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Coupling Process for 1D-2D Numerical Flash Flood Simulation: A Parameter Study of Involved Variables for Gullies and Manholes

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ABSTRACT

Urban flash floods and their hydronumerical coupled modelling are influenced by various parameters and assumptions for model setup and implementation. Hence, the present paper deals with coupling details of 1D-sewer and 2D-surface models. Considered hydraulic parameters will be analyzed concerning their impact on computed results for flood levels and the discharge rate (bi-directional) between both, 1D and 2D, model approaches. Additionally, flood durations will be investigated. Considered parameters are the inlet area, the limitation of the discharge capacity according to standards of the legislation and the discharge coefficient, which has only minor impacts on the discharge rates in this configuration. Comparisons of limited and unlimited numerical computation for discharge capacity at the coupled nodes show that the flood duration will be influenced more than flood levels. The quantitative exchange at each node is calculated by applying the Torricelli approach and by including variable parameters. Analyzing flash floods with coupled numerical models allows the implementation of measures and their evaluation regarding flooding depth and thus provides security against flooding. Exemplary improvements will be shown. Additionally, the model is primarily evaluated by comparing its results with measurements in the sewer system.

Keywords: *gullies, flash flood, coupled numerical modeling, parameter study, discharge coefficient, inlet capacity.*

1. INTRODUCTION

The present investigation is part of the “RainAhead” project, which is funded by the Federal Ministry for the Environment, Nature Conservation, Building and Nuclear Safety (BMUB). The main aim of the project is to develop an integrated planning and warning tool for heavy rain events and their resulting flash flood in urban areas. Heavy rain events and possible resulting flash floods occur locally or regionally as a result of convective precipitation of high intensity (e.g. Maniak 2010). These events can mostly be observed during the summer period (Hatzfeld et al. 2008). The analysis of an event data-base regarding flash floods (within the project URBAS) shows that flash floods arise with an occurrence of 72 % between March and September (Oertel 2012). Intensities of heavy rains can vary considerably regarding their precipitation quantity as well as their duration. Consequently, the German Meteorological Service defines three warning intensities (GDV 2015). DIN-4049-3 (1994) also describes heavy rain events by comparing their duration and intensity. IPCC (2014) expects an increase of intensive rain events and with it an increase of urban flash floods. The number of extreme events has increased significantly since 1980 according to MunichRE (2015). Additionally, with an increase of surface sealing, negative effects can be assumed regarding heavy rainfalls. Infiltration (g), evapo-transpiration (v) and runoff will change the water cycle in urbanized regions compared to natural conditions (Figure 1).

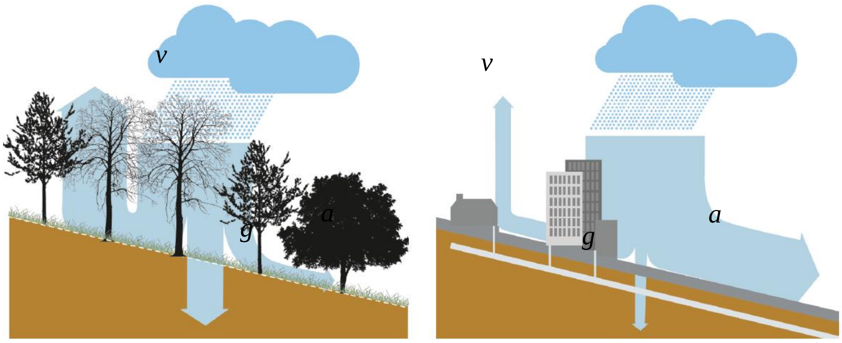

Figure 1. Increase of runoff (*a*) due to surface sealing and decrease of infiltration (*g*) and evapo-transpiration (*v*) in urban areas (Kruse et al. 2014)

Urban drainage systems are designed for defined water amounts and rainfall return periods within a particular catchment area. Hatzfeld et al. (2008) mentioned a variation of design boundary conditions for various catchment areas. Generally, the design return period for urban drainage facilities is defined in DWA-A 118 (2006). With residential areas, as an example, the typical design frequency (or return period) is n = 0.2 to 0.5 (or T = 5a to 2a respectively). DIN EN 752 requests a proof of the return period for the investigated catchment area depending on the usage (DIN EN 752 2008).

