Contributions to integrated modelling of stormwater flooding

By

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A thesis submitted for the degree of Master of Philosophy from Imperial College London

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Imperial College London

September 2019

Abstract

Urban stormwater models cover the main physical processes related with stormwater along urban catchments, from runoff generation to routing along overland surface and flow in the sewer networks. These models have been widely used to manage drainage networks in urban areas and have been increasing application for the analysis of floods in urban areas.

Urban stormwater models are based on coupling three main modules: rainfall-runoff, overland flow and sewer flow. The rainfall-runoff transform inputs such as rainfall into runoff, which is then applied to the overland flow module to route over the urban surface area or applied to the sewer flow module which represent the flow in the sewer system.

The present research intends to improve urban stormwater modelling tools, with respect to the analysis of different overland modules setup and formulations, and their interactions with sewer flow modules. The overland module setup can be based on semi-distributed models where rainfall is applied to subcatchments, or fully distributed models where rainfall is applied directly to the overland surface in the overland module; These two different overland modules setup were compared based on two real models implemented in InfoWorks ICM and a model setup that enabled their direct comparison. The interactions between the overland and sewer flow modules were analysed with an innovative approach to delineate stochastic flood hazard maps considering operational conditions of sewer inlets. The overland module was also formulated with the proposed Quasi-2D concept that to represent continuous surfaces based on a 1D scheme.

Declaration of Originality

The work presented in this thesis is my own except where otherwise acknowledged.

Rui Daniel da Silva Pina

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Acknowledgements

I would like to acknowledge the brilliant people I had the pleasure to meet during the work done for this thesis and the sponsors that supported me.

From all the people that I want to acknowledge, a special thanks to Dr. Susana Ochoa, Professor José Alfeu de Sá Marques, Dr. Nuno Simões and Dr. João Leitão, with whom I worked closely during this research and become close friends. I also want to acknowledge all my friends for their emotional support, to mention: João Ferreira, Ricardo Azeiteiro, José Abrantes and João Morais.

The main financial support was from the "Fundação para a Ciência e Tecnologia-Ministério para a Ciência, Tecnologia e Ensino Superior, Portugal (SFRH/BD/88532/2012)", and I also want to give a special thanks to the companies where I have been working that provided their full support for this work: AC, Aguas de Coimbra, EEM; Innovyze Inc; EmNet LLC / Xylem Inc.

Finally, I want to give a special thanks to all my family. For the older generation of the "Familia Pratas&Pinas" and specially to my parents, I want to acknowledge the inspiring example they give me. To the young generation, I want to acknowledge the good times we spend together and the culture "in family" we want to preserve. A huge thanks to my love Cristina and our son Nicolas!

Preface

Some of the chapters of this dissertation are partially based on works published by Rui Daniel da Silva Pina. Although these works have co-authors, the substantial intellectual, research and writing effort was done by the author of this dissertation, and any specific contributions of co-authors have been acknowledged.

Chapter 2. Comparison of semi-distributed and fully distributed urban stormwater models

Pina, R. D., Ochoa-Rodríguez, S., Simões, N. E., Mijic, A., Sá Marques, A., Maksimović, Č., 2016. Semi- vs. Fully-Distributed Urban Stormwater Models: Model Set Up and Comparison with Two Real Case Studies. Water, 8, 58. doi:10.3390/w8020058.

Pina, R. D., Ochoa-Rodríguez, S., Aivazoglou, M., Simões, N. E., Mijic, A., Sá Marques, A., Cedo, M., 2015c. Comparison of Semi vs. Fully Distributed Urban Storm Runoff Models, 17th Conference of SDHI and SDH, 5-6 October 2015, Vršac, Serbia.

Pina, R. D., Ochoa-Rodríguez, S., Simões, N. E., Mijic, A., Sá Marques, A., Maksimović, Č., 2015b. On the use of semi-distributed and fully-distributed urban stormwater models, 10th IWA/IAHR International Urban Drainage Modelling Conference, 20-23 September 2015, Québec, Canada.

Pina, R. D., Ochoa-Rodríguez, S., Simões, N. E., Mijic, A., Sá Marques, A., Cedo, M., Semi-distributed or fully distributed rainfall-runoff models for urban pluvial flood modelling?, 13th International Conference on Urban Drainage, 4-12 September 2014, Kuching, Malaysia.

Chapter 3. Development of improved representation of 1D-2D interactions with stochastic models

Pina, R. D., Leitão, J.P., Simões, N. E., Sá Marques, A., 2017. The impact of time-varying sewer inlet capacity on surface flooding, 10-15 September 2017, Prague, Czech Republic.

Leitão, J.P., Simões, N.E., Pina, R.D., Ochoa-Rodriguez, S., Onof, C., Sá Marques, A., 2016. Stochastic evaluation of the impact of sewer inlets' hydraulic capacity on urban pluvial flooding. Stoch. Environ. Res. Risk Assess. 1–16. doi:10.1007/s00477-016-1283-x.

Leitão, J.P.; Simões, N. E.; Pina, R. D.; Ochoa-Rodríguez, S.; Sá Marques, A.; Stochastic evaluation of sewer inlet capacity on urban pluvial flooding, 10th IWA/IAHR International Urban Drainage Modelling Conference, 20-23 September 2015, Québec, Canada.

Chapter 4. Development of the Quasi-2D concept for 2D modelling

Pina, R. D., Leitão, J.P., Simões, N. E., Sá Marques, A., 2016b. Quasi-2d Modelling with SWMM, 14th International CCWI Conference 7-9 November 2016, Amsterdam, Netherlands.

In addition to the above publications, during his PhD Rui led or substantially contributed to several other publications which are not included in this dissertation.

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1 Introduction

Urban stormwater models are simulation tools that describe the main physical processes related to stormwater routing across urban catchments. They are usually based on coupling three main modules: rainfall-runoff, overland flow and sewer flow. Rainfall is the main data input for the rainfall-runoff module that transforms it into the runoff. Runoff is then input to the overland module, which routes the flow over the urban surface area, and to the sewer flow module which accounts for the flow in the sewer system.

1.1 Background to this research

One of the first urban storm water models developed was the Storm Water Management Model (SWMM) (US EPA, 2014) with an initial release in 1971. It is based on the integration of a rainfall–runoff and one-dimensional (1D) sewer flow modules, which concept is known as 1D models, and was initially developed to analyse combined sewer overflows (Huber and Roesner, 2012). This modelling concept was extended by Abbott (1993) by adding a 2D model of the overland flow, which is known as 1D2D models, and became popular in the late 2000s with the advances in technology (Carr and Smith, 2007; Chen et al., 2005; Seyoum et al., 2012; Smith et al., 2004).

The research presented is focused on 1D2D models with respect to the analysis of different 2D overland modules setup, their interactions with 1D sewer flow modules, and alternative 2D modelling formulations based on 1D schemes, which background is detailed in the following subsections.

1.1.1 Semi-distributed and fully distributed urban stormwater models

Urban stormwater models can be considered semi-distributed (SD) or fully distributed (FD), depending on the spatial discretization of the rainfall–runoff module. SD models are based on sub-catchment units with various land use types, where rainfall is applied and runoff volumes are estimated and routed. In FD models, runoff volumes are estimated and applied directly on the elements of a two-dimensional (2D) model of the overland surface. In SD models, conceptual empirical or physically based methods transform runoff routing into inflows hydrographs, which are applied to the selected computational nodes of the sewer system. Not every inlet is modelled but they are clustered to computational ones. FD models are based on a more realistic approach, since the generated grid-cell runoff is directly routed in the 2D overland flow module.

Traditional urban stormwater models have mostly been SD. One of the first widely implemented urban storm water models is the Storm Water Management Model (SWMM) (US EPA, 2014) with an initial release in 1971. It is based on the integration of a rainfall-runoff and a one-dimensional (1D) sewer flow modules, and was initially developed to analyse combined sewers overflows (Huber and Roesner, 2012). Later on, Ellis et al., 1982 (Ellis et al., 1982), introduced the application of the overland flow module with the dual-drainage concept, by splitting the rainfall in 1D sewer flow module and on the 1D overland flow module that is known as 1D1D model. This concept was extended by Abbott, 1993 with a 2D model of the overland flow, which is known as 1D2D model.

The use of the overland flow module only had major developments with the introduction of the Geographical Information Systems (GIS) in the end of 1990s and first decade of 2000. At first, 1D1D models were significantly improved and opened the discussion about overland flow modelling (Djordjević et al., 2005, 1999; Maksimović et al., 2009; Mark et al., 2004; Vojinovic and Tutulic, 2009). In the late 2000s, 1D2D models become more popular with the development of technology and the increase in the computer power (Carr and Smith, 2007; Chen et al., 2005; Schmitt et al., 2004; Seyoum et al., 2012). Nonetheless, rainfall-runoff modules that have been usually applied in urban stormwater modelling are commonly simplified with SD models. FD models have been typically applied in the large-scale hydrology modelling, with models like Mike SHE (Christian Refsgaard et al., 2010; "MIKE SHE," n.d.) and MOHID Land (Mohid wiki, 2014; Neves et al., 2014) amongst others. In these large-scale applications, modelled

catchments usually have a larger area than the urban ones, coarser spatial resolution, and models do not consider urban features, such as buildings and curbs.

Recent developments, however, bring new opportunities for detailed and physically based modelling of urban stormwater systems. Examples of important advancements are: increase of available data (e.g. digital map (Leitao, 2009), advanced collaborative sources of information (OSM, n.d.), weather radar data (Wang et al., 2013)); advances in technology (e.g. remote sensing (Fletcher et al., 2013), computing techniques (Smith et al., 2013)); and improvements of numerical methods (e.g. reduction in simulation times in 2D overland modelling (Stelling, 2012), new mathematical approaches (Casulli and Stelling, 2013; Ghimire et al., 2013; Simões et al., 2011)). These improvements are opening the discussion for the application of FD urban stormwater models. Infoworks ICM (Innovyze, 2013) already implemented FD models, but its application has not yet become a standard practice in 1D2D modelling of urban catchments. Bailey and Margetts, 2008 (Bailey and Margetts, 2008) discussed the potential of FD models to replace the limitations of rainfall-runoff theories adopted in SD models. By analysing a small case study, the authors achieved similar results with SD and FD models to demonstrate the viability of FD models, but they noted that FD models may still be computationally limited for large scale catchments and should require a significant amount of detailed information to represent all roof and gully connections. (Chang et al., 2015) compared different approach setups of 1D2D models applied to a mid-size real case study. They compared flood extents with performance indicators for different models, and concluded that a combination of SD and FD models is the suitable approach for the analysed case study; however, they noted that FD models require information which is seldom readily available and pre-processing is therefore needed to generate/estimate such information (e.g. to define building connections).

1.1.2 1D-2D interactions and sewer inlets' capacity

The performance of urban drainage systems can be substantially affected by the operational condition of its components (Veldhuis et al., 2011). One of the main components of urban drainage systems are inlet structures; this is, gullies, kerbs, slotted inlets, etc. through which surface storm-water runoff enters the sewer system and occasionally exits it when sewers surcharge (Djordjević et al., 2005). Despite its importance, the discharge capacity of inlet structures and, its variability due to operational conditions is usually overlooked in urban drainage models and model-based assessment of urban pluvial flood hazard.

Several guidelines exist which describe the hydraulic behaviour and discharge capacity of inlet structures of different types. These guidelines, usually of national/municipal character, include recommendations about gully separation, geometry, type of cover, amongst others. Among the most popular guidelines are those by the US Federal Highway Administration – HEC-22 (Brown et al., 2009), which are often used in other countries outside the USA. In the case of Portugal, which is relevant for the present study, the Direcção Geral de Recursos Naturais published national guidelines for the design and construction of gullies (DGRN, 1991). While these guidelines provide valuable information to design inlets and improve their representation in urban drainage models, the mathematical representations they present are usually limited to optimal operational conditions (i.e. fully clean inlets). However, blockage of inlet structures by accumulation of debris can occur depending on the type and location of inlets, calendar season (e.g. higher tree leaf fall-rate in late summer and autumn), antecedent weather conditions (e.g. higher transport of debris with first flush) and sewer inlet cleaning regimes.

To take clogging conditions into account, several authors and local authorities suggest the assumption of constant reduction factors of inlet capacity in design projects. In the USA, example of municipalities suggest that a single gully should be sized with 50% capacity reduction and kerb opening-inlet with 10% capacity reduction (CCRFCD 1990, CDOT 2000). In Italy, the Manual of Sewer Systems (Artina et al., 2001) proposes a reduction of 25% of the inlet efficiency to consider clogging conditions. Guo, 2006 recommends reducing the inlet capacity to take into account clogging; for single-units, the author suggests reducing by

25% the opening area of kerbs and by 50% the length of gullies, and proposes a methodology to account for multiple inlet structures; Almedeij et al. (2006) considered a clogging factor of 50% to single-unit inlets located in terrain depressions and also suggested a 25% reduction for double gully structures. Gómez et al. (2013) monitored one urban catchment in Barcelona, Spain, for one year and observed that clogging can reduce hydraulic capacity between 26.5% and 67.4%. This highlights the importance of inlet clogging on the amount of surface runoff that can enter the sewer system; consequently, more runoff stays on the surface, eventually leading to surface flooding, even for relatively small rainfall events (Dawson et al., 2008; Veldhuis et al., 2011). Given the frequent occurrence of inlet clogging and the large impact that these can have on the performance of the drainage system and on the occurrence of flooding, it is important to properly account for these circumstantial factors in urban drainage models, by considering their stochastic nature.

1.1.3 Overland modelling in 2D and 1D

In urban drainage models, overland flow has been analysed with 1D and 2D schemes. These schemes are a representation of the urban overland surface where storm water can route or pond. The 1D schemes are typically based on links representing streets and pathways, and 2D schemes are based on continuous mesh of elements that covers the surface.

The application of overland flow models has been applied in the recent decades, and had major developments in 1990's driven by the introduction of Geographical Information Systems (GIS) (Ochoa Rodríguez et al., 2013). Ellis et al. (1982) introduced the dual-drainage concept by coupling a 1D sewer flow module with a 1D overland flow module in North America, that become later known as 1D1D model (one dimensional sewer flow and 1D dimensional overland flow model). Djordjević et al. (1999) improved the 1D1D model concept by improving 1D overland surface representation. 1D1D models are difficult to set up, since it is necessary to represent the overland in a one-dimensional scheme. They produced acceptable results when the overland flow is mostly unidirectional, which happens when the surface water flows mainly through roads (Mark et al., 2004). When the overland flow is higher than roads limits the flow can be multi-directional, which means that 1D1D models are not suitable and a two-dimensional approach can be necessary (Allitt et al., 2009). In addition, results of 1D1D models are not easy to visualise and floodplain delineation needs post-processing algorithms (Vojinovic and Tutulic, 2009). However, they have the advantage of being simple and require low computational efforts and runtimes, making them powerful for flood forecasting (Simões et al., 2011). 1D1D models recent developments are essentially related with building the surface network for the overland flow model and with the interactions between overland and sewer flow models (Djordjević et al., 2005; Leandro, 2008; Leitao, 2009; Maksimović et al., 2009; Simões et al., 2011).

More physically based models consider a 2D overland flow model linked with a 1D sewer flow model (1D2D). Besides being introduced in 1990's (Abbott, 1993), they only become popular in the last decade with the developments of the technology and increase in computer power (Carr and Smith, 2007; Chen et al., 2005; Schmitt et al., 2004; Seyoum et al., 2012). These models are easier to set up, because they consider a 2D overland surface without transforming it in a 1D scheme, as the 1D1D models. They enable more realistic analysis of overland flows than the 1D1D approach, especially in extreme events in which flood flows are not confined to roads profiles (Maksimović et al., 2009). However, they require detailed input data about the surface (e.g. Digital Elevation Models and land use with high resolution) and large computational effort and runtimes, which can make them not appropriate for several applications such as flood forecasting (Leitao, 2009; Simoes, 2012).

Hybrid models, which couple 1D1D and 1D2D together, were proposed by (Simões et al., 2011) to bring the best of both 1D1D and 1D2D together. The concept is to consider 1D1D models in the overall catchment with a 1D2D approach in problematic areas where detailed analysis is necessary. They still

difficult to set up, since it is necessary to build 1D overland schemes, but have the advantages of running faster and with less computational effort than 1D2D models.

The *Quasi-2D* concept was developed by the author of this Thesis and was presented for the first time in CCWI (Pina, R. D., et. al. 2016b). The *Quasi-2D* concept is based on the representation of the overland surface with a grid of rectangular elements that is converted to 1D nodes and links. This representation of the overland surface in 1D elements enables the direct application of 1D models to solve runoff routing in the overland surface. With this concept, 1D schemes are applied to solve routing on continuous 2D overland surfaces. This concept differs from the typical 1D concept applied to overland representations, because the surface is covered by a continuous regular grid of elements similar to 2D schemes, while in the typical 1D concept the surface is represented with channels and ponds to model pathways and surface depressions. The *Quasi-2D* concept enables SWMM to perform analysis over 2D surfaces with 1D schemes and equations.

1.2 Hypothesis and research aim

The hypothesis of the proposed thesis is: "Integrated urban stormwater models improve the holistic understanding of the urban water cycle". The main aim of this thesis is to improve urban stormwater modelling tools with more accurate representations of 2D overland modules and their interactions with 1D sewer flow modules. This aim is achieved through the following detailed objectives:

1. Comparison of semi-distributed and fully distributed urban stormwater models

The setup of 2D module can be based on semi-distributed (SD) and fully distributed (FD) discretisation, which were compared using models implemented in InfoWorks ICM for two real case studies (Cranbrook, UK, and Coimbra, Portugal). SD models consider rainfall applied to subcatchments while FD models apply it directly in the 2D models.

