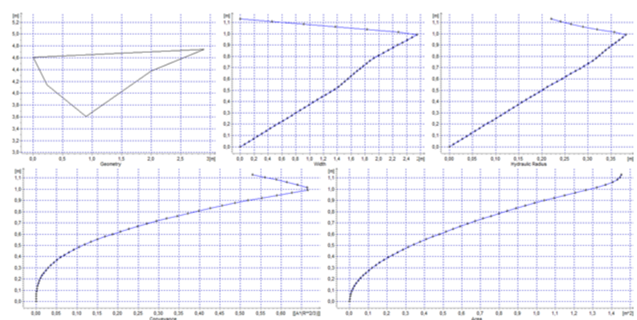


Fig. 28: Closed CRS channel (screen-shot taken by the authors).

As already mentioned, also the material of the links have an important role for the pipe flow. MIKE URBAN is using the Manning’s number to calculate the friction of the links. The user can choose one of the different pre-difined materials (compare to Table 2)
In the end the user can optionally define the maximum discharge for a link. This can be done with two different functions. First, as a function of the water level in one node or second, as a function of the water level difference between two nodes.

Settings for the study area:

All pipes have a circular shape, while the materials are concrete (normal). Therefore the values are:

- Manning: 75
- equivalent roughness: 0.0015
- H-W coefficient: 120
- Diameter: between 0.15 and 1.2 meter.

Weirs

DHI referred a weir “as a functional relation, which connects two nodes of a MOUSE network (two-directional flow and submerged flow possible), or is associated with only one node (free flow ‘out if system')” (DHI 2011:36).

While in reality a weir is normally located in a manhole or in a similar construction, in MIKE URBAN a weir is defined as a connection between two nodes. This occurs due to numerical solutions for the flow equations require this circumstance. For this model configuration the weirs will be placed between the nodes as flow connection. (DHI 2011:36)

A weir in MIKE URBAN owns different basic characteristics, such as crest level and width, orientation and the type. For latter there exists a number of different capabilities, like rectangular, V-notch, trapezoidal, long weirs or CRS weir. Depending on which weir type is given, some additional parameters have to be setup.

According to the selected weir type there are three different computation methods available, the standard Weir formula, the Q-H and the fragmented computation. Weirs in MIKE URBAN can be static or dynamic. For the latter one it is possible to control the weirs in “real time”. This can be done with the RTC (Real Time Control) tool in MIKE URBAN.

Finally it is also necessary to enable the “Coupling to 2D overland flow” checkbox. Anyway, there aren’t any further parameters for the 2-dimensional simulation. If the weirs are coupled to the overland model, the discharge of the weir will be added to the overland model as a point inflow which is distributed to the 2D cells in the coupled area. The water level in the 2D model will not influence the weir discharge, further the flow direction within the weir couplings is steadily from the 1D model to the 2D model. (DHI 2011:202)

Orifices

In the fashion of a weir an orifice in MIKE URBAN is likewise defined as “a functional relation”. Moreover it is as well necessary to define the orifice as connection between two nodes. It is possible to define more than one orifice between the same nodes. Figure 30 visualize the computation grid for the orifices, furthermore this is also adaptable for weirs, pumps and valves. (DHI 2011:40)

Orifices are specified by the network type, which can be storm water, waste water or a combination of both, the model type (rectangular, circular or CRS) and resultant the diameter as well as the height and width. Additionally it is necessary to define the discharge coefficient and the so called flap gate. By activating flap gates the user can control if a water backflow is possible or not. As well as weirs, also the can be controlled with the RTC tool dynamically in real time.

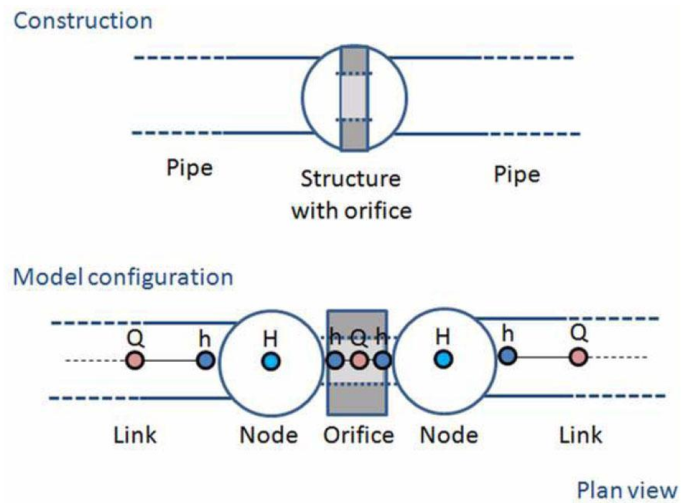


Fig. 29: Difference between real world and model configuration of orifice (DHI 2011:40).