Numerical flash flood analyses usually use 2D surface runoff simulations to describe resulting flood situations. Other methods are based on Geographic Information Systems (GIS) with integrated flow-path analyses (e.g. Koch et al. 2015 and Chen et al. 2009) or coupled 1D-1D models (Maksimovic et al. 2009). Additionally, experimental models are used to describe detail processes like inlet capacities of gullies and to compare these with in-situ measurements (Kemper et al. 2015) as well as with 3D numerical models only for gullies or other details of the drainage system (Djordjevic et al. 2013). Results can also be used for validating and calibrating numerical models. The present paper focuses on coupling processes of drainage systems with surface runoff by means of analyzing the coupling process of independent numerical hydrodynamic models. Thereby, the parameters considered within applied analytical solutions are in the main focus of interest. In contrast to other research projects and national handling rules for urban flash floods, a coupled hydrodynamic numerical 1D-2D model allows a more detailed analysis of occurring flow paths and pipe capacity utilization. It represents an expensive method regarding time consumption (Ghostine et al. 2015), data input, data accuracy, simulation duration and expert knowledge (see Henonin et al. 2013 and DWA-T1 2013). Nevertheless, coupled 1D-2D models are often used in urban areas (Hunter at al. 2008; Vojinovic and Tutulic 2009; Leandro et al. 2009). Additionally, measures to avoid flash flood damage (e.g. retention areas or barrier removal, see e.g. Hoppe et al. 2011) can be adopted and investigated accurately. Forecasting of storm events can also be used to manage flash floods (Einfalt et al. 2009).

2. INVESTIGATOIN AREA AND HYDRODYNAMIC NUMERICAL MODEL

The investigation area is located in the northern part of Germany within the city of Lübeck. The district St. Lorenz Süd is located close to the city center at the river Trave (Figure 2). The catchment area is A = 2.4 km². Within the current investigation the commercial numerical models DHI MIKE21 (2D) and MOUSE (1D) are used. The 1D model allows a time-dependent simulation of the sewage system. The 2D model is a surface runoff model with a mesh resolution of 1 m in *x*- and *y*-direction and 5.1 Mio. cells in total (Table 1). Both models are coupled via connecting nodes. These nodes represent gullies and manholes in the sewage system, allowing a bi-directional mass transfer. The governing equations are the Saint-Venant-Equations complying with the principles of the conservation of mass and momentum. A detailed description can be found in DHI (2015). Details of the coupling process, as implemented here, are shown in Figure 3.

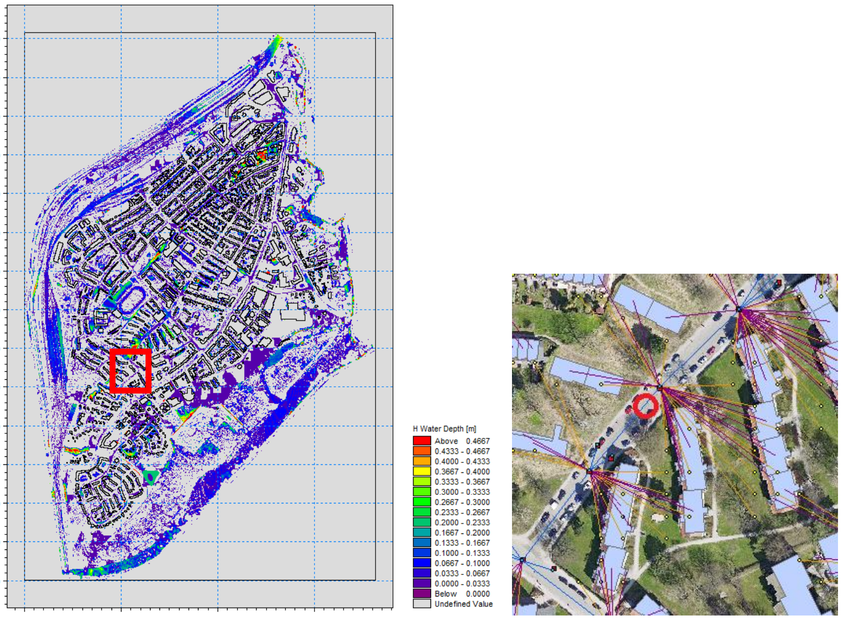

Figure 2. Overview of the investigation area (St. Lorenz Süd), left: exemplary flooding results for a once in 100 years rain event, right: exemplary investigation point within the catchment area

Table 1. List of control points within the catchment area, coordinates and nearest manhole ID in Mike Urban

Point no.	Location	Type	*x* [m]	*y* [m]	Manhole [MUID]
1	Stettinerstr.	low point	281	602	7480001
2	Hansestr.	elevated point	392	1004	7170014
3	Moislinger Allee	elevated point	501	501	7370309
4	Lachswehr Allee	close to low point	729	729	7280101
5	Lindenstr.	low point	613	1145	7310022