2. Development of improved representation of 1D-2D interactions with stochastic models

The interactions between 2D and 1D modules were analysed with the Coimbra case study. The analysis was based on considering the variable capacity of sewer inlets to generate Stochastic Flood Hazard Maps.

3. Development of the Quasi-2D concept for 2D modelling

The alternative 2D modelling formulation based on 1D models led to a new model concept called Quasi-2D to simulate surface runoff. This concept is based on 1D Saint-Venant equations to simulate runoff along 2D continuous surfaces and has the advantage of being applicable with commonly used 1D models, such as SWMM (US EPA, 2014). The mathematical formulation of this concept was demonstrated for the first time and results compared against 2D (full dynamic) models implemented in a full dynamics 2D model (MOHID).

1.3 Thesis outline

This Thesis is organized by main chapters for each detailed objective: Chapter 2 presents the "Comparison of semi-distributed and fully distributed urban stormwater models"; Chapter 3 introduce the "Development of improved representation of 1D-2D interactions with stochastic models"; Chapter 4 presents the "Development of the Quasi-2D concept for 2D modelling". The Chapter 5 presents the overall conclusions of the research presented.

2 Semi-distributed and fully distributed urban stormwater models

This Chapter presents a comparison between SD and FD urban stormwater models based on full-scale urban catchments models. Innovative concepts are proposed for the model building process and to establish the connection between modules of SD and FD models. The model building process proposed assigns the same data to both SD and FD models to enable a direct comparison of the two models. The connection between modules accounts for the limited sewer inlet capacity and enable representation of the same interactions in both SD and FD models.

The comparison of SD and FD models was based on two real case studies: Cranbrook catchment, London, UK; and Zona Central catchment, Coimbra, Portugal. The Cranbrook catchment has an area of 8.5 km² and a relatively flat topography, hence surface water ponding is the main cause of flooding. The Zona Central is a very steep catchment with an area of 1.5 km² and the main cause of flooding is related with the insufficiency of inlet capacity, i.e. overland and gutter flow that cannot enter the sewer system. Comprehensive and detailed analyses of modelling results were applied for both case studies. In the Cranbrook catchment, modelling results were compared with flows and water depths records in sewers. In the Zona Central catchment, flooding extents have been analysed based on photographic records of flooding events. Models were calibrated against monitoring data and photographic records of flooding events. Further analyses are presented with design rainfall events to access the importance of surface storage in both models.

This Chapter is structured as follows: The Section 4.1 presents insights into SD and FD modelling approaches and defines the concepts for model building and to represent the interactions between modules of SD and FD models. In Section 4.2, the case studies are introduced, and Section 4.3 presents the comprehensive and detailed analysis of modelling results.

2.1 Methodology: Semi- and fully distributed modelling approaches

2.1.1 Conceptual basis of semi-distributed and fully distributed models

SD models are based on the definition of subcatchment units, delineated based on analysis of the draining areas towards given discharge points - computational nodes (Figure 1). Each subcatchment is approximated by a regularly shaped (rectangle) surfaces which has uniform morphological and hydrological characteristics (e.g. area, slope, imperviousness, infiltration properties) used to define either physically based or conceptual or empirical rainfall-runoff models. A spatial rainfall input is assigned to each subcatchment, which generates inflow hydrographs discharged to a unique point that can be either a node of the sewer system or another subcatchment. SD models can be implemented in 1D, 1D1D and 1D2D models.

FD models are defined by a 2D overland mesh discretisation (Figure 1). The rainfall is directly applied to each 2D element, generating grid-point runoff, and the routing of surface runoff is then simulated directly by the 2D overland flow module. Therefore, FD models are more physically realistic that can replicate runoff processes more realistically. Moreover, because of the type of discretisation, FD models can only be applied with 2D overland flow modules (1D2D models).

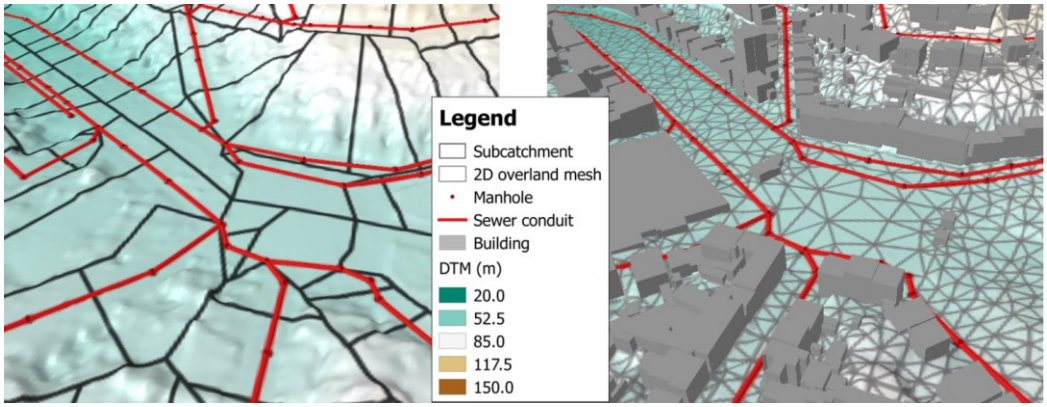


Figure 1. Semi- distributed (left) and fully distributed (right) models.

The main differences between SD and FD models are related to rainfall loss calculation (initial and continuing losses) and runoff routing. They can be summarised as follows:

Initial losses: the main difference is related to the representation of the depression storage. Depression storage is the stormwater that is retained in small depressions on the overland surface (puddle forming) and in pores of surface materials, both in impervious and pervious areas (surface wetting) (Mansell, 2003). In SD models these two phenomena can be considered with a constant value or a single value that is subtracted directly from the rainfall and is dependent on subcatchments' slope and surface type (Butler and Davies, 2011). In FD models, due to the finer resolution, the overland flow module can account for more detailed depressions that originate puddle forming (Bailey and Margetts, 2008).

Continuing losses: the main difference is related to the infiltration modelling. Infiltration is the percentage of rainfall draining into the soil. In SD models, infiltration is estimated for each subcatchment based on soil saturation and subtracted from the rainfall before being applied to the model. In FD models, rainfall is applied directly to the overland mesh and infiltration is estimated for each 2D element, based on soil saturation and water depth. Therefore, infiltration predicted by FD models takes into account the runoff quantity on the overland surface and can capture infiltration into permeable surfaces of runoff routed from upstream impermeable areas.

Runoff routing: In SD models, the generated runoff is transformed by the rainfall-runoff module into an inflow hydrograph that is usually applied to the sewer flow module. In FD models, the generated runoff is directly applied to the overland flow module and routed in the overland surface. SD runoff routing functions are based on both physically based as well as empirical or conceptual methods, with resolutions defined by subcatchments sizes (Elliott and Trowsdale, 2007; Pan et al., 2012). FD runoff routing is simulated by applying physically based approaches with resolutions defined by the surface overland mesh. While FD models enable the representation of the real connection between impervious and pervious areas on the surface, SD models usually merge the runoff discharges to sewers from impervious and pervious area of subcatchments, unless the subcatchments are either pervious or impervious. In addition, runoff volumes captured by surface ponds are captured by FD models, since they consider the runoff on the overland mesh, whereas in SD models can neglect these volumes depending on subcatchment delineation and their discharge definition.

2.1.2 Model building process

The proposed model building process was defined to assign the same data and representation to both SD and FD models. While the 1D sewer flow and the 2D overland flow modules are equally set up for both

models (both SD and FD models can be based on the same 1D2D model), the rainfall-runoff module needs a different procedure to assign data to subcatchments in SD models, and to the overland surface mesh in FD models. The procedure proposed is based on assigning percentages of land use types for each subcatchment (in the SD model) and a land use category for each element of the overland surface mesh (in the FD model) (Figure 2).

In SD models, each subcatchment is defined by the percentage of the land use cover (e.g. has a percentage of area covered by road, parks, etc., each surface having the modelling attributes of the defined land use type). In FD models, each mesh element is characterised by one land use type (e.g. can be considered road or park with corresponding properties). The 2D mesh should be delineated considering boundaries of land use polygons to ensure that each 2D mesh element has only one land use type. Buildings polygons can be considered as voids in the 2D mesh, and their roof runoff is modelled in the FD model as subcatchments that discharge directly to the sewer network to consider private connections. This procedure guarantees the input of the same data for both SD and FD models, despite their different spatial resolution.

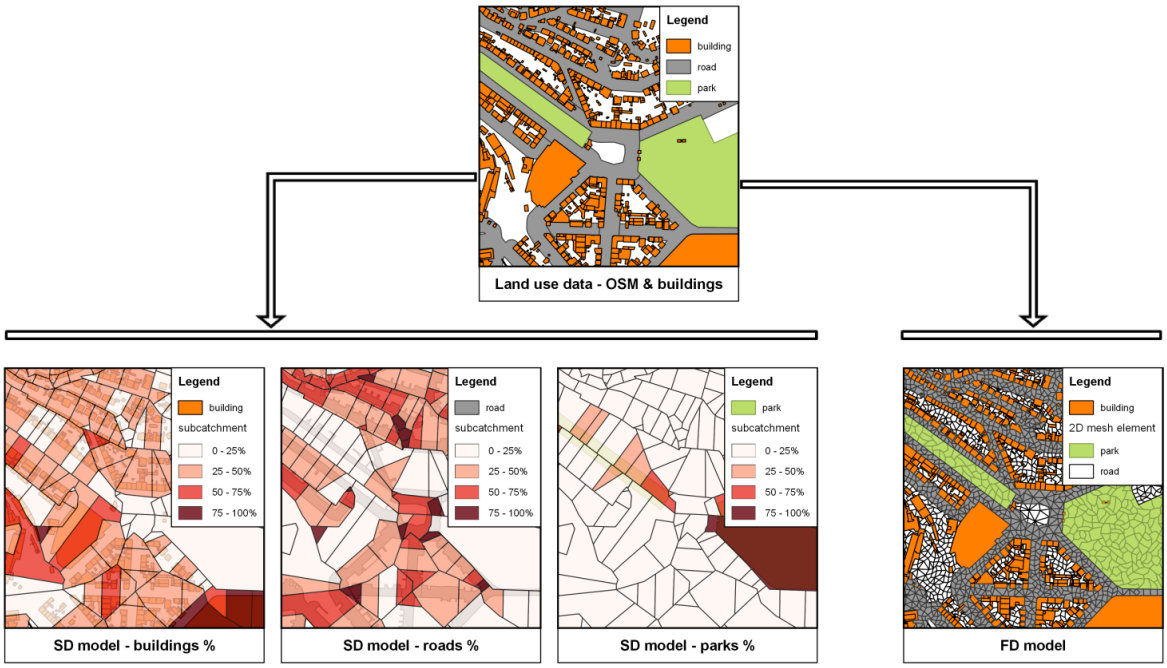


Figure 2. SD and FD rainfall-runoff land use assignment.

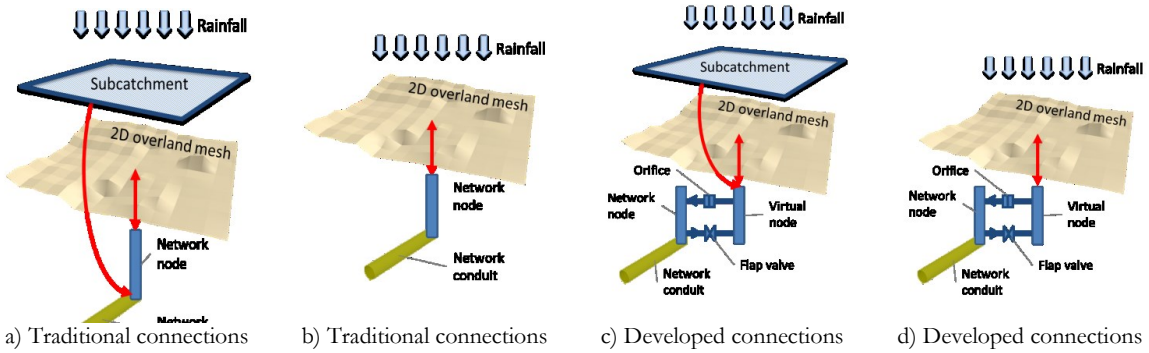
2.1.3 Connections between modules and inlet capacity

The amount of water entering the sewer system is limited by the capacity of sewer inlets; nonetheless, this fact is not always considered in urban drainage models. SD models can consider the inlet capacity if the subcatchments are delineated for each sewer inlet and sewer inlet represented in the model. However, most SD models simplify the representation of sewers inlets and the runoff estimated in a given subcatchment is directly discharged into a selected computational node of the sewer system, without accounting for the inlet capacity (Figure 3.a). Neglecting the limited capacity of inlets means that the model fails to represent the stormwater ponding and flooding that may occur due to limited inlet capacity, even before runoff reaches the sewer system. As a result, flooding only occurs when the sewer system surcharges.

FD modelling packages, such as Infoworks ICM v.5.5 software (Innovyze, 2013), have included the inlet capacity of sewer inlets in network nodes connected with the 2D overland surface mesh (Figure 3.b). In general, a weir or orifice equation is defined in the manhole to control the inlet capacity with the water level on the surface.

To overcome the limited representation of sewer inlets in SD models, a concept based on virtual nodes was developed as represented in Figure 3.c. These virtual nodes have an infinitesimal volume and are directly connected with the overland surface and with subcatchments. They are also connected with the sewer network manholes through orifices with the limited capacity of sewer inlets. Therefore, the inflow to the sewer system from subcatchments discharges and overland module is limited by the inlet capacity of gullies defined by the discharge curve of orifices. If the inlet capacity is exceeded, runoff remains on the overland surface, as it cannot enter the sewer systems. In addition to orifices links, flap valves links were also adopted in the opposite direction to enable runoff to flow from the sewers onto the 2D surface model once sewer surcharge occurs. The discharge curve applied to orifices links defines sewer inlets capacity is based on recommendations presented by Pina et al. (2010) and (Ally, 2011).

To consider the same inlet capacity in SD and FD models, the representation of sewer inlets in FD models was based on an equivalent concept as defined for SD models but without subcatchments (Figure 3.d). The sewer inlet concept typically defined in FD models (Figure 3.b) was not adopted to guarantee the same connections between modules in both SD and FD models, making them comparable.



a) Traditional connections for SD models. b) Traditional connections for FD models. c) Developed connections for SD models. d) Developed connections for FD models.

Figure 3. Connections of rainfall-runoff with overland and sewer flow modules.

2.2 Case studies

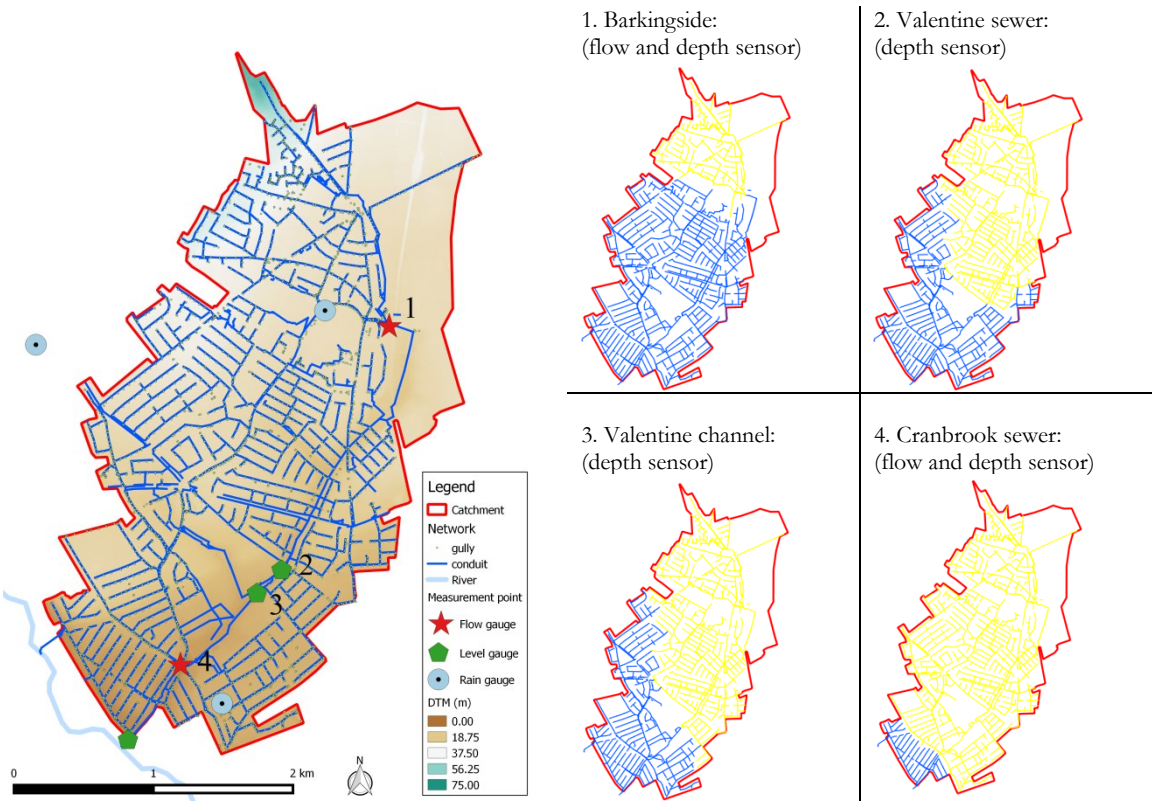
The selected case studies are the Cranbrook catchment, in London, UK, and the Zona Central catchments, in Coimbra, Portugal. For each case study, SD and FD models were implemented in Infoworks ICM vs. 5.5 (Innovyze, 2013) based on the same 1D sewer network and 2D overland flow models (1D2D models). To enable comparison between both case studies, similar data were collected to build the models. The sewer flow model was built with the network and operational data, provided by the respective water companies of the study catchments. The 2D overland flow model was created based on available LiDAR-based Digital Terrain Models (DTM) with 1 m horizontal resolution. Buildings polygons and land use data were used to characterise the model (e.g. roughness and infiltration parameters) and to define the surface mesh (e.g. mesh resolution, break lines, voids, boundaries). The land use data were obtained from the Open Street Map (OSM, n.d.) and buildings polygons were provided by local authorities. The SD models for these case studies have been developed and updated since 2010 and 2009, respectively (Pina et al., 2010; Simoes, 2012). These SD models were improved and calibrated following the UK standards (WaPUG, 2002) using local rainfall and flow records. The FD models for both case studies were developed with the exactly same data as the calibrated SD model, following the methodology presented in previous Section to achieve comparable models.