Pumps

Besides weirs and orifices also pumps are defined as functional relation. In the case that the pump is connected to just one node it is possible to let the water pumped out of the pipe system to the 2D overland model. MIKE URBAN offers different types of pumps, namely “Constant flow”, “Constant speed” and “Variable speed”. The constant flow pump is probably the “easiest” type. The discharge will be always the same for these pumps insofar the pump is switched on and acceleration or deceleration period is not given. The so called constant speed pumps have a discharge which will be a “function of the actual pressure difference (dH) between the pump wet well and the receiving point in the model.” (DHI 2011:50)

Figure 31 shows an example of the relationship of discharge and pressure difference. The user can define such a pump curve in MIKE URBAN with the “Curves & Relations” tool. During the simulation the pump operates within the range of the dH values in the table. So, at each calculated time step the actual dH value gets queried and the corresponding discharge will be determined. Much more complex appears the variable speed pumps. For this type of pumps it is necessary to define a number of pump curves, which describes the pump capacity at various percentages of the maximum power input respectively the maximum rotation speed. Figure 32 visualizes an example of these pump curves. (DHI 2011:52)

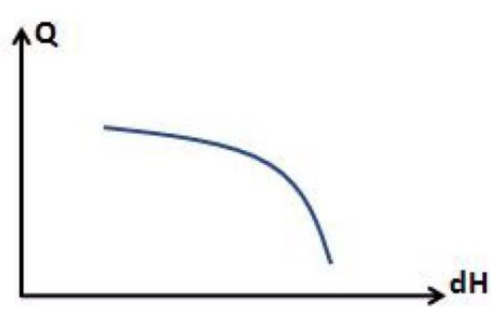


Fig. 30: Relationship of discharge and pressure difference (DHI 2011:51).

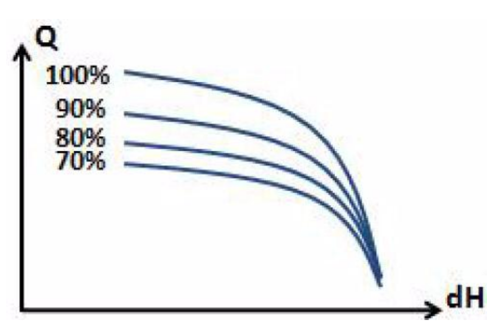


Fig. 31: Variable speed pump (DHI 2011:51).

Contrary to the constant pumps it is required to define besides of the maximum capacity (defined by rotation per minute) also the minimum capacity. For coupling the pumps with the 2D model the checkbox have to be activated. Anyway, pumps are coupled in the same way to the overland model like weirs.

4.2.3.2. Catchment

Catchments are essential for every hydrological model. Catchments in MIKE URBAN are handled "as hydrological units where storm runoff and infiltration (RDI) are generated on basis of a single set of model parameters and input data. In fact, MIKE URBAN catchments represent the level of spatial discretization of the hydrological model." (DHI 2011:68)
However, the geographical boundaries of a catchment do not always suit with the concrete drainage area. Therefore the user can specify a drainage area, which would overwrite the actual catchment area in all hydrological computations. (DHI 2011:68f)
A catchment in MIKE URBAN can be created as polygon in a very similar way as in ArcMap with the Editor Mode. Once a catchment is "drawn" they need to get connected to the network model so the computed runoff gets into the collection network. Thereby it is possible to connect more than one catchment to one node of the 1D model.

4.2.3.3. 2D Overland

DEM and 2D model area

The first step for setting up a 2D overland flow model is choosing the 2D model type itself. MIKE URBAN offers for this model selection three different types.

Single Grid:

a) "Single Grid by using a rectangular cell solver", whereby the 2D model area will be covered by square calculation cell
b) "Single Grid by using a rectangular multi-cell solver" with the same coverage as type A. However, this type is related to a coarser grid where each grid cell is moreover "subdivided into N by N cells called the fine grid." (DHI 2011:192)

Flexible Mesh solver

c) By using the "flexible mesh" solver the 2D model will be covered by a grid which exists of a mix of triangular and quadrangular cells. However, this opportunity is only available in MIKE FLOOD, in MIKE URBAN this type is currently not supported.

2D model parameters

Next up, a digital elevation model can be applied, whereby the data format has to be an ESRI Grid format or DHI intern dfs2 format. Latter format can be used just in DHI and ESRI products on computers with a DHI license. Furthermore some undisclosed formats, depending on installed GIS products, could be applicable.
Once the DEM is implemented the ground elevation band can be defined, furthermore it is possible to fill missing values within a specified radius. Moreover the cell size as well as the extent of the model area can be defined.

Afterwards some model parameters have to get setup, such as the “Land value”, the “flooding and drying depth, bed resistance, eddy viscosity and rainfall intensity.

Land value

The land value can be applied either to define areas where no flooding is expected or to setup a boundary where flow is not allowed to across. (DHI 2011:195) Therefore either a user specified value is added to the highest value of the DEM or a specific elevation hight gets defined.

Flooding and drying depth

These two parameters are used to declare which cells are to exclude or reintroduce into the computation of a simulation. DHI declares that the flooding and drying depth is an important mechanism for 2D overland flow simulations, since the runtime of the simulation can be reduced by not spending computational efforts on simulating cells which are dry. Naturally the drying depth must be smaller than the flooding depth.

Bed resistance

Concerning to the Manning number (m1/3/s) the bed resistance coefficient can be specified as constant value for the whole 2D Model or it can be specified by using an external raster layer.

Eddy viscosity.