Table 2. Types and number of the 1D sewer model MOUSE of nodes, links and catchments, and 2D overland model

Type	*Amount / length*
Catchments (roofs)	4216
Nodes (total)	2572
Manholes	1031
Gullies	1513
Basin	1
Inlets	3
Outlets	24
Coupled nodes	2549
Links (pipes)	2580
total length of links	73.9 km
Weirs	6
Area 2D	2.4 km²
Number of cells 2D (1m)	5.1 Mio
Number of cells 2D (2m)	2.8 Mio

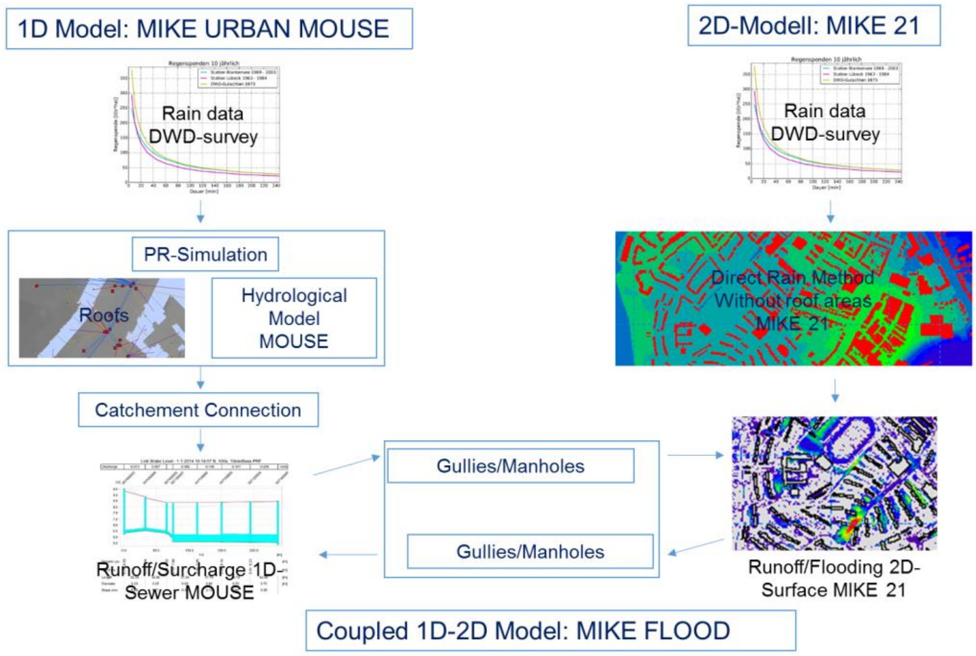

Figure 3. Schematic overview of coupling process for 1D and 2D numerical model (according to Kühnel 2015)

The input data is rainfall data provided by the German Meteorological Service (DWD), which is also used by local authorities to calculate drainage facilities. A precipitation runoff model computes the runoff from the house's roof areas; these areas are directly connected to the 1D sewage model. The same precipitation data set is used for the 2D surface runoff computation. Both models are connected via nodes so that an exchange of water can be calculated by an implemented analytical approach. Details of the coupling process are shown in Figure 4. Table 2 gives additional information about the models used in the present investigation.

Other connective configurations of the catchment areas with the 1D-sewer model are described and applied during different investigations. Runoff computation without a separate precipitation runoff model can be applied. The rainfall boundary is thus applied to the bathymetry (2D) only. Therefore, the roughness values for the buildings should be lowered allowing the flow time towards the coupled nodes to be shortened (Babister and Barton 2012). The direct connection of runoff into the sewer system is not possible in this way.

Figure 4 illustrates two varying flow directions for the coupled nodes: (1) surcharge and (2) inflow. The mathematical description is implemented in the numerical model via the analytical Torricelli approach (DHI 2015):

$$Q = A \cdot C_D \cdot \sqrt{2 \cdot g \cdot h} \tag{1}$$

where Q is the total gully discharge, A is the inlet area, C_D is the discharge coefficient, g is the acceleration due to gravity and h is the flow depth above the coupled node. The principle is the exchange of energy between the potential energy at the surface and the kinetic energy at the opening. Q is also defined as the inlet capacity, which can be limited to a maximum allowed value within the numerical model according to standards like Ras-Ew (see e.g. Kemper et al. 2015) out of FGSV (2005).