2.2.1 Cranbrook case study

The Cranbrook catchment is located in the North-East part of London, UK, and is presented in Figure 4. It is predominantly urban (residential and commercial units), with some open green spaces. It covers an area of 8.5 km^2 with an average slope of 5 %. The stormwater sewer system is nearly 98 km long; it is mainly

separate and discharges into the Roding River. This catchment has suffered several floods during recent years (e.g. in 2000 and 2009), which have affected hundreds of properties.

A monitoring system has been operational in the Cranbrook catchment since April 2010 (Figure 3.b). It includes four rain gauges, three water level sensors (one in sewers and two in channels) and two flow gauges in sewers that record water depth and velocity. The most upstream sensor (Barkingside) was installed in December 2014 and covers a limited area with 2km² that is mostly residential. Valentine sewer and Valentine channel sensors are located almost in the middle of the catchment, with upstream drainage areas of 5.0 and 5.5km² respectively. As the names suggest, one sensor is installed in the sewers entering the Valentine Park and the other on an open channel in the Park. Cranbrook sewer is a sensor installed in the downstream area and covers most of the catchment area (8.0km²). There is also a level gauge in the main discharge of the catchment to validate the outfall conditions, since can be influenced by the level of Roding river.



a) DTM and network data. b) Monitoring stations and upstream network.

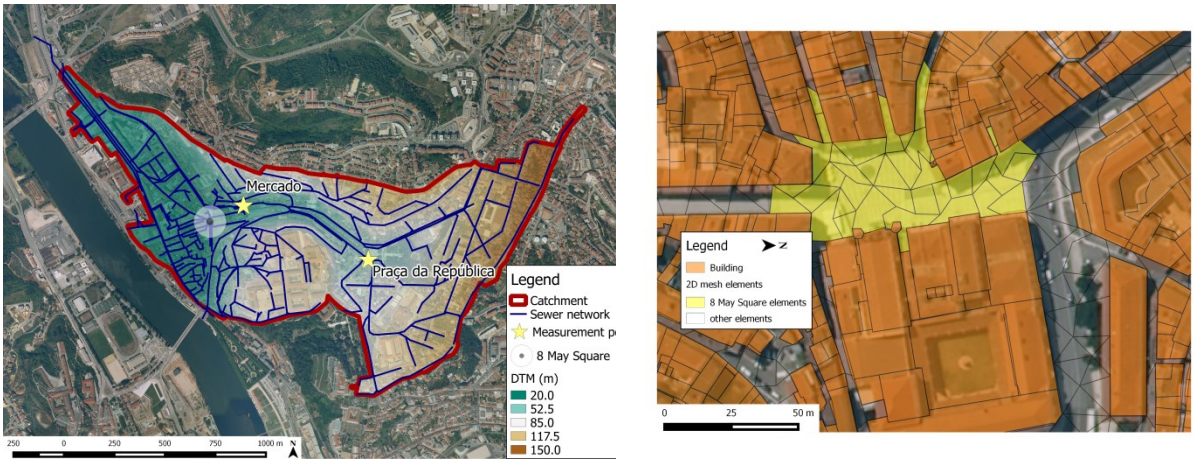
Figure 4. Cranbrook case study – London, United Kingdom.

The SD and FD models for the Cranbrook case study include a 1D network based on a sewer system with 2,596 conduits and 2,546 manhole nodes. The conduits have an average slope of 1 % and cross sections with diameters ranging from 100 mm to 1,950 mm. The 1D network also includes 565 m of open channels with cross sections of up to 6 m width, and 5 storage ponds, 4 of which are recreational lakes. The SD rainfall-runoff model has 4,409 subcatchments with areas ranging from 50 m² to 40 ha, and average of 0.2 ha; slopes are varying from 0.015 m/m to 0.408m/m with an average of 0.05m/m, and widths are ranging from 4m to 357m, with an average of 22m. It considers initial losses dependent on subcatchments' slopes and the Wallingford routing model. Infiltration losses are estimated for both SD and FD models with fixed runoff coefficients with a range from 0.075 to 0.675 for each land use type. The overland flow module, which defines the resolution of the FD model, is based on a 2D mesh with 117,712 elements with areas ranging from 25m² to 992m² and mean of 61m², and Manning's "n" roughness from 0.0125 to 0.020.

Both infiltration data and roughness of 2D elements was assigned to the model based on land used data from Open Street Maps (OSM).

2.2.2 Zona Central case study

The Zona Central catchment is located in Coimbra, Portugal, and is presented in Figure 5. It covers highly urbanised zones, mainly residential and commercial, including the downtown area of Coimbra, where important services and historical buildings are located. It has a total drainage area of approximately 1.5km² with an average slope of 24%. The sewer system is nearly 35 km long, most of which is combined and discharges to the Coselhas brook and into the Coimbra Waste Water Treatment Plant, from where it is further directed to Mondego River. This catchment has suffered several floods during recent years, the occurrence of which is exacerbated by the steep topography and the limited inlet capacity of the sewer system. The area at highest risk of flooding is the Praça 8 de Maio (Figure 5.b), a square in the centre of the catchment, where important services are located (e.g. City Council and tourist attractions) and where flood waters tend to pond due to topographic conditions.



a) Sewer network, DTM and monitoring point locations. b) Extents of Praça 8 de Maio.

Figure 5. Zona Central catchment – Coimbra, Portugal.

This catchment was monitored during 2010-2012 by Simões, 2012 (Simoes, 2012), with 3 rain gauges at different locations in the catchment, and 2 water depth sensors installed in the main sewer upstream of the Praça 8 de Maio, covering drainage areas of 0.4 km² in the "Mercado" station and 1.0 km² in the "Praça da Républica" gauge (Figure 5.a). Continuous rainfall records have been registered with one rain gauge by the water utility company, AC, Águas de Coimbra, E.M., at the border of the catchment. The data collected between 2010 and 2012 was used to calibrate the SD model and the rainfall records collected by the water utility company are the input for the simulations of the flooding events presented in this paper.

The SD and FD models for the Zona Central case study are based on a 1D sewer network with 1,016 conduit links and 1,014 manhole nodes. The conduits have an average slope of 5%, and cross-sections with dimensions ranging from 200mm circular diameter to closed rectangular with the size of 3.5x1.7m. The SD rainfall-runoff model has 911 subcatchments with areas ranging from 50m² to 4.8ha and a mean of 1 722m², slopes ranging from 0.00m/m to 1.13m/m and a mean of 0.24m/m, and widths ranging from 6m to 493m and a mean of 51m. The model implements absolute initial losses and the SWMM routing module. Infiltration losses are estimated for both SD and FD models with the Horton equation for pervious areas and fixed runoff coefficients on impervious areas (from 0.75 to 0.85). The overland flow module, which defines the resolution of the FD model, is based on a 2D mesh with 10,741 elements, with areas ranging from 25m² to 678m² with a mean of 89m², and Manning's "n" roughness from 0.0130 to 0.025. Both infiltration data and roughness of 2D elements was assigned to the model based on land used data from Open Street Maps (OSM).

2.3 Results and Discussion

2.3.1 Cranbrook case study

The analysis of the Cranbrook case study was based on three selected events, for which rainfall and water depth and flow records in sewers were available. The rainfall records are summarised in Table 1, considering the average rainfall calculated based on Thyssen polygons for each rain gauge covering the entire catchment area. For each event, the entire day was simulated, but only the time frame corresponding to the main rainfall event was analysed to minimise errors related to the impact of the initial conditions.

Table 1. Summary of rainfall records used for the Cranbrook case study.

Event	Start	End	Duration (h)	Maximum rainfall over 1 minute (mm/h)	Total rainfall depth (mm)	Average rainfall intensity (mm/h)
141212	12/12/2014 01:30	12/12/2014 08:00	6.5	12	10.9	2
150103	03/01/2015 03:50	03/01/2015 17:00	13.2	12	16.6	1
150108	08/01/2015 07:30	08/01/2015 14:30	7.0	12	11.6	2

The balances presented in Figure 6 show the distribution of volumes among the modules for all the events analysed. Runoff volumes were generated by the rainfall-runoff module, and they were calculated with the subcatchments discharges in SD models, and with the runoff volume generated on the 2D mesh in FD models. Volumes in the overland flow module were calculated with the difference between volumes at the end of the simulation with the ones at the beginning of each event. The volumes in the sewer flow system were defined by the discharges at outfalls in the 1D sewer network.

In all three events, total runoff volumes are similar to the total combined volumes from overland and sewer flow modules. The insignificant differences in the total runoff volumes are caused by small differences in subcatchments' areas in the SD model when compared to the FD model. However, in all simulations FD model retained more water on the surface in contrast to SD model, where most of the runoff is conveyed through the drainage system.

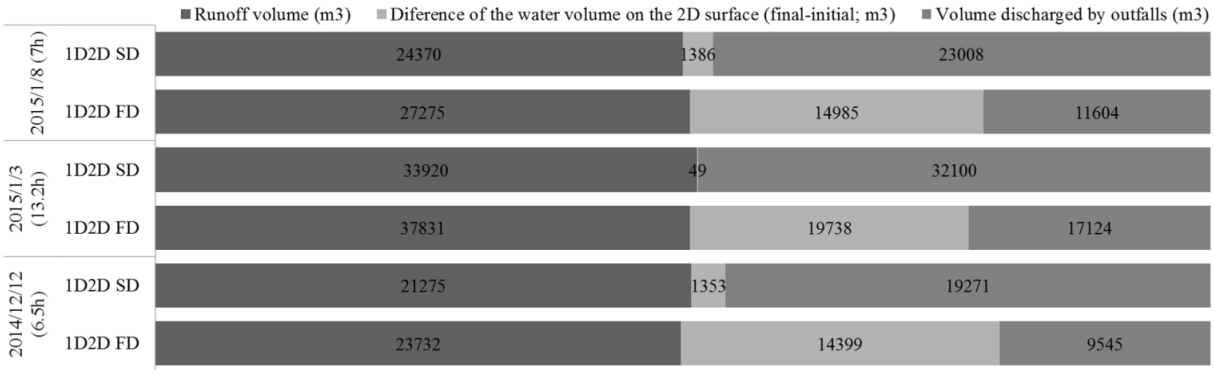


Figure 6. Volume balance for the Cranbrook case study model runs.

To further explore the source of water accumulation on the overland surface, the differences between FD and SD maximum volumes at the surface were divided by land use groups (Figure 7). The most significant differences are observed for roads and buildings (residential, retail and industrial areas) zones, leading to the conclusion that runoff on the overland in FD models is retained due to surface ponding and building

singularities. The category 'Other areas' includes non-classified zones in the land use data that cover a mix between open areas and zones covered by buildings.

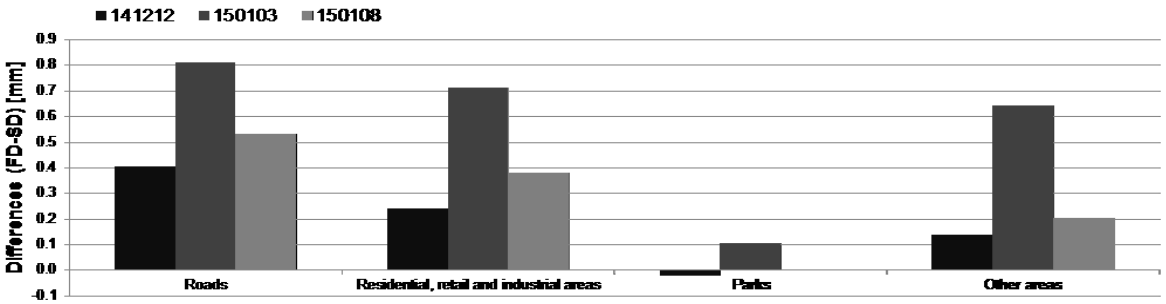


Figure 7. Differences between runoff volumes on the overland surface generated by SD and FD models, distributed by land use groups in Cranbrook case study.

As water volumes in the sewer flow module are generally lower in FD than in SD models, FD results tend to underestimate water depth and flows in sewers, as exemplified in Figure 8 and Figure 9 with two monitoring locations for the event of 2015/01/03. SD results tend to predict temporal pattern and peak values with more accuracy.

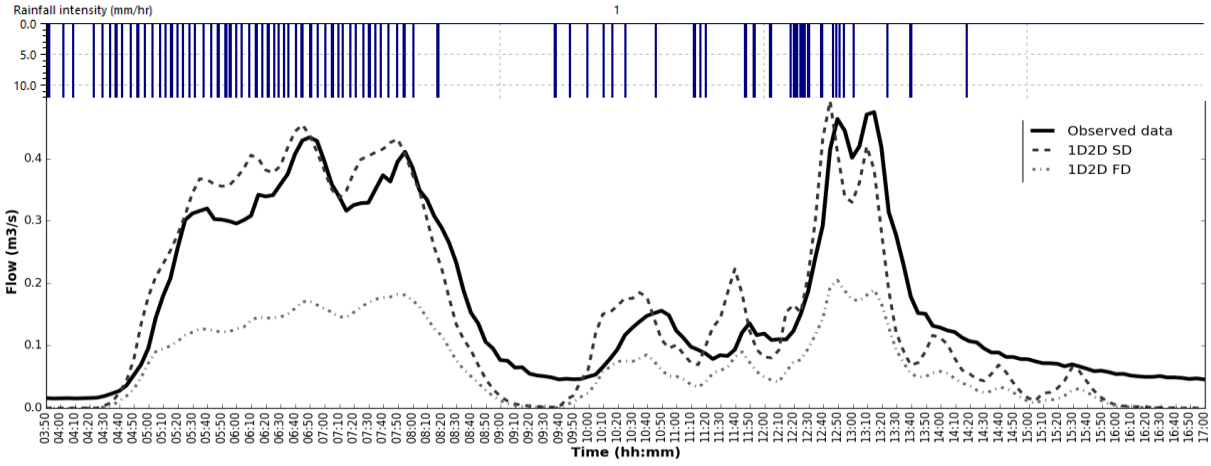


Figure 8. Predicted flow and observed data in the Barkingside gauge, on 2015/01/03 – Cranbrook case study.

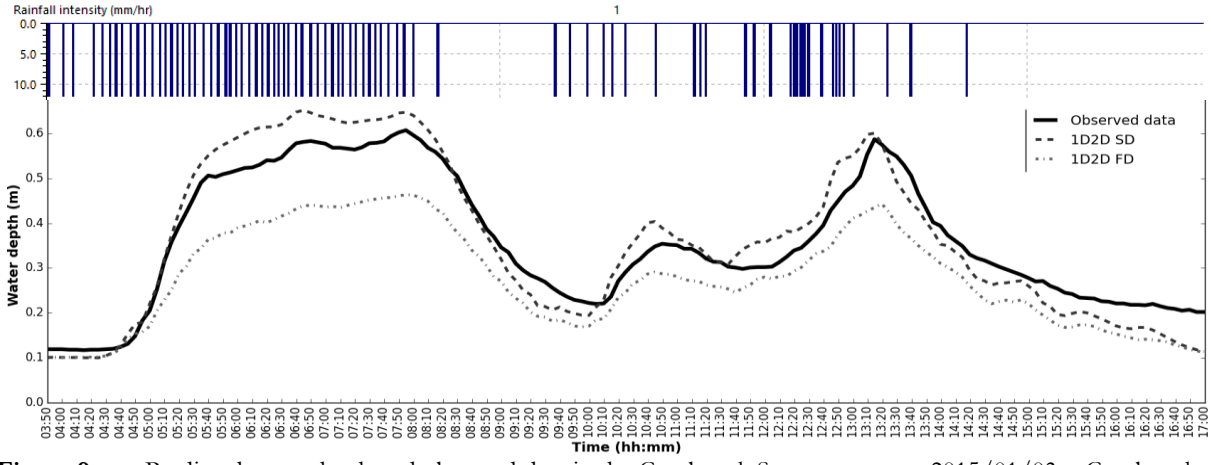


Figure 9. Predicted water depth and observed data in the Cranbrook Sewer gauge, on 2015/01/03 – Cranbrook case study.