A bit more complex arranged the eddy viscosity. By including eddy viscosity the simulation takes eddies into account. The choice of the formula influence how eddies are simulated as a result of flux gradients (or spatial velocity). The user can choose between the flux based default value (which is defined as $E = 0.02*\Delta x*\Delta y/\Delta t$, where Δx and Δy are the cell size of the model and Δt is the time step of the simulation), or by defining an eddy value and an eddy formulation.

Rain DEM

As a last resort the rainfall can be added as an external file to the DEM. However, the rainfall intensity can be also included later on as boundary condition.

Settings for the study area:

- Selection of 2D model: MIKE 21 Single grid using rectangular cell solver
- Land value: Highest DEM value + 10 meter
- Drying depth: 0.002
- Flooding depth: 0.003
- Bed resistance: Constant Manning number à 32
- Eddy Viscosity: Included with default value

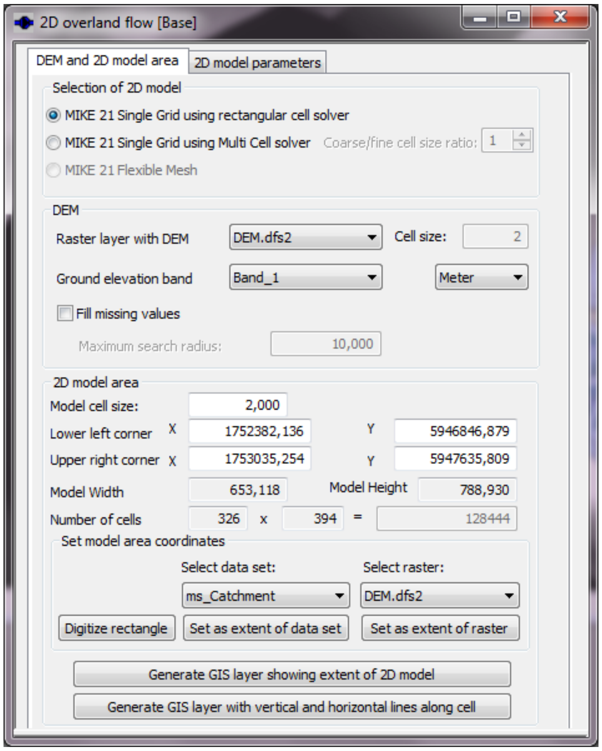


Fig. 32: Settings in MIKE URBAN for the DEM and the 2Dmodel area (screenshot taken by the authors).

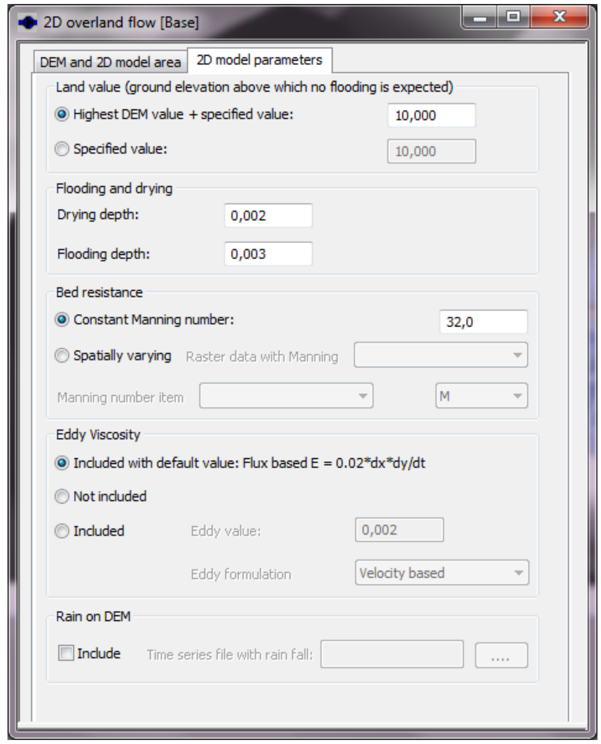


Fig. 33: 2D model parameters in MIKE URBAN (screenshot taken by the authors).

4.2.3.4. 2D Overland Coupling

After the 1D model and the 2D model are finished it is possible to couple them together. Therefore following steps are necessary

- selection of the coupling pipe flow components
- election of the specific calculation cells
- selection of the flow formula for basins and manholes (only necessary if not already done before).

At first of all it is necessary to couple the pipe flow components with the overland model. Usually these 1D model components are manholes, however it is also possible to couple with weirs, pumps outlets and basins. The coupling of 1D component with specific calculations cells of the 2D model are selected automatically, based on the users defined number of cells or search radius (Figure 35). By defining a specific number of height/width cells a squared area of 2D cells which are nearest to the node getting selected. By defining a radius all cells within the radius (given in meters and starting from each node) will be included in the coupling area. Figure 36 shows coupled nodes with different settings.