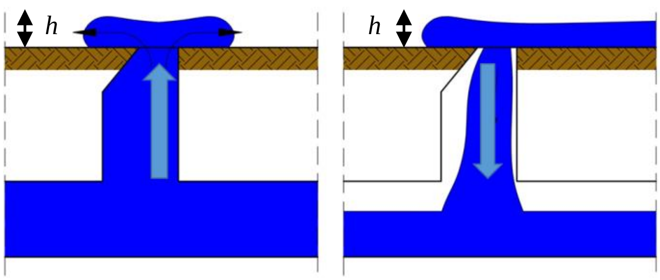

Figure 4. Schematic plot of coupled nodes (manholes and gullies) with bi-directional exchange of water between 1D-2D model, left (1): surcharge, right (2): inflow from 2D overland (DHI, 2015)

Default values for maximum gully discharges and their discharge coefficients within the numerical software are Q = 0.1 m^3/s and C_D = 0.98. The gully's flow area A is computed with geometric specification of each node (the same is valid for manholes), g is constant and the flow or flooding depth h is computed during the simulation process because of the time-dependent flow development.

Within the present paper above mentioned parameters will be studied concerning their influence on resulting sewage and surface discharges. Therefore, particular DIN regulations or maximum inlet capacities for gullies and manholes are used. As an example, Figure 5 shows two established gully dimensions in Germany especially chosen, as they are located in the analyzed catchment area. Within the investigated numerical model, various pre-defined inlet areas will be set as a constant.

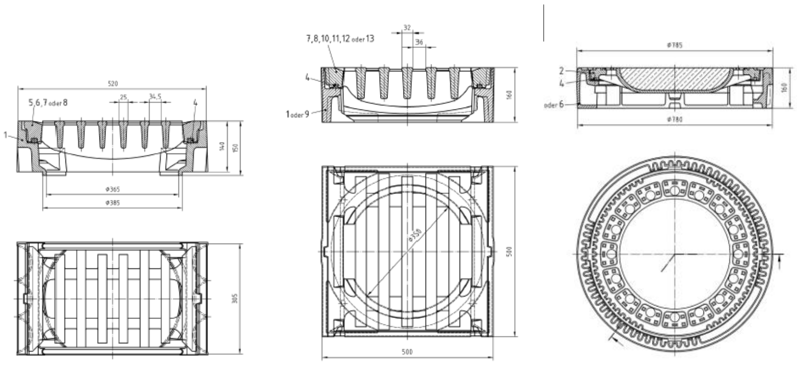

Figure 5. Exemplary gully geometries, left: 300 mm x 500 mm, A = 0.0515 m², middle: 500 mm x 500 mm, A = 0.0815 m², right: example of a manhole with an inlet area A = 0.025 m² (DIN-19583-1, DIN-19594-1, DIN-19584-1)

The conducted simulation runs with different set- ups and different variations of applied parameters of the coupling process are listed below. The boundary condition for the rainfall intensity is the once in 100 years storm event with a rainfall duration of D = 10 min. for each model run. For variations of the inlet area a duration of D = 30 min. was chosen with the same return period of once in 100 years and its correlating intensity of rainfall.

3. RESULT ANALYSIS

3.1. General remarks

Results will be analyzed concerning the influence of parameter variation within the analytical coupling approach (see Eq. 1) on flooding processes, like e.g. flooding depth, flood duration and exchange rate between both models. This exchange rate is called discharge MOUSE to MIKE21. Therefore, it is positive for a surcharge flowing from the 1D-sewage model into the 2D surface model. Furthermore, it is negative for an inflow running from the 2D model into the 1D model. Generally, a surcharge can be observed with early time steps, which is characteristic for the investigated catchment area. After a defined time, the exchange turns into an inflow discharge into the gully. (The boundary condition of the rainfall time series starts after 10 min. of the simulation period for both models).

Following model variations will be investigated:

(1) Influence of implemented measures (simulation period 2 h, grid resolution 2 m, 2D and 1D-2D)
(2) Comparison between 2D and 1D-2D model (simulation period 2 h, grid resolution 1 m)
(3) Influence of inlet area and inlet capacity variation (simulation period 2 h, grid resolution 2 m
(4) Influence of discharge coefficient variation (simulation period 2 h, grid resolution 2 m)
(5) Comparison of 1D sewage model with in-situ measurements (simulation period 8 h)

For all parameter variations, a once in 100 years rainfall event is used, with an intensity of rainfall of $r_{(15,1)}$ = 106 l/(s ha)[1] which corresponds with an investigation of the DWD of rainfall data analysis of Lübeck

[1] In this case D = 15 [min]. and n = [1/a] with D: duration and n: frequency of rain event per year and T = [a] annuity

from 1973. For data analysis, an exemplary control node was selected (point nr. 1) to identify flooding variations and changing exchange rates. The chosen node is a low point with a nearby manhole (see Figure 2 and Table 1). In this case, only point 1 is referred to in the results. It is located on the street and is a low point where flooding depth usually increases significantly during storm events (See Figure 2).