The last considerations are generalised to all the events simulated with the statistical analysis presented in Figure 10, based on the following indicators:

Relative error (RE) in peak:

$$\mathrm{RE} = (\mathrm{Vmax}_{\mathrm{obs}} - \mathrm{Vmax}_{\mathrm{res}})/\mathrm{Vmax}_{\mathrm{obs}} \quad (1)$$

where RE is the relative error in the peak results ($\mathrm{Vmax}_{\mathrm{res}}$) compared to the peak in observed data ($\mathrm{Vmax}_{\mathrm{obs}}$). RE was applied to flow and water depth peaks. Positive RE values indicate underestimation by the peak results and negative values imply overestimation. The RE has the advantage of being a 'tangible' statistic that evaluates the performance of a critical parameter such as the peak flow or water depth. It is important to note that very large RE can be obtained when low values are evaluated, even if the absolute difference in peak is small.

Coefficient of determination (R^2) and Regression coefficient (β): resulting from a simple linear regression analysis applied between each simulated results time series and the observed data. These two statistics provide an indication of how well the results replicate observed data, both in terms of pattern (R^2) and accuracy (β). The R^2 measure ranges from 0 to 1 and describes how much of the observed data variability is according with the simulated results. In practical terms, R^2 provides a measurement of the similarity between the patterns of the observed data time series and the simulated results time series, i.e. indicates how the hydrodynamics are captured by the model. The regression coefficient, β, is employed to provide supplementary information to the R^2. $\beta \approx 1$ represents good agreement in the magnitude of observed data and results time series; $\beta > 1$ means the results are overestimated against observed data (by a factor of β); and $\beta < 1$ means the results are underestimated against observed data (by a factor of β).

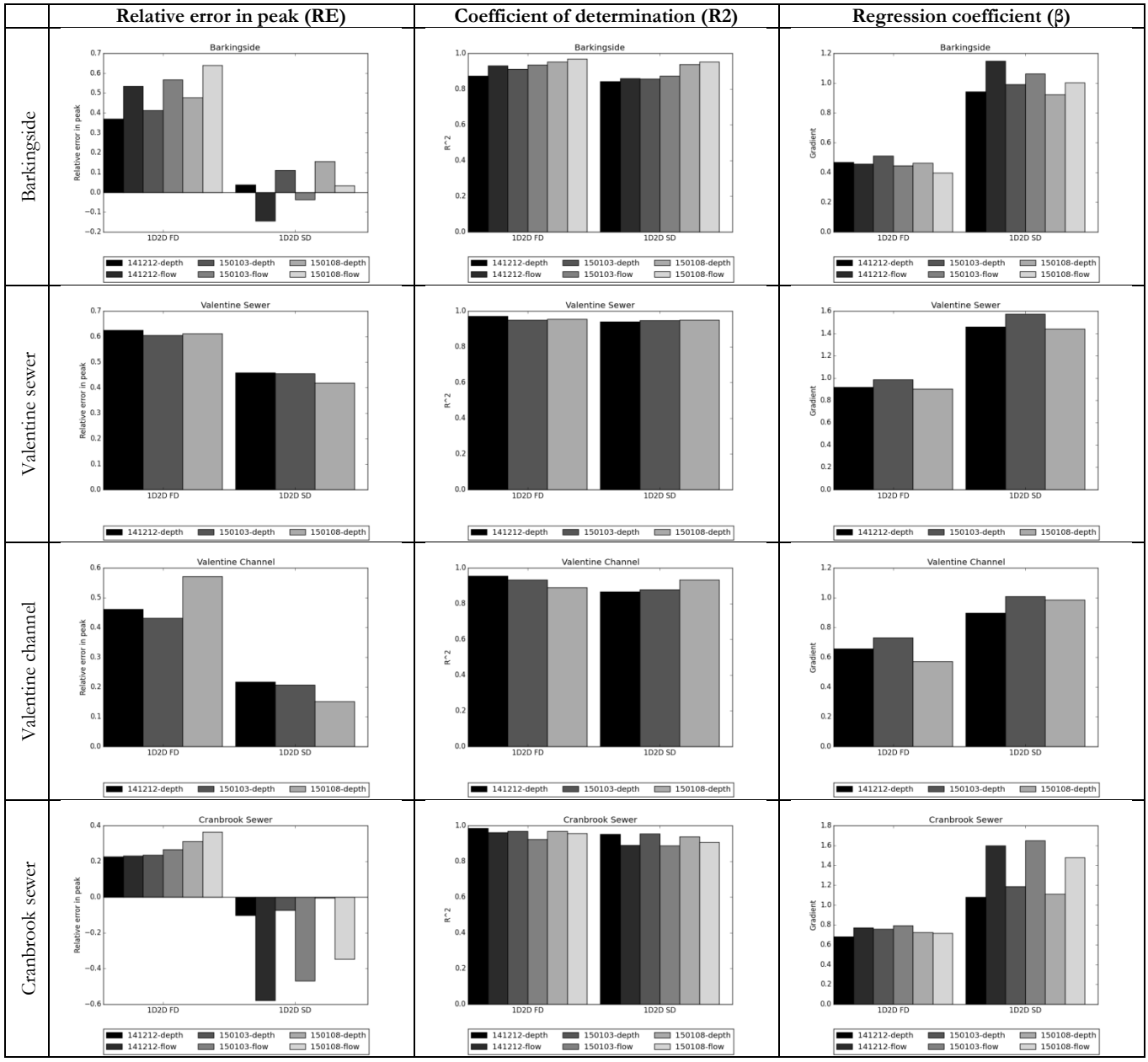


Figure 10. Statistical analysis of modelling results against observed data of Cranbrook case study.

In general, for all simulated events the RE is higher in FD than in SD models, which means that FD underestimates results against observed data. In the Cranbrook sewer sensor, the SD model predicted accurate water depths but overestimated flows. This can be due to the location of the sensor in a zone where turbulence can occur and affect monitoring data accuracy. In the FD model this variation does not occur, because both water depth and flows results are smoothened and underestimated. For most simulated events, the R^2 is close to 1 for both SD and FD models, which implies that the variations of modelling results are matching the observed data, i.e. the models can capture the hydrodynamics of observed data. The differences between SD and FD models are not so evident, but FD models tend to have higher R^2, which suggest that they have the potential to better represent the dynamic behaviour of stormwater flows in urban catchments. The β is in general closer to 1 for SD model results, which indicates that SD results are matching the observed data more accurately than FD ones. The exceptions in the results analysis can be noticed in the data for the Valentines and the Cranbrook Sewer sensors. For the Valentines Sewer sensor, network elements tend to overestimate water depths in the SD model, which is not verified in the FD

model as it underestimates overall results. For the Cranbrook Sewer sensor, the errors in observed flow data due to turbulence also affect this indicator as verified for the RE.

2.3.2 Zona Central case study

The analysis of the Zona Central case study was based on four flooding events for which rainfall records and photographic records of the flooding in Praça 8 de Maio were available. The rainfall records are summarised in Table 2, considering the average rainfall calculated based on Thyssen polygons for each rain gauge covering the entire catchment area and a time resolution of 1 minute for the first two events and 10 minutes for the last two events. The Return Period of each event was estimated based on IDF curves from the Portuguese legislation "Decreto Regulamentar 23/95, 1995-08-23".

Table 2. Summary of rainfall records in Zona Central case study.

Event	Start	End	Duration (h)	Maximum rainfall (mm/h)	Total rainfall depth (mm)	Average rainfall intensity (mm/h)	Estimated return period (yr.)
060609	09/06/2006 14:50	09/06/2006 16:30	1.7	144.0	36.6	22.0	50
061025	25/10/2006 00:30	25/10/2006 05:30	5.0	102.0	56.6	11.3	50
080921	21/09/2008 15:10	21/09/2008 17:20	2.2	60.6	21.4	9.9	5
131224	24/12/2013 06:40	24/12/2013 18:00	11.3	31.5	48.9	4.3	5

The balances presented in Figure 11 show the distribution of volumes between the modules for all the events analysed, in accordance to the analysis presented before for the Cranbrook case study. In this case study, FD models also tend to have higher water volumes at the 2D overland surface than in SD models, and less volume discharged by the outfalls of the 1D sewer network. However, in this case study the differences between SD and FD models are not as significant as for the Cranbrook catchment. This is because the rainfall events selected caused floods in both SD and FD models, which increased the overland surface volumes in the SD model. There are also insignificant differences in the runoff volumes caused by small differences in buildings' area at the boundary of the catchment in the FD model.

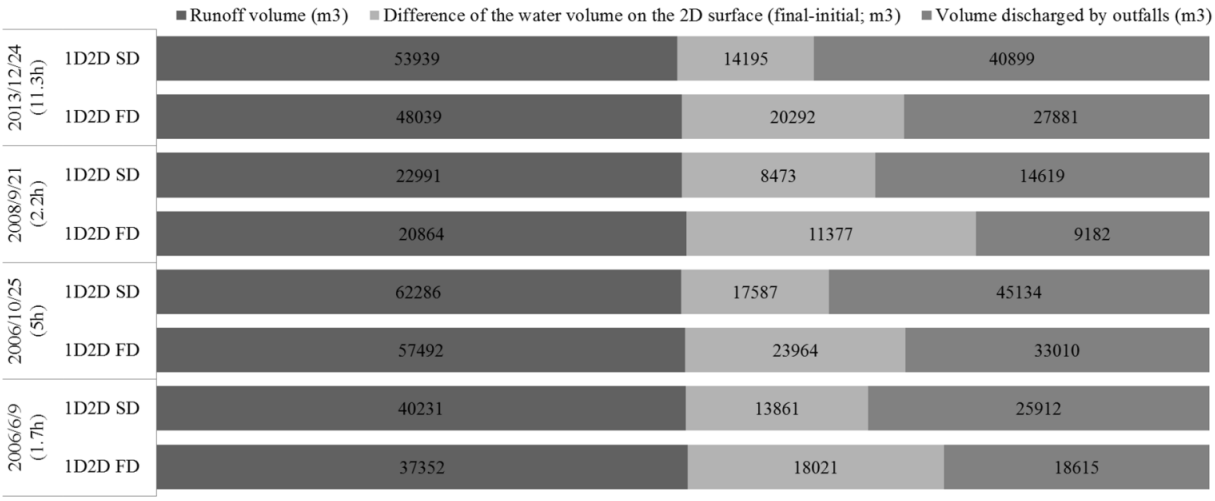
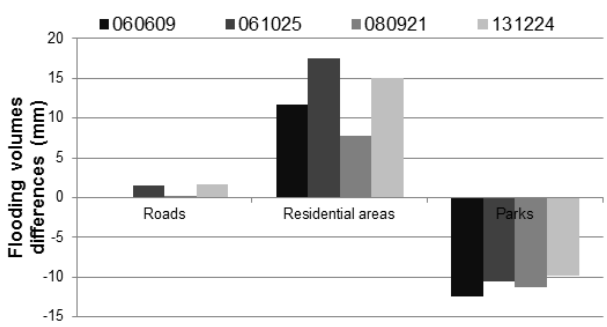
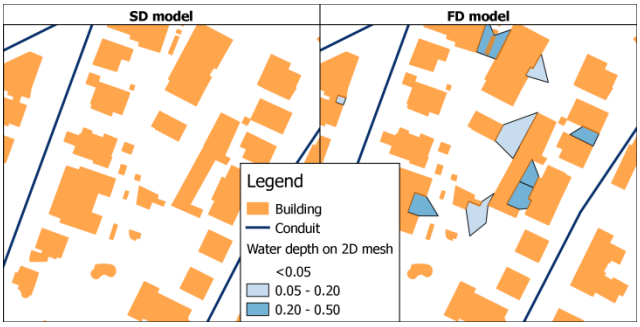


Figure 11. Volumes balance for the Zona Central case study model runs.

To investigate where the FD model retains water on the surface, Figure 12.a presents the differences between FD and SD maximum volumes on the surface divided by the land use groups. Larger discrepancies are registered in zones covered by buildings (residential areas). This means that, opposite to the Cranbrook case study, runoff is retained on the overland surface in FD models due to building singularities, and surface ponding on roads is not a significant problem, as exemplified with Figure 12.b.



a) Runoff volumes distributed by land use types.



b) Example of runoff retained on the overland surface in FD models due to building singularities.

Figure 12. Differences in runoff volumes on the overland surface generated by SD and FD models in Zona Central case study.

The comparison of flood maps generated in the Praça 8 de Maio is summarized in Table 3 for all the events analysed. In general, flooding volumes are higher in SD than in FD model, and water depth and flooding areas follow the same pattern. This means that as FD model stores more water on the overland surface, runoff volumes are retained in the upstream areas and do not get to lower zones where water accumulates in reality. The predicted floods at Praça 8 de Maio were also compared to photographic records of floodplains (taken by local newspaper *Diario de Coimbra*), as presented in Figure 13 for the events with the two highest return periods. It can be concluded that flooding extent is well predicted with the SD model, but underestimated with the FD model.

Table 3. Summary of modelling results on Praça 8 de Maio, Zona Central case study.

Event	Max water depth (m)		Flooding volume (m3)		Flooding area (m2)	
	SD	FD	SD	FD	SD	FD
060609	0.44	0.11	275	68	1092	324
061025	0.51	0.25	360	138	1693	630
080921	0.10	0.08	63	42	324	248
131224	0.14	0.06	79	34	569	248

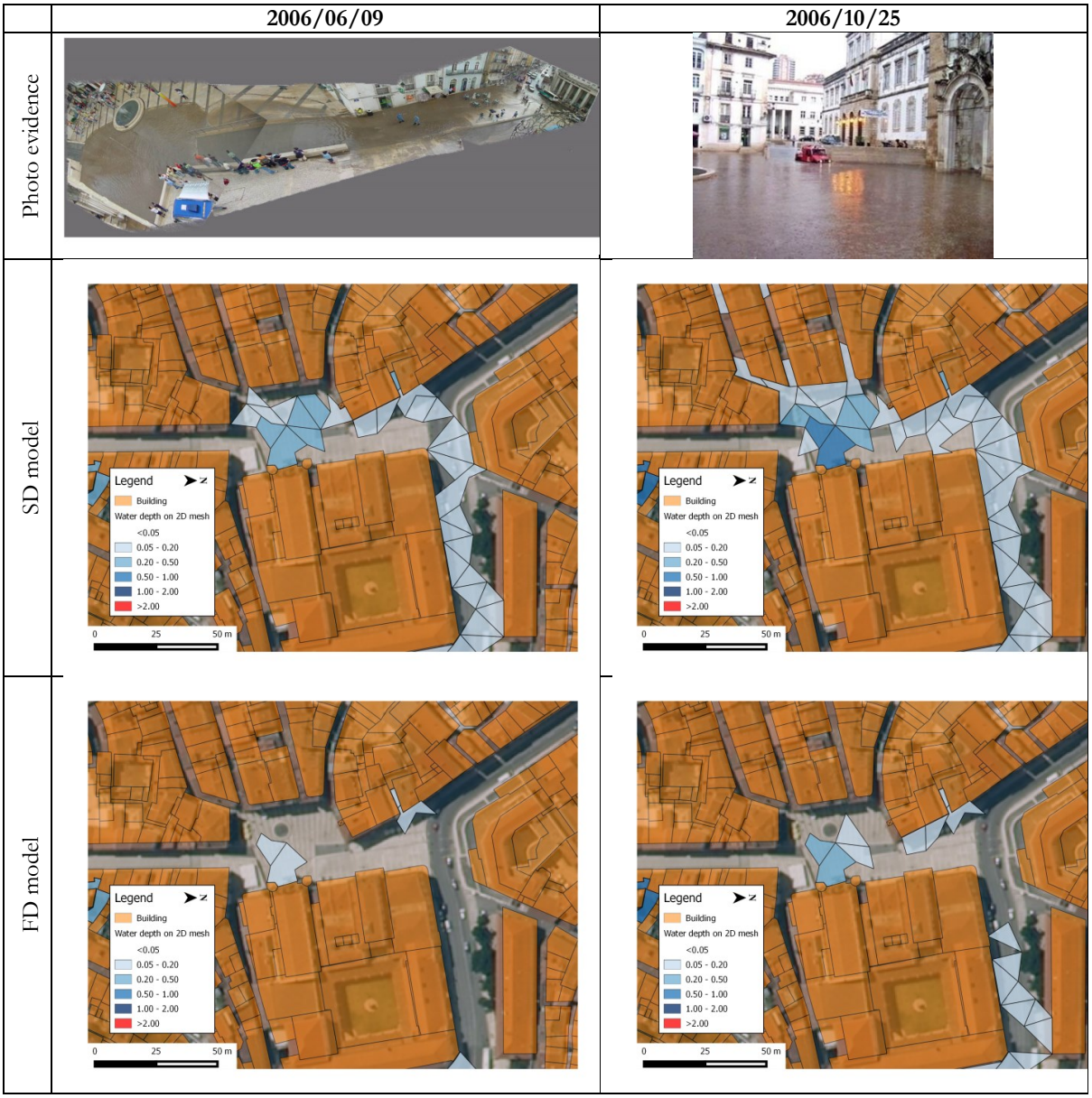


Figure 13. Comparison of photograph records with predicted floodplains on Praça 8 de Maio, Zona Central case study.