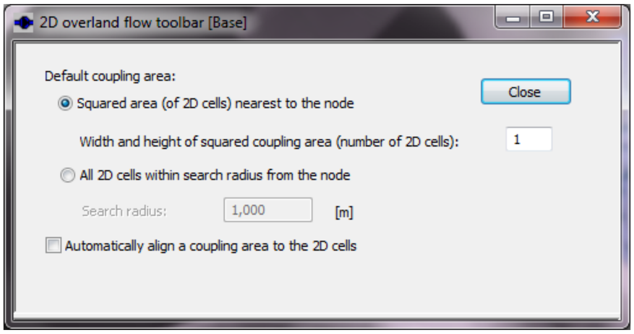


Fig. 34: Coupling settings in MIKE URBAN (screen-shot taken by the authors).

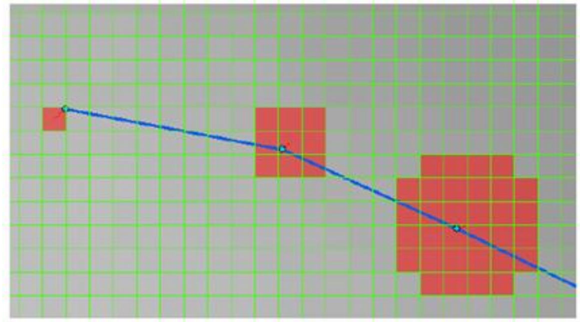


Fig. 35: Various coupled Nodes to 2D cells (DHI 2011:198).

Settings for the study area:

- Coupling Area: Squared area nearest to the node
- Number of cells: 1

4.2.3.5. Boundary Conditions

Under "Boundary Conditions" DHI understand the rainfall intensity and other meteorological circumstances such as air temperature and evapo-transpiration. The rainfall intensity is probably the major and most important part of setting up boundary conditions. In MIKE URBAN the rainfall intensity is usually given as time-series. Therefore the user has to create first of all a "Time-Series" file which is stored as a DHI intern "DFS0" format. This can be done with the software tool "TSEditor" which can be started either within of MIKE URBAN or external. In any case it is possible to define the rainfall intensity very much in detail (Figure 37). The rainfall, as well as other boundary conditions, can be applied to the whole study area or just for specific catchments. For the first case coordinates can optionally be specified.

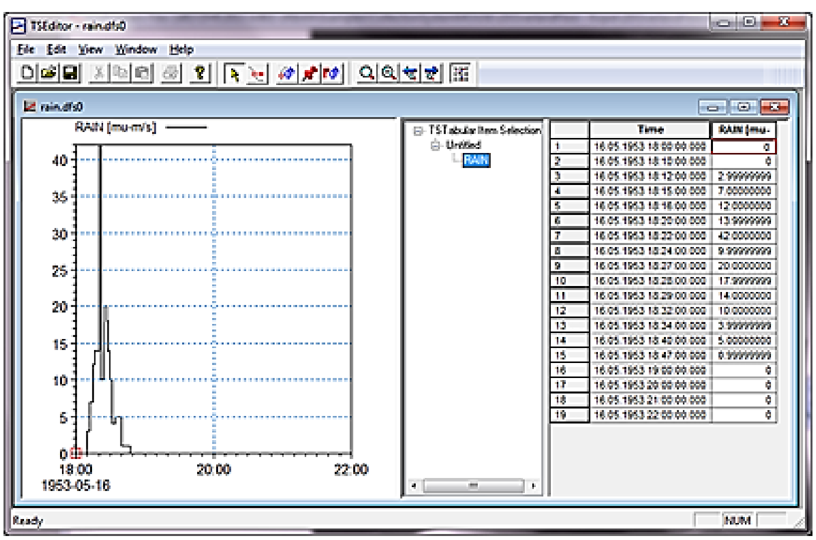


Fig. 36: TSEditor for the rainfall intensity (screen-shot taken by the authors).

4.2.3.6. Set up and Running the Simulation

Setting up the simulation takes only few steps into account. Howsoever, it is necessary to define the simulation type as "Network + 2D overland". Furthermore the simulation period, some outputs (like the flood map) and the computation time steps have to get defined.

The outputs usually are stored as DHI intern formats like DFS2 for the flood maps, DFS0 for time-series and CRF for the network model. Latter includes the discharge of the 1D model and is necessary for hydrodynamic calculations.

A major specification of the simulation is the hydrological model. As already mention in chapter 3.3, there exists different concepts for simulating a surface runoff, DHI offers thereforee four different models: (DHI 2010c:9)

- Model A: Time/area (with or without RDI)
- Model B: Non-linear Reservoir (kinematic wave) (with or without RDI)
- Model C: Linear Reservoir
    - Model C1: Dutch runoff model
    - Model C2: French runoff model
- UHM: Unit Hydrograph Model (with or without RDI).

Each of these models uses different runoff computations concepts and model parameters, as well as a different set of input data. It is not possible to combine the computations within one simulation for various model areas. For the Models A, B and C it is possible to include continuous runoff components, like the rainfall-induced infiltration. Howsoever, some implementations, as well as the rainfall-induced infiltration, require additional DHI licenses.

The user has to decide which of the models fits best to his application respectively to the catchments. Thereforee it is indispensable that the user understand the background of each model. The appreciation of the used runoff computations which is applied in the model "as well as the meaning and the interrelation of the various data and model parameters" is necessary. (DHI 2010c:9)

For this study the Unit Hydrograph Model was used for simulating the rainfall runoff.