The coupling process will be analyzed by investigating both the separated 2D surface model and the coupled 1D-2D sewage-surface model. Figure 6 gives exemplary results. It shows that the flooding depth and its time-dependent development vary significantly. With the 2D surface model, a continuous increase of resulting flow depths can be observed. As opposed to this, the coupled model shows a major increase just after the beginning of the rainfall event for the first 25 min. until a peak flow depth is reached. Finally, the sewage system will be involved in the discharge process and subsequently a decrease of inundation can be observed. It should be noted that the flooding process occurs with greater intensity within a shorter time period due to the effects of involved underground pipe systems. In this regard, Johnson (2013) refers to the Direct Rainfall Method (DRM), which cuts the flood peak due to losses especially in cases of low intensities with separated 2D overland computation (trough losses due to the bathymetry). This phenomenon is presented in Figure 6. The DRM takes only trough and depression losses of the bathymetry into account, but no evaporation and infiltration.

The influence of the 1D sewage system can be claimed to contribute to defined and fast flood levels on the one hand and to drain the surface floods on the other hand. Whereas the 2D computation results in an accumulation of flood level during the simulation as there is no drainage capacity available and no infiltration or evaporation implemented in the model. The results show reasonable differences between the two model configurations and leads to the statement that the coupling is implemented correctly, because the peak of the flooding depth is not shifted towards the end of the simulation period as in the 2D computation. Correctness can also be verified according to the above-mentioned correlations.

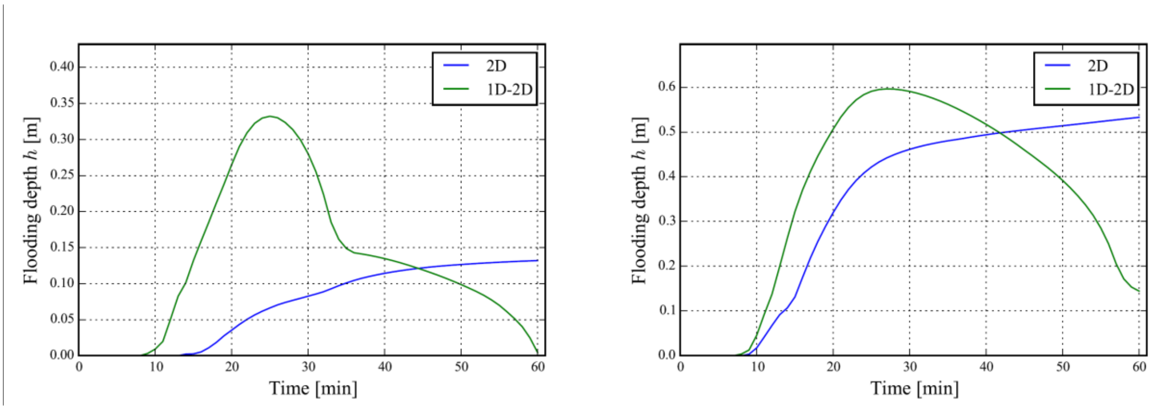

Figure 6. Comparison of exemplary model results (flooding depth) for the simple 2D surface model (2D) and the coupled sewage-surface model (1D-2D), left: T= 5a, right: (b) T = 100a of D = 30 min. (according to Kühnel 2015)

3.2. Inlet area and inlet capacity

The inlet area was varied for manholes only between the aerated area and the completely open manhole without the lid. With gullies the areas are bound to the known values according to their dimensions (see Figure 5) Investigated gullies have their particular area as described previously. The inlet area has a direct impact on the exchange rate between the two models (discharge MOUSE to MIKE21). The effect is comparable to a limitation of the inlet capacity itself (Figure 7 and Figure 8).

It can be found that the discharge for the fully opened manhole is Q = 280 l/s. In comparison with resulting gully discharges, which are mainly responsible for surface drainage, these values are much too high compared to those of gullies defined in Ras-Ew corresponding with Thiele (1983). Consequently, a limitation of the area is recommended to be set as A = 0.025 m² (see Figure 7).
When limiting the capacity to a certain threshold value, the resulting influence on exchange discharges is comparable to an area limitation. Since limiting values for manholes are not established, they have been set as default values within the numerical model (Figure 8) which means to Q = 100 l/s for the manholes, as they are not defined in Ras-Ew.