2.3.3 Assessing the importance of surface storage

The aforementioned analyses indicate that, in general, FD models retain larger volumes of water on the overland surface as compared to the corresponding SD setup. Depending on the case study, the volume retained can be stored in surface depressions, as occurred in the Cranbrook case study, or it can be retained in building singularities, as occurred in the Zona Central case study. To analyse the importance of the surface storage in relation to the rainfall intensity, an analysis with design storms was performed. The models of Cranbrook case study were tested with five design storms with return periods (RP) of 10, 20, 30, 100 and 200 years, and duration of 1 hour; and the models of Zona Central case study were tested with six design storms with RP of 2, 5, 10, 20, 50 and 100 years and duration of 45 minutes.

Figure 14 and Figure 15 present the relative flooding volumes for each land use group type, calculated based on the maximum water depth predicted at the 2D mesh of the overland surface. It can be concluded that the percentage of flooding volume on roads is higher in the SD model for the two case studies. In both situations, the increase of the rainfall return period, and thus intensity, led to decrease of the relative volumes in roads and an increase of volumes in areas covered by buildings for both SD and FD models.

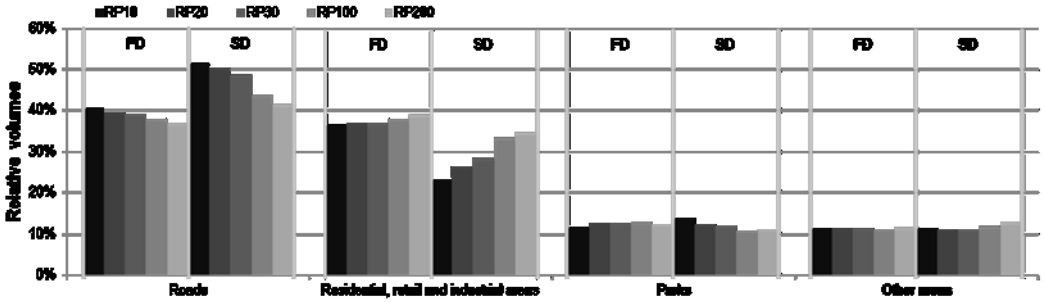


Figure 14. Relative flooding volumes on each land use group for design storms events, Cranbrook case study.

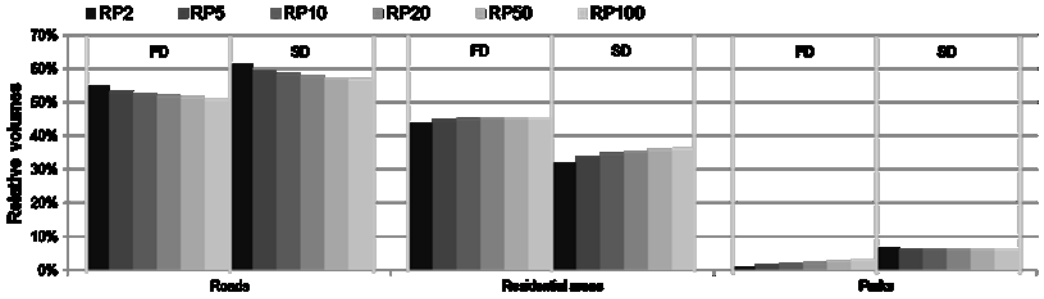


Figure 15. Relative flooding volumes of each land use group for design storms events, Zona Central case study.

Figure 16 shows the difference between SD and FD models in predicting flooding volumes for each land use group type in the Cranbrook case study. In addition to the increase in the relative volume in residential areas, as shown in Figure 14, the differences in flooding volumes between SD and FD models are similar in residential areas for all events. For roads, the flooding volume decreases as water tends to accumulate more in these zones leading to higher volumes in the SD model for high rainfall intensities. Figure 17 presents the same analysis applied to the Zona Central case study. In this catchment, the increase in the rainfall return period led to the increase in the difference between FD and SD volumes for both roads and residential areas. This means that with the increase in the rainfall intensity, higher volumes are retained on the overland surface by the FD model, as compared to the SD simulations.

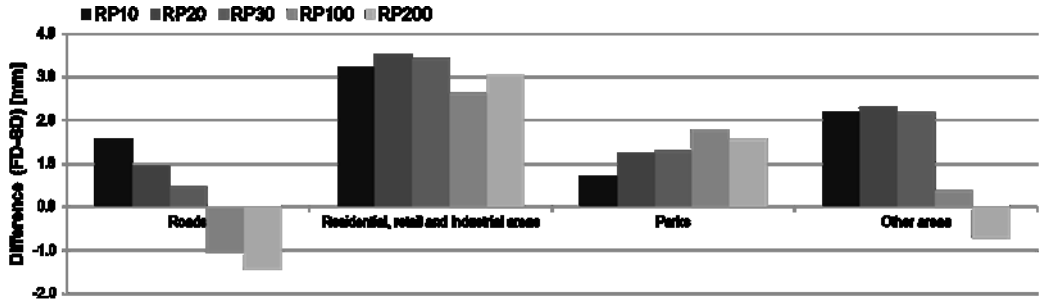


Figure 16. Differences in flooding volumes of each land use group for design storms events, Cranbrook case study.

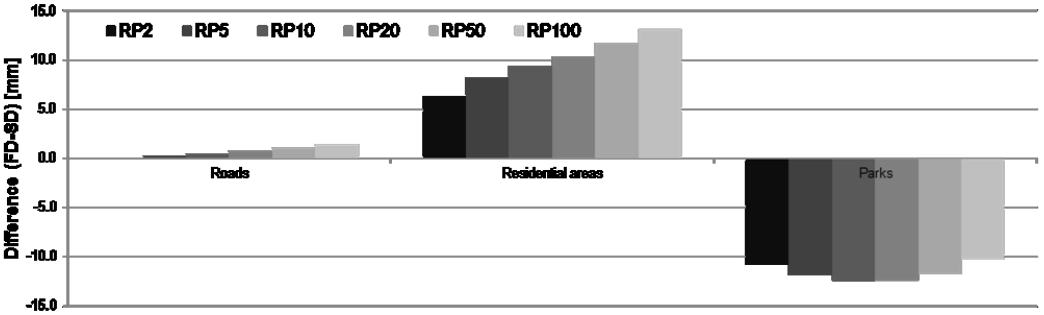


Figure 17. Differences on flooding volumes of each land use group for design storms events, Zona Central case study.

To assess the importance of the volume that is retained on the overland surface, Figure 18 and Figure 19 analyse the differences between flow volumes in the 1D network. It can be verified that differences between SD and FD models increase with the increase in upstream areas and decrease for higher intensity rainfalls. However, the differences have significantly distinct trends for each case study. In the Cranbrook case study, surface storage can be neglected for high intensity rainfalls, converging to low percentages for all monitoring point locations. In the Zona Central case study, however, surface storage is significant for downstream monitoring point locations for all the rainfall intensities tested. While in Cranbrook case study the surface storage is mainly related with surface depressions, in the Zona Central catchment the surface storage verified in the FD model is also related to buildings singularities, and the absence of data about private drainage networks and connections.

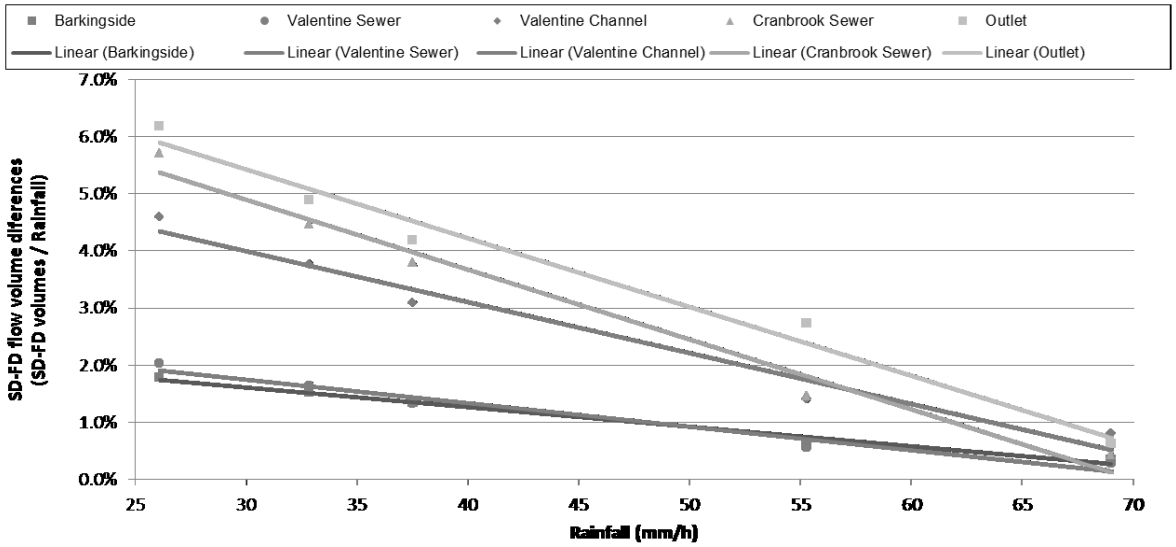


Figure 18. Differences on flow volumes in the 1D network for design storms events, Cranbrook case study.

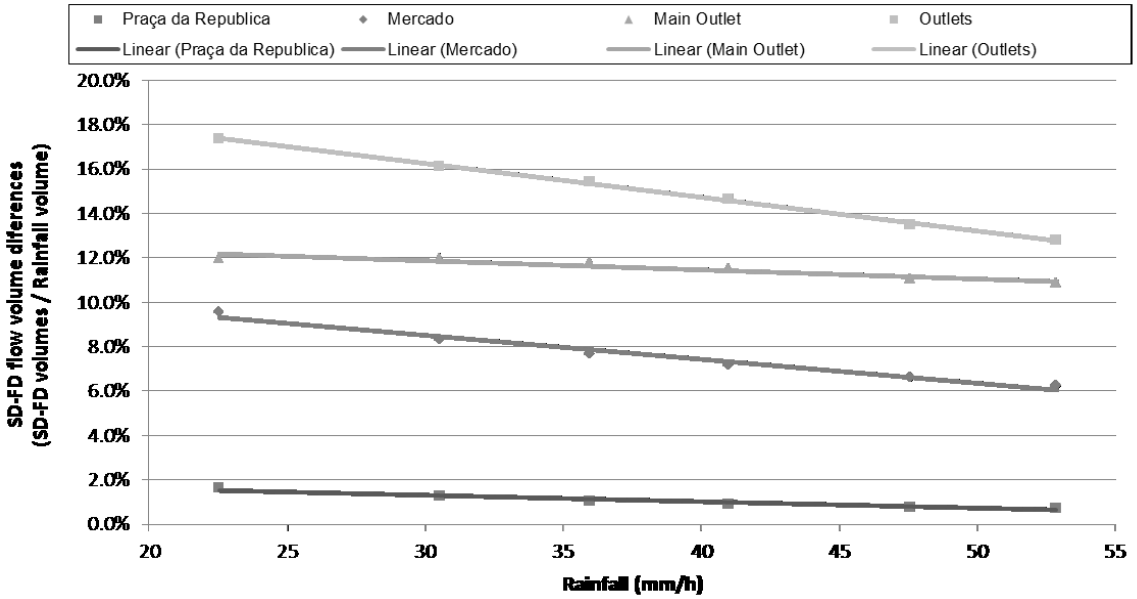


Figure 19. Differences on flow volumes in the 1D network for design storms events, Zona Central case study.

3 1D-2D interactions with stochastic sewer inlets' capacity

This Chapter presents the analysis of the interactions between 2D and 1D modules. The model of Zona Central case study (Coimbra, Portugal) was also used for this analysis and the interaction between 1D and 2D modules was analysed considering the variable capacity of sewer inlets in space and in time.

This Chapter is organised as follows: Section 3.1 describes the methodology with details about the geo-spatial and stochastic methods employed; Section 3.2 presents the case study with the model and dataset used to test the proposed methodology; Section 3.3 discuss the results of the testing.

3.1 Methodology

The general methodology was based on the following steps described in the following sub-sections:

A. Identification of sewer inlets prone to clogging;
B. Monte Carlo simulation of the capacity of inlets prone to clogging and subsequent simulation of flooding for each scenario;
C. Delineation of flood hazard maps.

3.1.1 Identification of sewer inlets prone to clogging

The clogging patterns of sewer inlets can be associated to debris mobilised by runoff and are usually dependent on road slope and on the position of inlets in the catchment (Gómez et al., 2013). The sewer inlets most prone to clogging are generally those near trees or located in terrain depressions. Considering this, the selection of sewer inlets prone to clogging was done based on the identification of roads with trees alongside and of terrain depressions. Roads with trees alongside were identified in this study based upon visual analysis of orthophotos covering the case-study catchment. The identification of terrain depressions was done based upon geospatial analysis of the high resolution digital elevation model (DEM) of the study area. For this purpose the Automatic Overland Flow Delineation (AOFD) tool (Maksimović et al., 2009) was used. The sewer inlets located in the identified streets with trees and/or in terrain depressions were considered prone to clogging. Ideally, information about the location of all inlet structures should be used as input for this analysis. However, when information at this level is not available, as is the case of the present study, manhole location can be used as a proxy to sewer inlets.

3.1.2 Stochastic representation of sewer inlet capacity

3.1.2.1 Modelling sewer inlet capacity

There are several types of sewer inlets: grate inlets; curb-opening inlets, and combination inlets have a grate and a lateral curb-opening. Each of these types has different inlet capacity that can be defined according to its dimensions, intersected flow and connections to the sewer network. There are several guidelines that define their capacity and the most popular are those by the US Federal Highway Administration – HEC-22 (Brown et al., 2009), which formulations are implemented in most commonly used urban drainage software. In the case of Portugal, the Direcção Geral de Recursos Naturais (National Directorate of Natural Resources; DGRN) published national guidelines for the design and construction of gullies (DGRN, 1991), which leads to similar results to those of the American guidelines (Sá Marques et al., 2013). Besides these guidelines, several laboratory and numerical studies have been conducted in recent years which have further investigated the discharge capacity of different inlet structures (e.g. (Ally, 2011; Almedeij et al., 2006; Gómez et al., 2013, 2011; Guo, 2006). In general, sewer inlets can behave as a weir until a certain water depth limit, from which they can be considered an orifice. Their maximum limiting discharge depends on the opening's geometry and dimensions that define the weir and orifice equations, and on the capacity of the connection to the sewer system. In the case of combined systems, inlets are installed with traps to stop releasing odours which hardly limit the inlet capacity. For typical UK inlet structures with traps, (Ally, 2011) proposes a

discharge limit of 10 $l \cdot s^{-1}$; in a field study conducted by Pina et al. (2010) in Coimbra (Portugal), a maximum discharge limit of 20 $l\ s^{-1}$ is proposed to common inlet structures found in Portugal.

To represent the sewer inflow process more realistically, the representation of sewer inlets for SD models presented in Section 2.1.3 was considered for this analysis. The above mentioned concept for accounting for the sewer inlet capacity makes use of a virtual node in the discharge point of each subcatchment. This node acts as intermediate connection between the subcatchment outlet node, a node of the sewer system and the 2D mesh of the surface. Each virtual node has infinitesimal volume and has the following hydraulic connections: a link that discharges to a network node according to a weir and orifice definition and a maximum limit that represents the capacity of the sewer inlet; and another link with a flap valve in the opposite direction of the weir and orifice link that allows runoff to flow from the sewers onto the 2D surface model once sewer surcharge occurs. In this way, whenever the capacity of the sewer inlet is exceeded, runoff discharged from subcatchments or on the 2D mesh remains in the overland surface and can directly flow into the 2D overland model, thus adequately representing flooding due to insufficient inlet capacity. The link with the weir and orifice is defined by an orifice element, which works as a weir until water depth exceeds the orifice diameter, and as an orifice when water depth exceeds the orifice diameter.

Based on previous studies carried out in the case study catchment (Pina, et al., 2010), the maximum capacity for the sewer inlet was validated by comparing flooded areas generated with the hydraulic model against photographic records for the largest flooding event recorded in *Praça 8 de Maio* (09 June 2006). Since the exact location and type (e.g., different size, inlet grate, with or without water seal trap, etc.) of the sewer inlets is usually not available, sewer inlets are aggregated and represented in the model network nodes. As a simplification, each node considers an average number of inlets, with the maximum limit of inlets capacity defined after calibration. Several software, including Infoworks ICM (Innovyze, 2013), define the sewer inlet capacity as a tabular relation between water depth and inlet flow (an adaptation of weir and orifice equations), thus defining a maximum inlet flow based on the water depth in each node; these values can be changed to mimic sewer inlet clogging conditions, and were used in the present study.

3.1.2.2 Stochastic representation of clogging conditions

The mathematical representations of sewer inlets presented in the last Section are limited to optimal operational conditions (i.e. fully clean inlets). However, blockage of inlet structures by accumulation of debris can occur depending on the type and location of inlets, calendar season (e.g. higher tree leaf fall-rate in late summer and autumn), antecedent weather conditions (e.g. higher transport of debris with first flush), and maintenance regimes. The accumulation of debris and subsequent blockage as presented in Figure 20, can lead to further reduction of their maximum discharge. However, this limit is seldom considered in urban drainage models. Recent studies (e.g., (Russo et al., 2015)) considered clogging factors to reduce inlet capacity on a 1D/2D urban drainage model; however, they did not consider the clogging variability as suggested in the present thesis. This is partly because currently available urban drainage software tools provide little flexibility for representing the different flow conditions that occur at inlets. To overcome this problem, a novel methodology was developed and is presented in this Section.