Settings for the study area:

- Runoff Model Type: UHM
- Time step: 0,5

4.2.4. Result Interpretation

Following an overview of the simulation results for the 1D network model as well as for the 2D overland model are given. The interpretation of the 1D model is necessary to understand in a better way the overland flooding, since the pipe network interacts with the overland via manholes and outlets. Figure 38 visualises a pipe section of the study area at a specific time (11:59 am). It is readily identifiable that the pipes in the lower section are overloaded and how the manholes get overfilled and discharge the water to the overland.

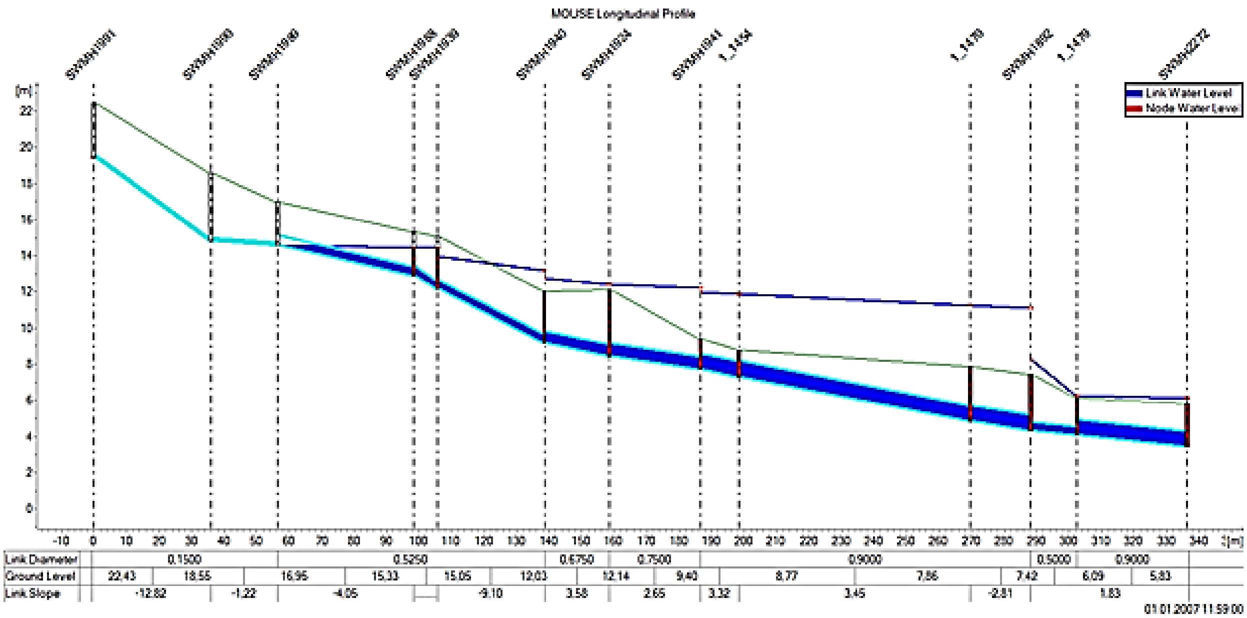


Fig. 37: Specific pipe section of the study area at 11:59 AM (author's design).

By a reflection of the overland at the same time it is discernible how the overland is flooded equally in the lower section.
With a look to rainfall intensity (see figure 37), which has the peak around 12 am, it is identifiable that it took quite a long time until the overland situation gets more or less relaxed. While after the heavy rainfall the flooding goes relative quickly back in the higher parts of the study area, in the lower parts the flooding is still increasing. The maximum flood extent is around 1 pm.
The comparison of the simulations, once with the DTM and once with the DSM, confirmed the importance of objects in a digital elevation model. Especially by interpreting the simulation as video it is clearly recognizable how the streets officiate as channels (see clip 1 and 2). Furthermore areas with buildings cannot be flooded anymore consequently the water has to find another way. Map 1 visualizes the maximum flood extent during the calculated time period.

Rendering Time:

- 16,2 minutes

Computer specification:

- Win 7 Home Edition
- 8 processors
- 12GB RAM
- Intel i72600 - 3,4 Ghz
- 1GB external graphic card