The gullies' inlet capacity has been limited due to their area and according to Ras-Ew FGSV (2005) to Q = 2.5 l/s and 5.0 l/s respectively (according to their area shown in Figure 5) for a longitudinal slope of 2.5 % and a lateral slope of 1 % to 2 %, according to Ras-Ew FGSV (2005).

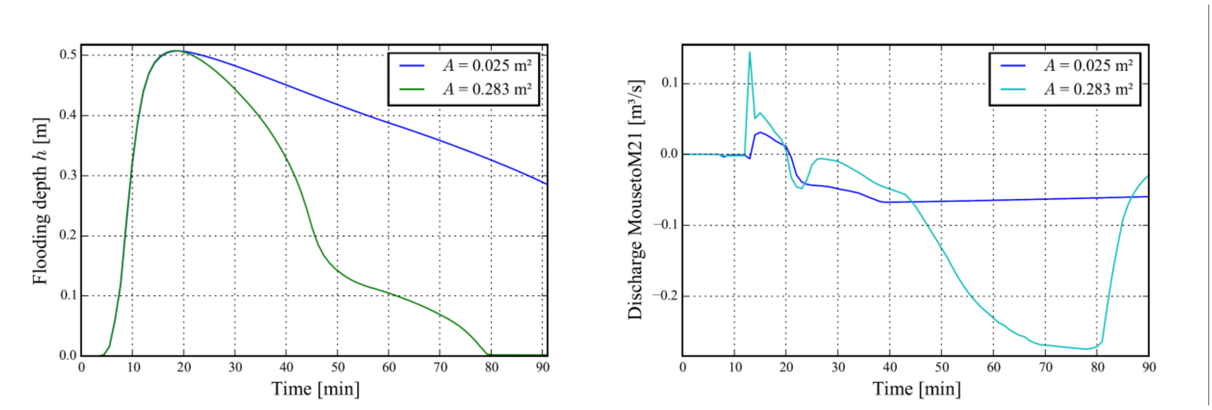

Figure 7. Comparison of manhole inlet area variation (A = 0.025 m² = aerated area and A = 0.283 m² = manhole fully opened), left: influence on flooding depth and flood duration, right: influence on exchange rate (discharge MOUSE to MIKE21) T = 100a and D = 30 min.

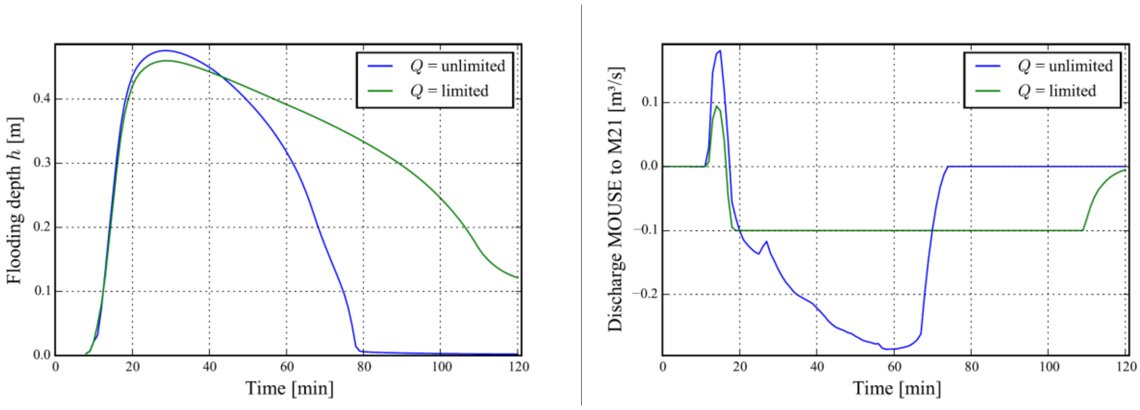

Figure 8. Comparison of limited and unlimited inlet capacity of gullies, left: influence on flooding depth and flood duration, right: influence on exchange rate (discharge MOUSE to MIKE21), T = 100a and D = 10 min.