(a) Clean inlet grate (b) Partially blocked inlet grate (after rainfall event) (c) Fully blocked inlet grate (after rainfall event)

Figure 20. Sewer inlet cleaning conditions. Images (b) and (c) were taken after a storm event on 15 April 2015 which caused flooding in the Praça 8 de Maio (Coimbra, Portugal).

One way to study the impact of model parameters on model outputs is through Monte Carlo simulations in which the given parameter is assigned different values, randomly sampled from a probability distribution. In many cases, this requires a few hundreds or thousands of simulations. In order to study the effect of the hydraulic capacity of the sewer inlets on urban pluvial flooding, the level of inlet blockage was allowed to vary stochastically between 0% and 100% using the Beta-PERT probability distribution. This distribution is a useful tool for modelling expert data and is frequently used for the construction of an approximate probability distribution representing the outcome of future events, based on very limited information (and Rahimifard, 2009; Lake et al., 2010). It is a three-point estimation technique that is easy to define and based on prior experience or best-guesses, such as the minimum (or best-case) estimate (*a*), the maximum (or worst-case) estimate (*b*), and the mode (or most likely) estimate (*m*). Unlike the uniform distribution, three-point distributions, like the Beta-PERT, emphasise the most likely value. In practice, this means that we "trust" the estimate for the most likely value, and we believe that even if it is not exactly accurate, as estimates seldom are, we have an expectation that the resulting value will be close to that estimate. Another significant difference from the normal distribution is that events that are far from the most likely value (mode), have larger probability than they would have under a normal distribution (Hubbard, D.W., 2009); this is an important feature that should be considered for stochastic (flood) risk assessment (as can be seen in Figure 21.a). The Beta-PERT probability distribution has been applied in various risk probability assessment studies in different fields; these include, for example: infrastructure design and decision-making (Salling and Leleur, 2012; Thorhallsson and Sveinbjornsson, 2012), software project management (Barros et al., 2000), food quality control and risk analysis (Straver et al., 2007; Lake et al. 2010) and pollution risk assessment (Carroll and Harms, 1999; Jing et al., 2013).

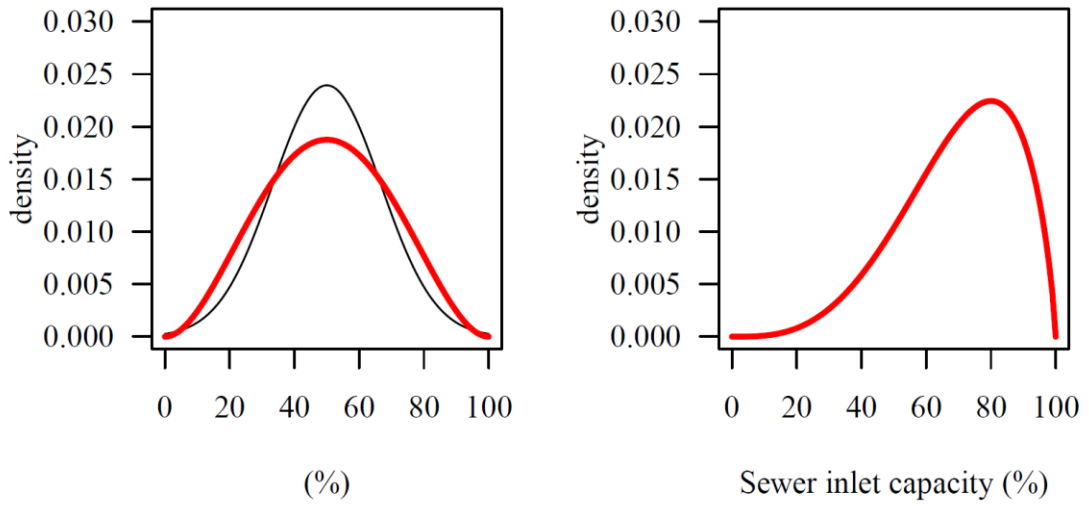
The Beta-PERT distribution is a special case of the Beta distribution. The probability density function of the Beta-PERT distribution is calculated using Equation 1.

$$f(x) = \frac{1}{\mathrm{B}(\alpha_1,\alpha_2)} \frac{(x-a)^{\alpha_1-1}(b-x)^{\alpha_2-1}}{(b-a)^{\alpha_1+\alpha_2-1}} \quad (1)$$

where *a* is the minimum, *b* is the maximum and *m* is the mode parameter; B is the Beta function, $\alpha_1 = \frac{4m+b-5a}{b-a}$ and $\alpha_2 = \frac{5b-a-4m}{b-a}$. For more details about the Beta-PERT distribution the reader may refer to Vose (2000).

Figure 21.b presents the negatively skewed probability density function of the Beta-PERT distribution adopted in this study; the minimum, maximum and most likely (mode) parameters adopted were, respectively, 0%, 100% and 80% of the maximum inlet discharge. These parameters were chosen based

upon the experience of the company in charge of the drainage system of the pilot catchment, which are in agreement with the reduction factors suggested by several authors (Guo, 2006; Almedeij et al., 2006; Gómez et al., 2013) to account for inlet clogging conditions. The hydraulic capacity can vary between 0 and 100% of the theoretical value ($Q = \beta\, f_{(h)}$, where Q is the sewer inlet hydraulic capacity, β varies between 0 and 100% with 0% meaning a fully blocked inlet and 100% a blockage free inlet, and $f_{(h)}$ is a function of the water level and the hydraulic capacity of the inlet) and, in most of the cases, sewer inlets are assumed to be relatively clean. The inlets prone to clogging are considered to have 80% of their full hydraulic capacity during most of the time, as the performance of annual cleaning process of the inlets is not 100% effective and the accumulation of debris can make the clogging process more acute (Gómez et al., 2013).



(a) Normal (black line) and PERT (red line) distributions with the same mean and standard deviation: μ=50%; σ=16.667)

(b) PERT distribution used in this study with parameters min: 0%; max: 100% and mode: 80%

Figure 21. Probability density functions

3.1.3 Delineation of stochastic flood hazard maps

For each hydraulic model used in the Monte Carlo simulation, the inlets identified as prone to clogging were assigned a hydraulic capacity factor, sampled from the Beta-PERT distribution. Hydraulic simulations for each of the resulting urban drainage models, i.e., for different sewer inlet hydraulic conditions, were conducted. After the convergence of the Monte Carlo simulation, the entire set of runs is analysed to assign a flooding probability to each element of the 2D overland mesh. The output is a more comprehensive map based on flooding probability rather than a deterministic floodplain.

3.2 Pilot catchment, datasets and model

The methodology proposed in this Section was demonstrated using as pilot location the *Zona Central* catchment, located in Coimbra (Portugal), and presented in Section 2.2.2 with the SD 1D2D model implemented with the concept presented in Section 2.1.

The importance of sewer inlet capacity and cleaning conditions in the occurrence of pluvial flooding in the *Zona Central* catchment has been discussed in previous studies (Pina *et al.*, 2010, Pina *et al.*, 2014). According to these studies and existing records (see Table 4), some of the rainfall events that have caused flooding often have a relatively small return period ($\lesssim$ 5 years) and during these events no surcharge in the upstream or downstream sewer pipes, nor backwater effects were reported. In contrast, other storm events with larger return period (~ 10 years) have not resulted in flooding. One potential explanation for this could be the spatial variability of rainfall fields (Segond *et al.*, 2007; Leitão *et al.*, 2009; Gires *et al.*, 2012; Simões *et al.*, 2015); however, given the small size of the pilot catchment, the impact of spatial rainfall variability is likely

to be small. Thus, the source of flooding must be explained by other factors than the precipitation. *In situ* observations suggest that the limited hydraulic capacity of sewer inlets may be a key factor which exacerbates flooding in this area.

Over the catchment, and especially in the area upstream of the *Praça 8 de Maio*, there is a significant number of deciduous trees planted along the streets. The water utility responsible for the operation and maintenance of the drainage system, *AC, Águas de Coimbra, E.M.*, (AC) has a maintenance plan for the sewer network of this catchment, which includes cleaning the inlets four to five times per month. The cleaning procedure is focused on removing solids from the inlets' sandbox and replacing the water in inlet traps, in order to ensure that sewer inlets have adequate hydraulic capacity and that odours are not released from the combined sewer system. However, during heavy rainfall events debris is dragged along the roads and accumulates on top of sewer inlets, blocking these structures and potentially leading to localised flooding in subsequent storm events.

Table 4. Rainfall events and flooding in *Praça 8 de Maio*

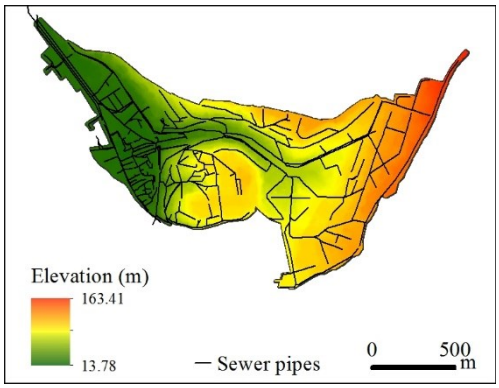
Date	Flooding in Praça 8 de Maio?	Return period of the storm event		
		30 mins duration	45 mins duration	60 mins duration
09/06/2006	Yes	100	100	50
25/10/2006	Yes	50	50	50
20/04/2007*	**No**	**5**	**2**	**2**
21/09/2008*	**Yes**	**5**	**5**	**5**
29/04/2011	No	10	10	5
24/12/2013	Yes	5	5	5

*The events in bold are the ones used for testing in the present study.

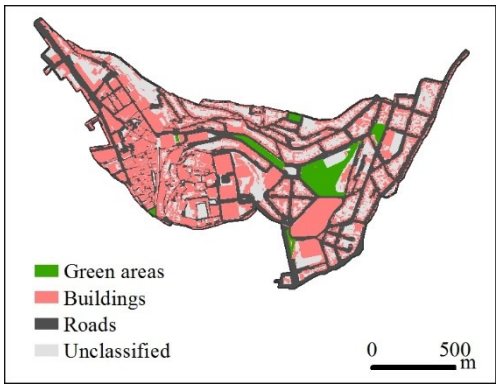
3.2.1 Geo-data used for model building and identification of inlets prone to clogging

The following geo-datasets were used for the implementation of the urban drainage model of the Zona Central catchment, as well as for the identification of inlets prone to clogging, following the methodology described in Section 3.1:

- *Orthophotos* with a pixel resolution of 0.5 m (used to identify roads with trees);
- *LiDAR-generated Digital Elevation Model (DEM)* with 1 m horizontal resolution (used to identify terrain depressions) (Figure 22a);
- *Land use* and *buildings data* in vector format (Figure 22b);
- *Sewer network topology and geometry information* in vector format, including location of manholes (used to identify clogging prone inlets and set up the hydraulic model) (Figure 22a).



(a) DEM and sewer system datasets



(b) Land use dataset

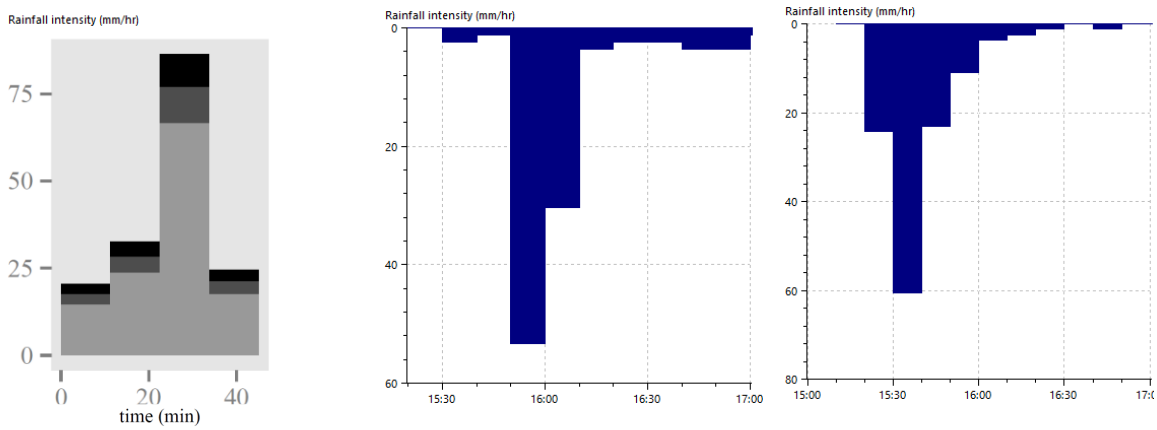
Figure 22. Data sets used in this study.

3.2.2 Storm events selected for analysis

Three design storms and two real rainfall events were used to test the proposed methodology (Figure 23).

The design storm events used in the analysis correspond to return periods of 5, 10 and 20 years. The Intensity-Duration-Frequency (IDF) curves for Coimbra, along with the alternate block method were used to generate the 5-, 10- and 20-year design rainfall hyetographs with a duration of 45 min (~the *Zona Central* catchment concentration time) (Figure 23.a).

The recorded storm events were those of 20 April 2007 and 21 September 2008 (Table 4; Figure 23.b and Figure 23.c). Both events had a similar return period (~5 years); however, only the 2008 event caused flooding in *Praça 8 de Maio*. During these events rainfall was recorded using one tipping-bucket rain gauge located near the catchment. This gauge is operated and maintained by the water utility "Aguas de Coimbra" (AC). It continuously records rainfall data with a temporal resolution of 10 minutes.



(a) design storms (black: T=20 years; dark grey: T=10 years; light grey: T=5 years)

(b) storm event of 20 Apr 2007

(c) storm event of 21 Sep 2008

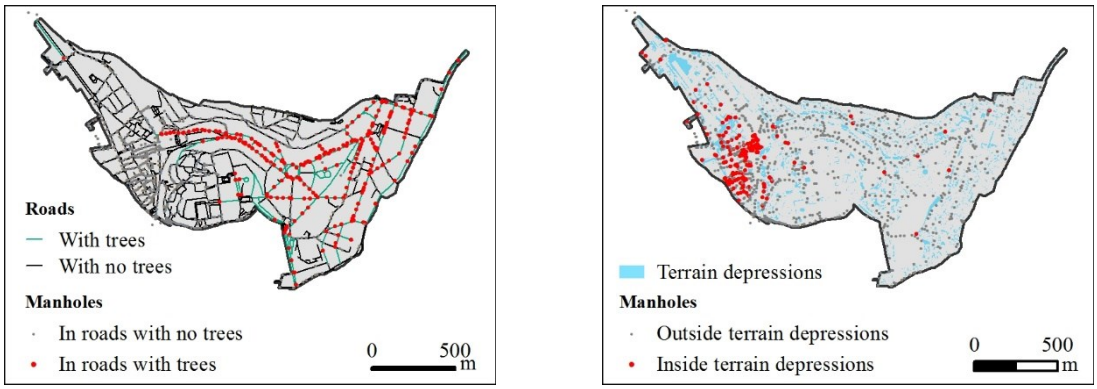
Figure 23. Hyetographs of storm events.

3.3 Results and discussion

The analyses presented were performed with the SD 1D2D model of the *Zona Central* catchment described in Section 2.2.2.

3.3.1 Identification of sewer inlets prone to clogging

As described in Section 3.1, inlets prone to clogging were identified based upon their relative location to trees and/or terrain depressions. Inlets located in roads with trees are shown in Figure 24a and inlets located in terrain depressions are shown in Figure 24b.



(a) manholes located in roads with trees

(b) manholes located in terrain depressions

Figure 24. Identification of sewer inlets prone to clogging.

3.3.2 Convergence of the Monte Carlo analysis

The convergence of the Monte Carlo method was validated based upon simulations conducted with the 20-year return period storm as input. For each of the runs, sewer inlet orifice parameters, for clogging-prone inlets, were sampled from the Beta-PERT distribution, as described in Section 3.1.2.

A water depth threshold of 0.1 m over the elements of the 2D overland model was adopted to determine flooding occurrence. Based upon this, the following flooding indicators, which are directly related to the aim of the proposed methodology, were estimated:

- number of flooded 2D mesh elements;
- total flooded area, estimated as the sum of the individual area of each of the flooded 2D elements ($A_{2D\ element}$);
- total flood volume, estimated as the sum of the maximum flood volume at each flooded 2D element. The individual flood volume of a 2D element ($V_{2D\ element}$) is estimated as $V_{2D\ element} = A_{2D\ element} \times D_{2D\ element}$, where $D_{2D\ element}$ is the maximum water depth over a given 2D element, for each simulation.

As expected, as the number of model runs increases, variations in the distribution of the outputs (determined based upon the above mentioned indicators) are likely to reduce, until convergence is achieved. From the performed convergence analysis, 200 runs produce variations (errors) of less than 0.001%, which was used as the acceptance (convergence) criterion for the minimum number of Monte Carlo simulation runs. Based upon this, it was decided to conduct 200 simulations for all Monte Carlo analyses carried out in this study. Each simulation took approximately two minutes to run (136 s for the case of 20-year return period storm, using a computer equipped with a Intel® Core™ i7-3770 (3.40 GHz) processor unit; 16 GB RAM and a 64 bits Windows 7); simulation times showed some variation due to the different model conditions, e.g., sewer inlet capacity and storm events.