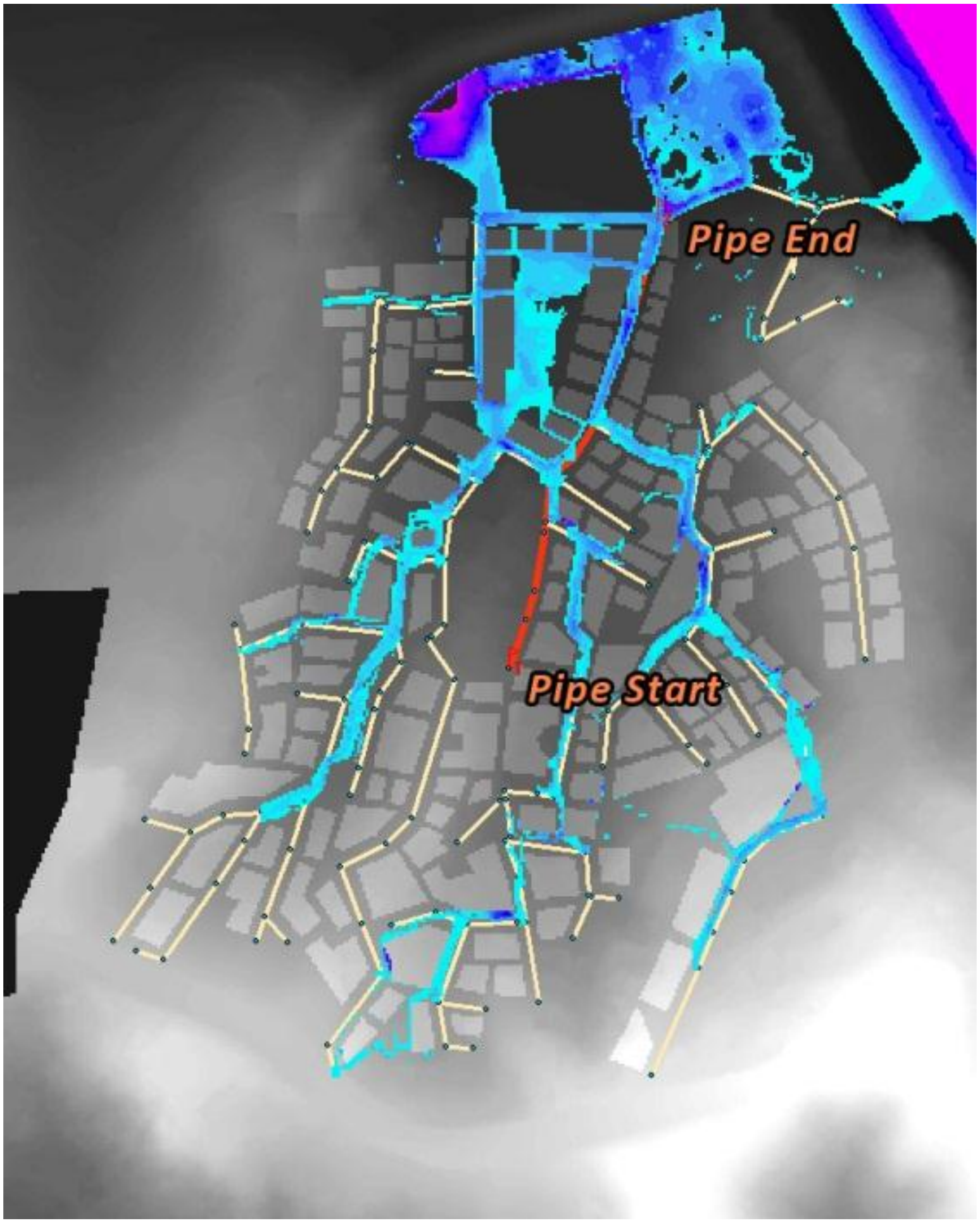


Fig. 38: Study Area with the flood extent at 11:59 AM (author's design).

Clip 1: Flooding scene with buildings and streets (author's design).

Clip 2: Flooding scene without buildings and streets (author's design).

4.3. Usage of the Model Outcome

There are several possibilities how the outputs of the simulation can be used. One common way is the creation of a flood risk assessment map. For this purpose the maximum flood extent can be exported directly as a map in diverse formats. Beside of others it is possible to export the map respectively the files as a GeoTIFF, whereby further adaption with other software products is possible.

Once for a study area all models and parameters are given it is feasible to create rapid flood assessments. Thereby it is only necessary to define the rainfall intensity for a given time period. The coupling with a precipitation radar brings the possibility to evaluate controller options of pipe system and react correspondingly.

Finally flood contingency planning, like strategy of evacuation routes and priorities, can be done with the outputs of the simulation. (DHI 2012a)

4.4. Coupling with Other Software

MIKE URBAN can handle data formats from different software producers. Especially the import of network data is supported, while the export of results and models is provided only in minimal way. Following data formats can be amongst others imported for network data: (Telegdy 2012)

- AutoCAD (.dwg, .dfx)
- ESRI (Shapefiles, GeoDataBase)
- Excel-Spreadsheet
- Text-Files (.txt, .csv, .tab)
- XML
- MapInfo

Since MIKE URBAN store data usually in File-Based-Geodatabase respectively in Personal-Geodatabase, software products like ArcMap can access to these data. However, this is only possible if MIKE URBAN is installed on the PC, otherwise the freely available extension “MIKE Urban Class Extension” is necessary. The DHI proprietary data format “.dfs2” for raster files can be opened end edited with diverse software products from ESRI. However, it is necessary to have a MIKE URBAN license on the computer otherwise the ESRI software is not able to open this format.

4.5. Discussion

For both the 1D and the 2D model it is possible to set up an environment that takes account for a lot of detailed information. Also, there is the opportunity to define methods and formulae, for example in regard to rainfall runoff, that build up on pre-defined parameters and can be changed, according to preferable adjustments. However, the wide range of parameters also comes along with some drawbacks, regarding the complexity of the models. Thus, it is necessary that the model user possesses a well-founded knowledge about both the software and the hydrological/hydraulic processes that take place in the background. Especially for non-hydrologist engineers some parameters, methods and formulae are rather difficult to understand and to deal with. Moreover, it takes quite some time to get to know to how the feature-rich software needs to be handled. Thus, especially the installation and first use of the software tended to be frustrating. It took a number of computers and installations until the software finally run and a model was set up. The documentation in form of manuals and tutorials was quite helpful and comprehensive in this respect. Besides, a contact person, responsible for MIKE URBAN (Mr T. Telegdy) was a great help.