3.3. Discharge coefficient

Another investigated parameter is the discharge coefficient C_D. Therefore, default values of C_D = 0.98 will be decreased in steps of 1/3 and 2/3 of the default value (C_D = 0.98; 0.65; 0.32). It can be found, that there is only a minor influence on resulting flooding depths and exchange discharges compared to the variations of the area (A = 0.283 m² and 0.025 m²) and the inlet capacity (Figure 9), limited according to the geometry of gullies (Q = 5 l/s or 2.5 l/s) or unlimited capacity. The variation of C_D seems to have minor influence on the flooding depth and the discharge. Similar behavior was observed when changing the area for the manholes. For small variations, no changes were observed. Only for changes of a factor of 10, the different discharges are visible, as shown in Figure 7. Thus, C_D could well be lowered further in order to see a clear impact on the discharge. The implementation of the coefficient is not well documented in the software and needs to be investigated further as the results are not reasonable.

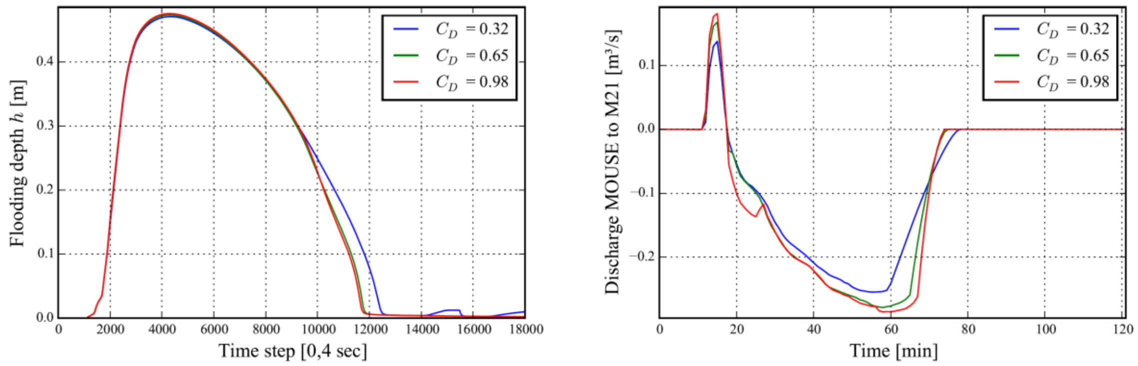

Figure 9. Comparison of discharge coefficient variation, left: influence on flooding depth and flood duration, right: influence on exchange rate (discharge MOUSE to MIKE21), T = 100a and D = 10 min

A reduction of C_D of 1/3 or 2/3 should also result in a reduction for Q in the same order of magnitude. Instead, a change of Q is observed only by appr. 15 to 20%.

3.4. Model verification

The model results have been compared to measured data so far in one spot that drains an area with a separated sewer system. The parameters compared are the water depth in the sewer pipes and the flow velocity respectively. The precipitation data gathered and defined as boundary condition in the model was radar data from the HydroNet-SCOUT portal of hydro & meteo GmbH. The rain intensity was relatively low (6 mm/6h) and did not cause a surcharge given off by the sewer system. Nevertheless, the hydrographs measured and simulated correspond to each other and are in the same order of magnitude (see Figure 10).

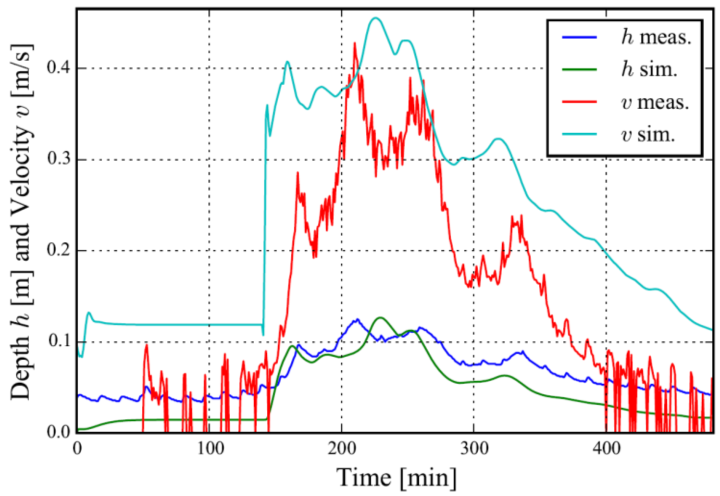

Figure 10. Comparison of measured water depth and flow velocity and simulated results for these parameters with the 1D model, duration from 8 a.m. until 4 p.m. at the 15th of April 2016

Velocity is simulated to a greater extent than it is measured. This can rely on the fact that ground water was infiltrated into the pipes and this caused an algae layer on the surface, which increased the roughness compared to the roughness implemented in the model matching the material of the specific pipe. If no measured data are available, recorded damages and fire brigade data can be used to validate the model (Velasco et al., 2016) by comparing these data with the results of the inundation computation. Both methods where applied here and helped to trust the results.