3.3.3 Stochastic flood hydrographs and hazard maps

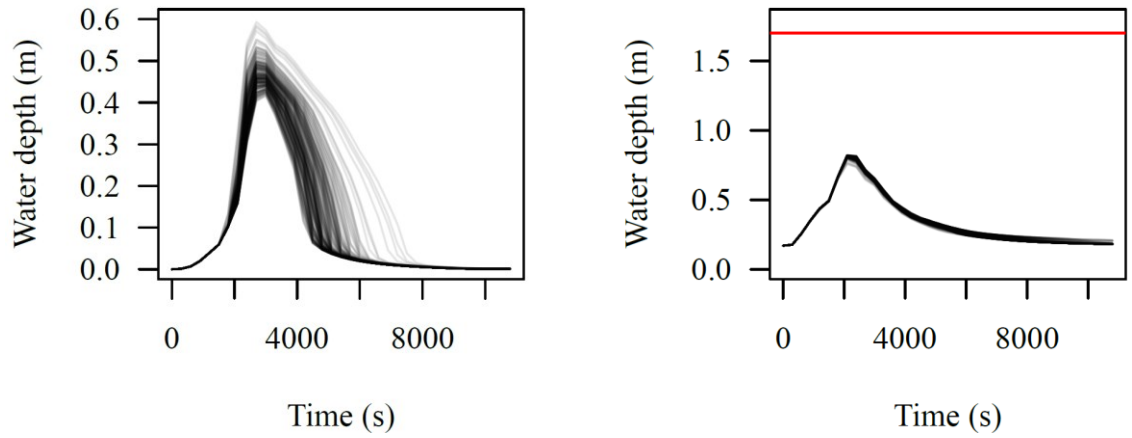
In this Section, the results of the stochastic analysis, including stochastic flood hydrographs and hazard maps are presented. In order to highlight the benefits of accounting for variations in sewer inlet capacity, the results of the stochastic analysis are compared against deterministic flood simulations. The deterministic simulations consider the inlets prone to clogging with 80% of the theoretical hydraulic capacity. This value was adopted as it was considered the most likely value in the Beta-PERT distribution adopted in this study to represent variations in sewer inlet capacity. The stochastic simulations comprise a set of 200 runs per storm event, each run with variable inlet capacity sampled from the Beta-PERT distribution (Section 3.1.2.2). The deterministic flood hazard maps correspond to the unique floodplain resulting by running the hydraulic model once (with fixed inlet capacity) for a given rainfall input. The stochastic flood maps, on the other hand, display the probability of a given 2D element being flooded; this is estimated as the ratio of the number of times a 2D element is flooded, in relation to the total number of simulations under consideration (i.e., 200).

In what follows the results for the design storms are first presented; these are used to examine the sensitivity of the catchment to sewer inlet capacity, as well as to better understand flooding mechanisms in the area. Afterwards, the results of the real (observed) storm events are presented; these are used to further prove the importance of accounting for sewer inlet capacity in urban drainage models.

3.3.3.1 Design storms

All simulations resulting from the 20-year return period design rainfall, with variation in the hydraulic capacity of clogging-prone inlets, showed flooding in *Praça 8 de Maio* (Figure 25a). As can be seen, when the hydraulic capacity of the inlets is limited, more water stays onto the surface, increasing the water depth

over the 2D model and causing delays in the entrance of runoff into the sewer system. Despite the visible variations, most of the water depth results were within a 0.05 m range (notice interquartile range between approximately 0.45 and 0.5 m). The time to water depth = 0 m also showed a significant variation (more than 3,600 s), however most of events vary within a range of 10 minutes (600 s).



(a) Water depth over the most critical element of the 2D mesh at *Praça 8 de Maio*

(b) Water depth in the sewer pipe located upstream of *Praça 8 de Maio* (red line represents the maximum pipe depth)

Figure 25. Hydrographs for different sewer inlet hydraulic capacities

As a consequence of the variation of water entering into the sewer system, the flow in the pipes also varies. Figure 25b shows the flow in the sewer pipe upstream *Praça 8 de Maio*. Besides the variation of flow due to variations in inlet capacity, this figure also shows that the maximum water depth of the sewer (1.7 m) was never achieved. This clearly indicates that the flooding problem in this catchment, and in particular in *Praça 8 de Maio*, is related to the limited hydraulic capacity of inlets and consequent inability of surface water to enter to the sewer pipe system.

Figure 26 and Figure 27 present the stochastic and deterministic flood extent for the whole catchment using the design rainfall with a return period of 20 years. As can be seen, some areas which are identified as prone to flooding in the stochastic map (Figure 26) are not identified as such in the deterministic map (Figure 27).

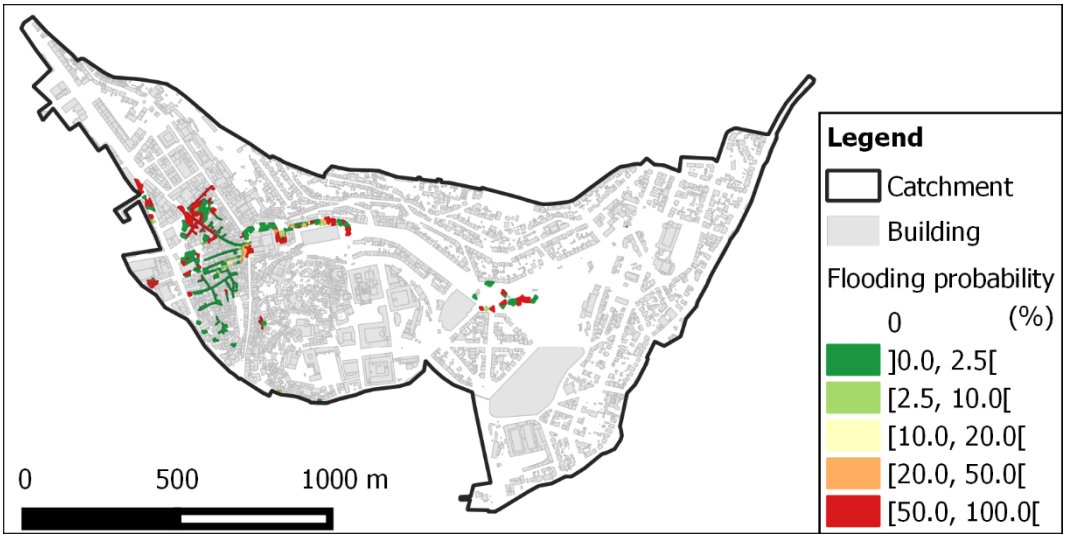


Figure 26. Flood extent for a stochastic 20 years return period design storm simulation.

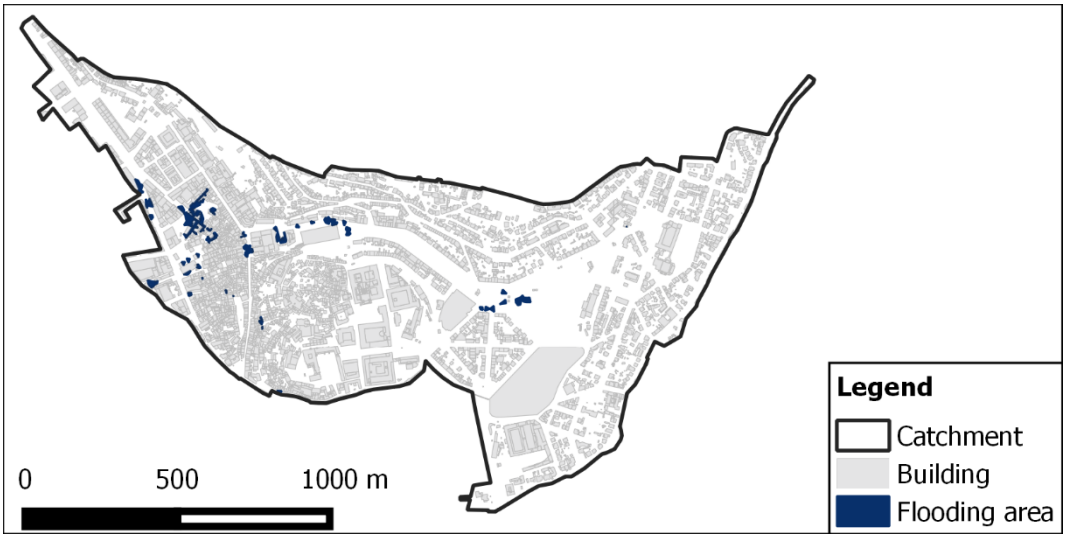


Figure 27. Flood extent for a deterministic 20 years return period design storm simulation.

Uncertainty associated to rainfall return period was also investigated with stochastic simulations using design storms with different return periods. Figure 28 shows the water depth variation for the stochastic simulations for three design storms with different return periods. It can be seen that, despite the increase of water depth with return period (as expected), the variation range reduces with increasing return period - this suggests that the uncertainty also decreases. Another interesting aspect that can also be seen in Figure 28 is that the highest water depth values obtained using a rainfall with low return period can be higher than the lowest water depth values obtained using higher return period rainfall. This highlights the importance of correct estimation of the sewer inlet capacity and the importance of the developed stochastic approach presented.

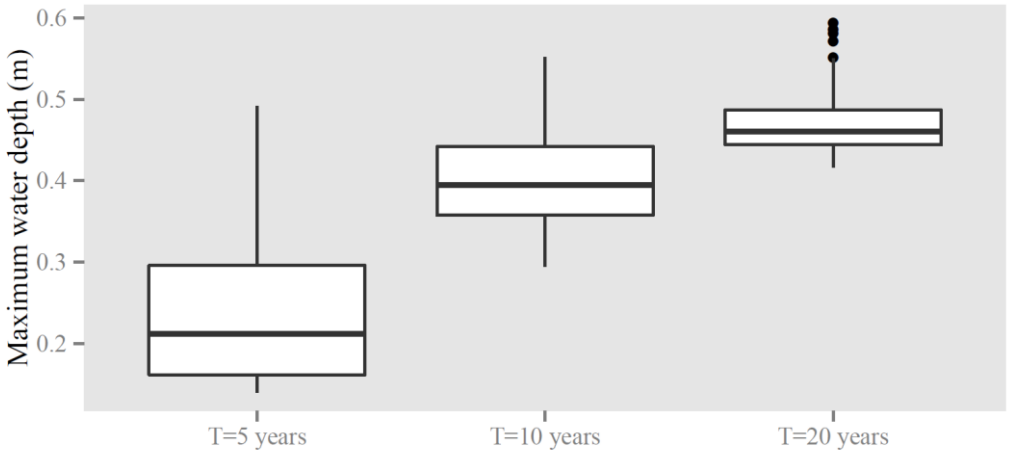
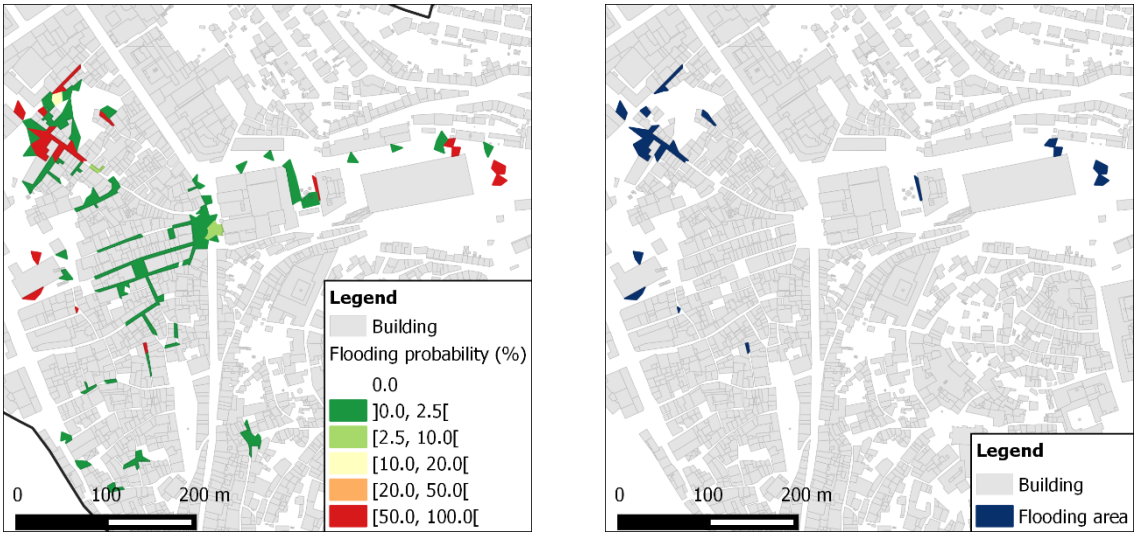


Figure 28. Water depth over the most critical element of the 2D mesh at *Praça 8 de Maio* - variation for rainfall events with different return periods (T).

3.3.3.2 Observed storm events

The rainfall event of 20 April 2007 has a return period that varies from 5 years in 30 minutes to 2 years in 45 minutes. It did not cause flooding in *Praça 8 de Maio* and this can be seen by the modelling results presented in Figure 14. The results obtained with the stochastic simulation show a small likelihood of flooding of less than 10% (Figure 29a), while the deterministic simulation results show no flooding in this area (Figure 29b). In fact, in 2007 no flooding was reported in *Praça 8 de Maio*.



(a) (b)

Figure 29. a) Detail of the flood extent for a stochastic simulation of the 20 April 2007 event; (b) Detail of the flood extent for a deterministic simulation of the 20 April 2007 event.

The rainfall event of 21 September 2008 has a return period of five years for durations of 30 minutes and 45 minutes. This rainfall event is similar to that of 20 April 2007 (see Figure 23). Photographic records provided by AC illustrate the flood occurred and debris accumulated and clogging sewer inlets in *Praça 8 de Maio* (Figure 30). This result is captured by the stochastic simulations which attribute a flood probability higher than 50% for this location (Figure 31a), whereas the deterministic simulations show no flooding at *Praça 8 de Maio* (Figure 31b).

Figure 30. Flooding in *Praça 8 de Maio* caused by the rainfall event of 21 September 2008 (source: *Diário de Coimbra*).

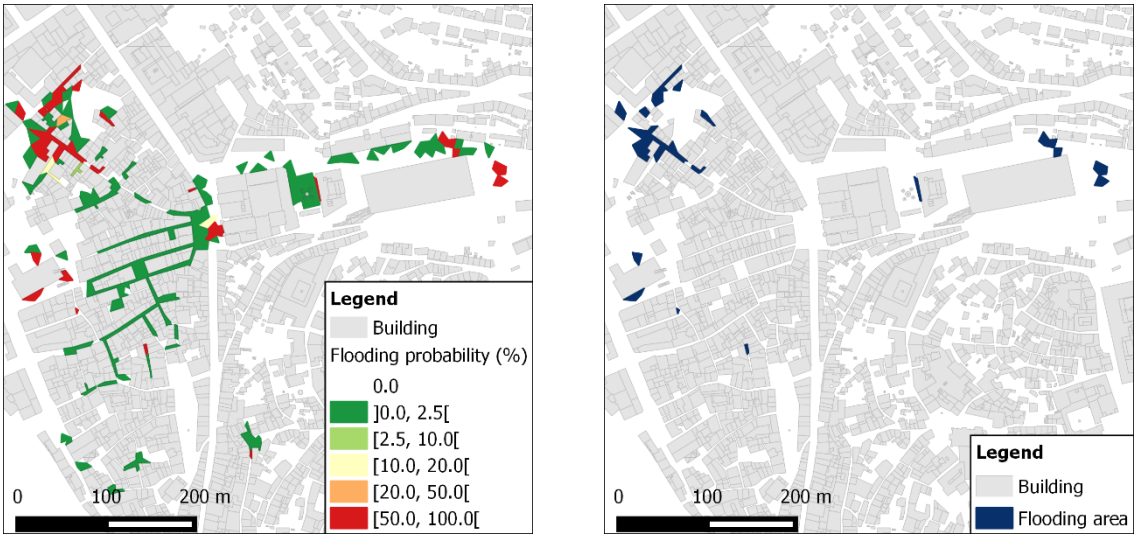


Figure 31. (a) Detail of the flood extent for a stochastic simulation of the 21 September 2008 event; (b) Detail of the flood extent for a deterministic simulation of the 21 September 2008 event.

4 Quasi-2D: solving 2D with 1D models

The Quasi-2D is a new concept developed by the author of this Thesis to simulate runoff along 2D continuous surfaces with 1D Saint-Venant equations. The mathematical formulation of this concept was demonstrated for the first time by the author of this Thesis, who also developed a plugin in QGIS to automatically generate 1D SWWM models with the Quais-2D concept from a digital elevation model (Figure 32).