The reliability of the final simulation results depend more on the qualifications, experience and care of the analyst as well as the disposability of suitable data than on the software respectively the model itself (Russ 1999 cited in Butler and Davies 2011:494). Furthermore also the available data and the model complexity itself play a major role in the simulation. As Butler and Davies (2011) visualise in Figure 39, the coherence of the data availability and the model complexity influence the accuracy of the prediction in an essential way. For example low model complexity with high data availability might result into low prediction accuracy. On the other hand also a high model complexity combined with high data availability is not the optional solution, since the possibility that the available data might include some deficiencies. Therefore a complex model with a minimum of data might results in the most accurate prediction. Anyway, for a specific level of model complexity there is an optimum level of data availability (for example the dotted line for high model complexity and the dashed line for a simplified model), as well as the other way around (solid line). This means that the increasing of model complexity respectively data availability leads to improve the predictive performance, but only up to a certain level. (Buttler and Davies 2011:493)

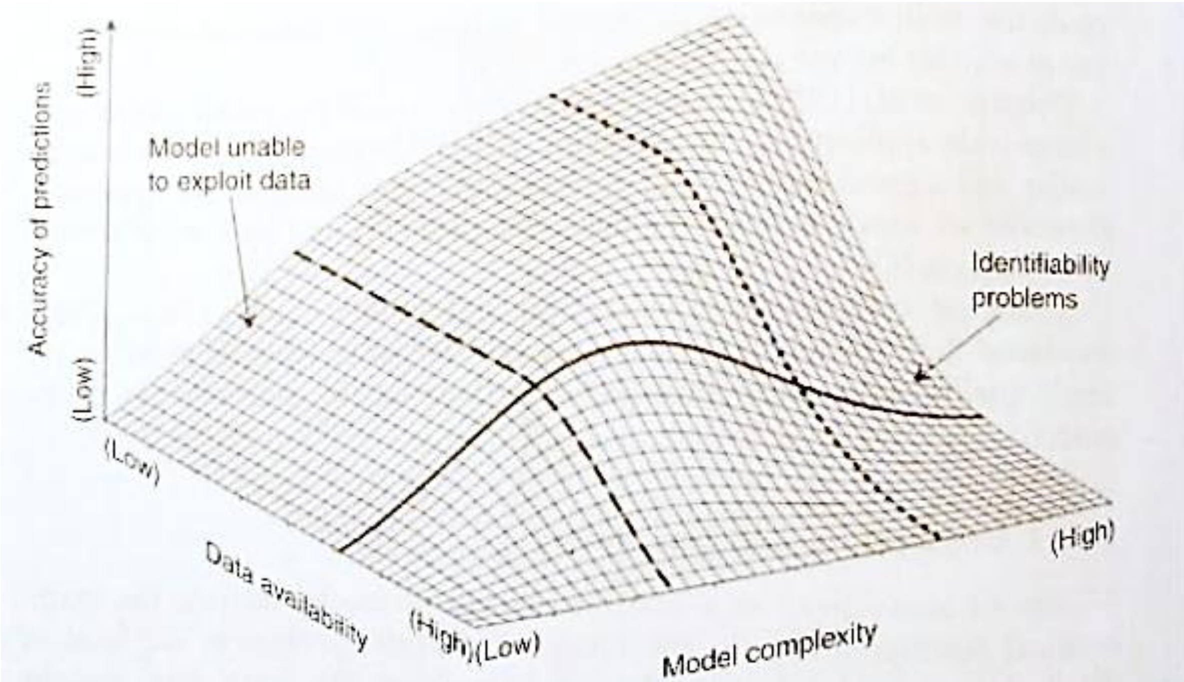
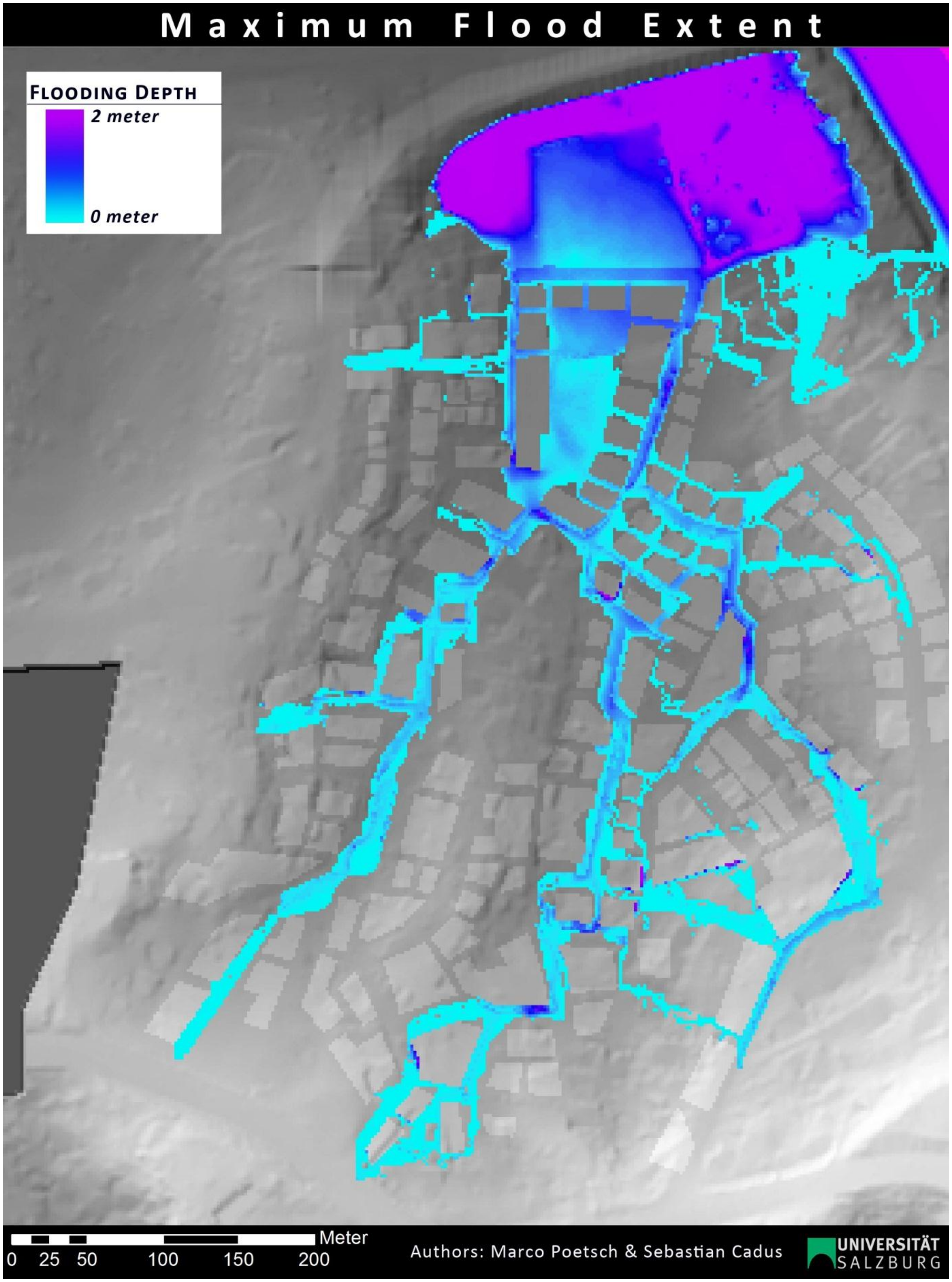