The model can be used for implementing possible measures at the flooded areas. The comparison of the change of the flood levels can lead to a judgement about the efficiency of different measures.

Figure 11gives an exemplary result of the comparison of implemented measures in the coupled model and of the comparison with a simple 2D model approach.

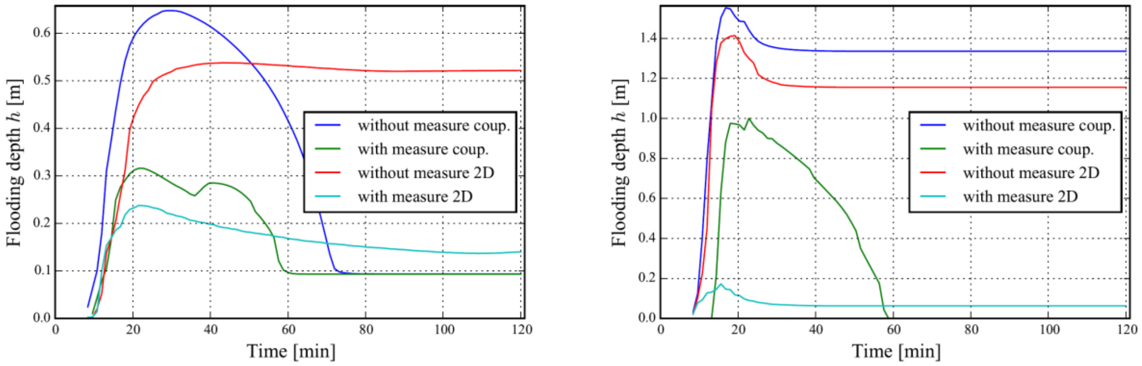

Figure 11. Exemplary results for the comparison of implemented measures at two points within the modelled catchment area for a simple 2D surface model and 1D-2D coupled model, left: a low point on the street, without and with retention basin as measure, right: a low point in front of a house without and with gully as measure, T = 100a and D = 10 min

3.5. Parameter value choice and recommendations for coupled numerical 1D-2D simulations

According to the parameter analysis, the variables can be adopted and changed different from their default state. In terms of manholes the area is set to the aerated area A = 0.025 m². The areas gullies require are set to (A = 0.0515 and 0.0815 m²) the inlet capacity is limited for the gullies in reference to the Ras-Ew (Q = 2.5 and 5.0 l/s). The discharge coefficient with a value of C_D = 0,6 is defined due to sharp-edged openings found in literature (Schneider et al. 2010). The limitation for the inlet capacity of manholes cannot be defined, nevertheless the limitation is implemented by the aerated area because the capacity of manholes is not defined in any regulation or standard as they are no drainage facilities like gullies. Despite of this, manholes do influence the situation in case of completely flooded streets by draining the water.

4. CONCLUSIONS

The different parameter variations have a relatively minor impact on the flooding depth concerning its maximum. The flood duration is clearly influenced. The limitation of the inlet capacity has the greatest impact on the exchange rate between the two models apart from the area limitation of manholes. The default values in the model are not adopted to the situation in urban sewer systems. The values computed due to the default settings do not relate to the values found in the Ras-Ew (FGSV, 2005).

The differences between Ras-Ew and the computed values as well as in in respect to other surveys might be bound to the fact that there is a certain safety factor implemented and that gullies are usually blocked by leaves and sand and are not maintained or cleaned regularly.

Concluding it can be stated that hydro numerical coupled modelling for urban flash flood analysis is an appropriate tool for the identification of areas susceptible of flooding and to implement appropriate measures. The model shows reasonable changes of the flooding depth and duration as well as the exchange rate by the parameters that were varied. The comparison between the results of the only use of a 2D overland computation and the coupled results can also be explained and show that the coupled method is the most accurate analysis in urban areas.

Further events need to be captured, different spots need to be measured and their results need to be compared to the simulated results. Especially storm events with high intensities need to be surveyed in order to prove the parameter settings for the coupling details. For the discharge coefficient, more variations need to be conducted and the implementation needs to be investigated.

To analyze suitable measures and to evaluate the improvement of flooding depth the coupled numerical model is an appropriate tool and should be preferred to the 2D overland modelling as these results show only limited validity.

5. ACKNOWLEDGMENTS

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