Fig. 39: Coherence of data availability and model complexity (Butler and Davies 2011:496).

To sum up, the usage of MIKE URBAN and MIKE FLOOD is for geoinformatic students only suitable to a up to a certain extent since the complexity of the software and the necessary appreciation of hydrological and hydraulic givens are rather demanding. Nevertheless the software is for flood modelling and especially for an urban pipe flow evaluation very useful and commendable. With some practical work in MIKE URBAN and a good comprehension of the concepts and methods the software from DHI can also be used by geoinformatics engineers.



Map 1: Maximum flood extent by using the DSM (authors' design).

5. CONCLUSION

This paper discussed the theory and the different methods of urban flood modelling. Furthermore also the concept basics of rainfall runoff and urban drainage have been outlined. For the practical part of this work a method coupling a 1D “MIKE-MOUSE” model and 2D “MIKE-21” model was used to simulate an urban storm water event.

The method of combining hydraulic and hydrological mechanism is useful for water management and overland flow analysis in urban areas. Moreover with the outcome of the simulation flood risk areas can be identified respectively defined. Thus strategies for minimizing repercussions of heavy rain fall events can be elaborated by experts. Since it is possible to include real-time data like weather radar information and to do some real-time modifications (for example opening and closing pumps) during the simulation, the method can also be used as an early-warning system and give the agencies the opportunity to intervene timely into the urban sewer system. The complexity of a dynamic urban flood simulation by coupling a 1D with a 2D model is great. For both models it is inevitable that the analyst understands a range of parameters and necessary settings, otherwise the simulation might result in error prone outcomes.

With the software from DHI the user has the great possibility to do a dynamic 1D-2D simulation. Once the structure of the models is given it is possible to run the simulation very quickly. Also changes can be applied in an easy and fast manner. However, it is still necessary that the analyst understand the essential factors, as well as the meaning of the formulae and the parameters which must be defined. The link to ESRIs ArcGIS is quite good while the connectivity to other GIS software could be improved.

To sum up, both MIKE URABN and MIKE FLOOD seem to be very sophisticated and holistic tools. This brings advantages on the one hand but also drawbacks on the other. In order to make use of all provided product features and to gain sensible results from the application, a high degree of experience and background knowledge are necessary. However, taking into account the outcomes that the software provides, a solid base can be derived to be used for further investigations.

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