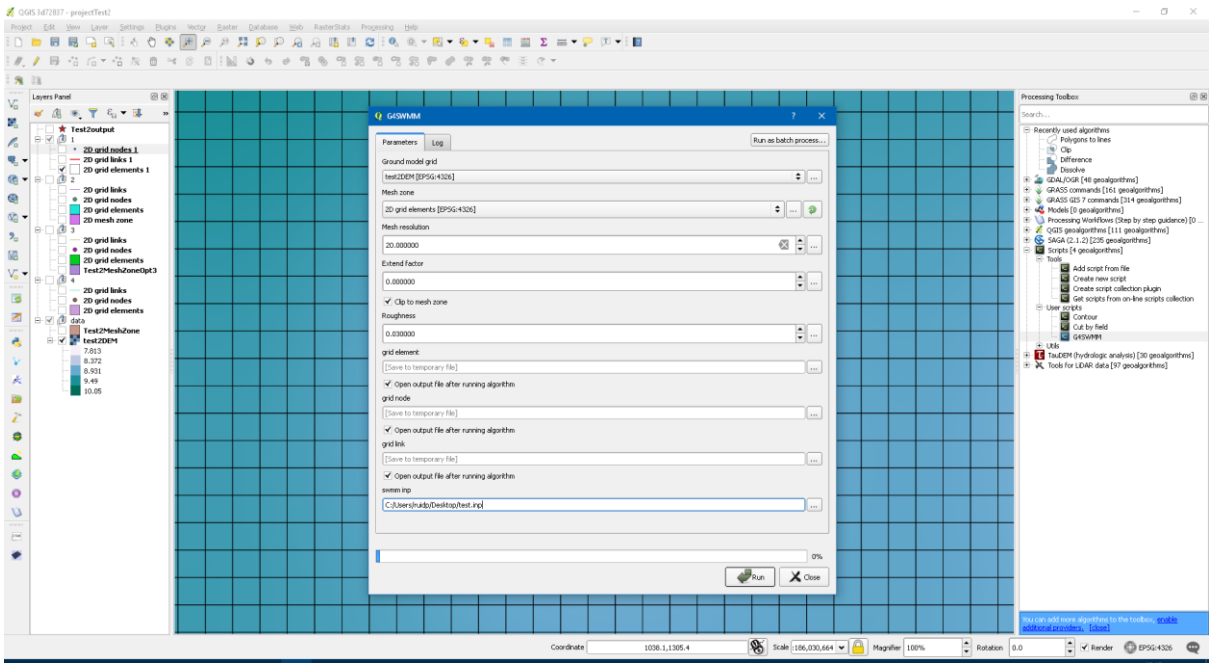


Figure 32. Plugin developed in QGIS with the Quasi-2D concept.

The results generated with SWMM models developed with the Quasi-2D concept were compared against full dynamics 2D models implemented in MOHID (Mohid wiki, 2014). The dataset selected for this comparison was two tests presented by S. Néelz and G. Pender (2013) who did a benchmark of available 2D models on the market.

This Chapter is organized in the formulation of the *Quasi-2D in* Section 4.1 followed by a summary of the tests conducted with benchmarking data sets on Section 4.2.

4.1 Methodology: *Quasi-2D* concept

The *Quasi-2D* concept is based on the representation of surfaces in a grid of rectangular elements. Each rectangular element is represented in 1D scheme with a 1D node connected to adjacent elements by a rectangular grid of 1D links (Figure 33). The nodes have infinitesimal areas and elevation corresponding to the rectangular elements. The links are defined by rectangular cross sections without sidewalls, and their width and length defined by the area covered by rectangular elements and the continuity equation applied with the Gauckler-Manning Strickler equation to reproduce friction losses.

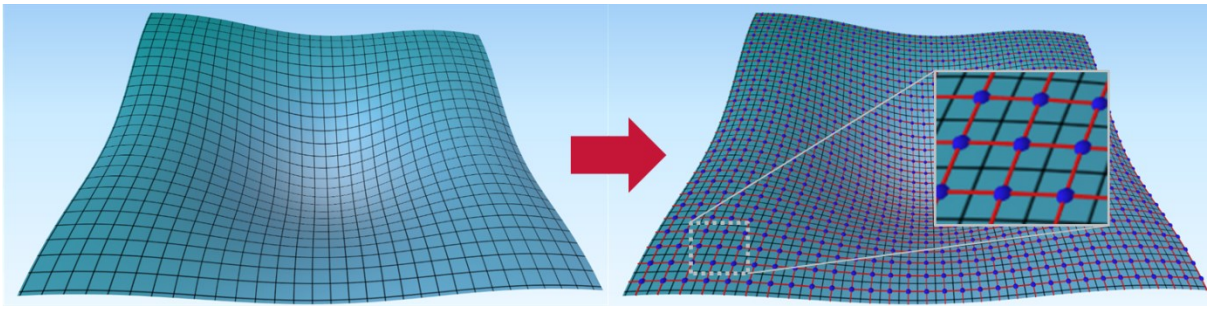


Figure 33. *Quasi-2D* concept with 2D rectangular grid and 1D nodes (blue) and links (red).

The mathematical equations to define the width and length of links can be formulated assuming the equilibrium of flow in the horizontal direction "x" of a channel. The Equation 2 defines the total flow in the "x" direction of the channel must be equal to the sum of the flows in each element in the same direction. The Equation 3 divides the area of each element by half of their four links. The mass conservation equation (2) can be generalised with the Gauckler-Manning Strickler equation for 1D models that use it to define head losses, such as SWMM. This defines the *Quasi-2D* Equations 4 and 5 for the width and length of 1D links.

$$\sum_{i=0}^{j} q_i = Q \quad (2)$$

$$A = 2 \times l \times b \quad (3)$$

$$b = 0.5^{1/3} \times r \quad (4)$$

$$l = 0.5^{2/3} \times r \quad (5)$$

where j is the number of elements in the perpendicular direction to "x" along the channel, q is the flow in each 1D link, Q is the flow in the channel in the x direction, A is the area of each surface element, l and b are the length and the width of each 1D link, respectively, and r the resolution of the surface elements.

An example of the *Quasi-2D* concept applied to rectangular elements with a resolution of 10 m is presented in Figure 34.

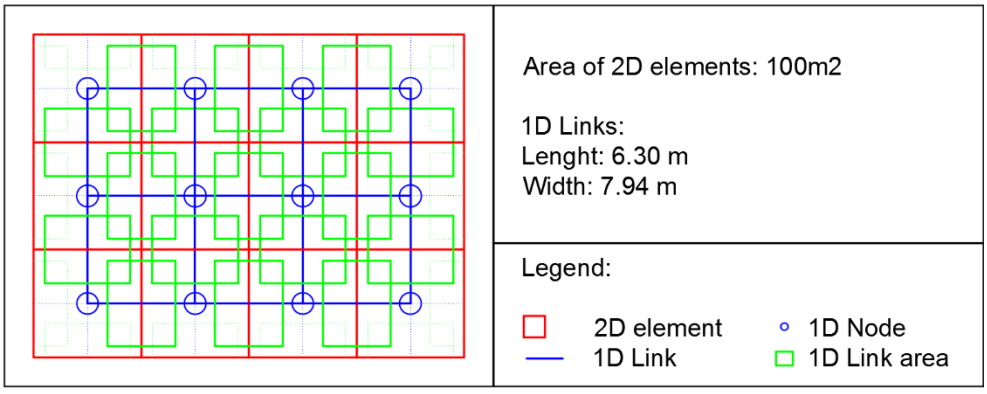


Figure 34. Example of a *Quasi-2D* model with SWMM.

4.2 Work packages

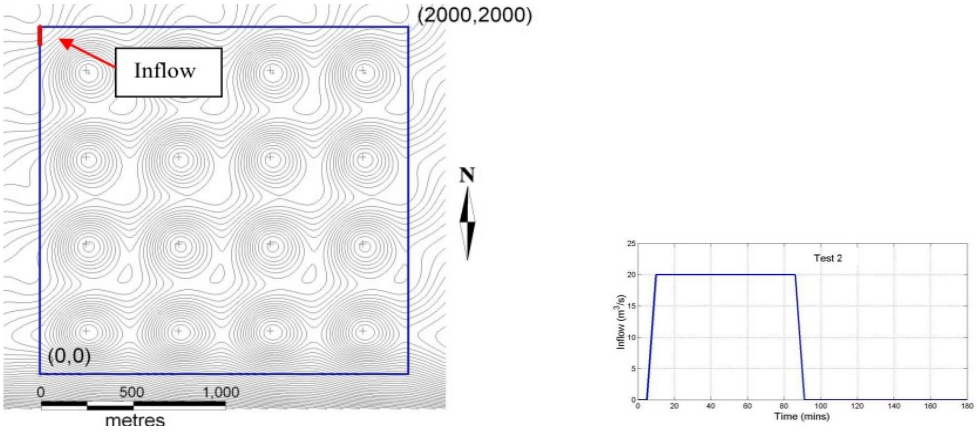
The *Quasi-2D* concept was tested using SWMM with the benchmark data sets presented by S. Néelz and G. Pender (2013). The results are compared with the most commonly used 2D software presented by those authors and with MOHID Land. The proposed Tests 2 and 8A are presented in this Section. MOHID Land solve the 2D dynamic wave equations with a numerical solver defined by an explicit finite-volumes scheme (Mohid wiki, 2014). SWMM is based on the 1D dynamic wave equations with an explicit numerical scheme (Rossman and Huber, 2016).

The results presented from MOHID and from SWMM (*Quasi-2D*) were conducted on the same machine with the flowing specifications: Intel® Core™ i7 CPU M 620 (2.67 GHz), 4 cores; 4.00 GB RAM; Windows 7 professional 64-bit. The tests with MOHID Land were implemented with MOHID Studio Professional version 2.0.0.1608 which is CPU parallelised (Braunschweig et al., 2012). The *Quasi-2D* concept was tested with SWMM 5.1.010 with 4 threads.

4.3 Results and discussion

4.3.1 Test 2

The objective of Test 2 is to assess the package's ability to handle disconnected water bodies, wetting and drying of floodplains, and to predict the inundation extent due to momentum flooding on a complex topography, with an emphasis on the final distribution of flood water rather than peak levels. The models are generated based on a ground model grid and the boundary condition is an inflow hydrograph, as presented in Figure 35.



a) Dgital elevation model (DEM) and location of the inflow boundary condition.

b) Boundary condition – inflow hydrograph.

Figure 35. Dataset and boundary conditions for Test 2 (adapted from S. Néelz and G. Pender (2013)).

The Table 5 summarizes the MOHID and *Quasi-2D* models and presents the total simulation times. The both models are similar and the minimum time step is in agreement, but the *Quasi-2D* requires more time to complete the simulation. The total volumes at the end of the simulation are very similar and correspond to relative increase of 0.13% to the inflow volume in MOHID and 0.19% in *Quasi-2D*.

Table 5. Summary of the Test 2 models, simulation times and final volumes.

Model	**MOHID**	***Quasi-2D***
Elements area (m2)	400	400
Elements (un.)	10000	10000
Time-stepping (s)	60	5
Minimum timestep (s)	3.06	4.5
Total simulation time (s)	107.72	821
Volume final (m3)	97330	97372

The results for Point 4 and 1 are presented in Figure 36 and Figure 37, which correspond to the left-top and left-bottom depressions. It can be verified that MOHID and *Quasi-2D* have a very good agreement and are very similar to the results achieved with other full dynamic models presented by S. Néelz and G. Pender (2013).

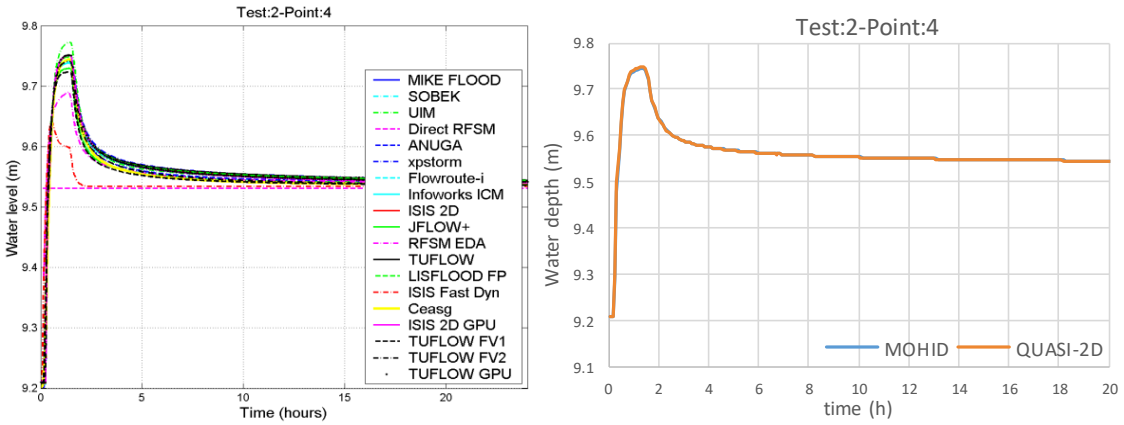


Figure 36. Results for Point 4 of Test 2 (left: from S. Néelz and G. Pender (2013), right: MOHID and *Quasi-2D*).

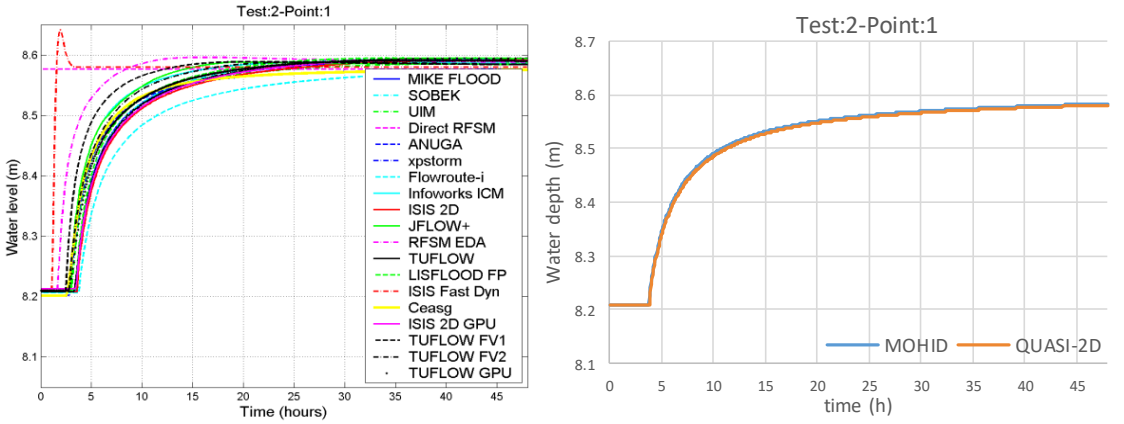


Figure 37. Results for Point 1 of Test 2 (left: from S. Néelz and G. Pender (2013), right: MOHID and *Quasi-2D*).

The floodplain generated at the end of the simulation is presented in Figure 38. The results presented by S. Néelz and G. Pender (2013) are the generated by full dynamic models and are in very close agreement with the results from MOHID and *Quasi-2D*.

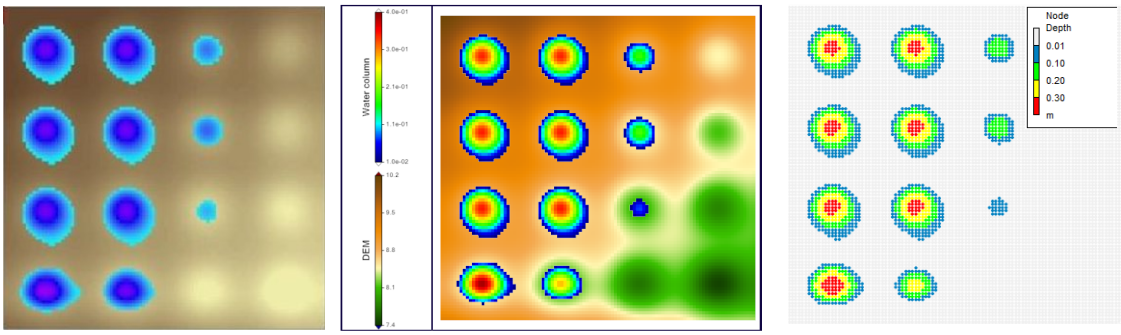


Figure 38. Flood map generated at the end of the simulation of Test 2 (from left to right: S. Néelz and G. Pender (2013) (with water ponds in blue), MOHID and *Quasi-2D*).

4.3.2 Test 8A

The objective of Test 8A is to access the package's capability to simulate shallow inundation originating from a point source and from rainfall applied directly to the model grid, at relatively high resolution. The data available represent a real urban model with a ground model grid and polygons with roads and pavements. The boundary conditions are a rainfall hyetograph and a point-source inflow, as shown in the next Figure 39.

Figure 39. Dataset and boundary conditions for Test 8A (details in the map: purple lines are the outline of roads and pavements; triangles are output point locations – adapted from S. Néelz and G. Pender (2013)).

The Table 6 summarizes the MOHID and *Quasi-2D* models and presents the total simulation times. The number of elements has minor differences, because *Quasi-2D* did not consider elements intersecting borders of the 2D zone. The high space resolution of this test leads to very small time steps in both models; however, the *Quasi-2D* model required significant more time than MOHID.

Table 6. Summary of the Test 8A models and simulation times.

Model	**MOHID**	***Quasi-2D***
Elements area (m2)	4	4
Elements (un.)	97000	95719
Time-stepping (s)	30	0.1
Minimum timestep (s)	0.01	0.1
Total simulation time (s)	3422.84	45952

The Figure 40, Figure 41 and Figure 42 present the comparison of the results of all models in locations where runoff accumulates (Points 1 and 3) and in main the pathway (Point 2) (these locations are labelled in the layout of Figure 39). The results have a very good agreement which demonstrates the ability of the *Quasi-2D* model to be applied in urban drainage. The minor differences between results obtained by MOHID and *Quasi-2D* are acceptable and inside the range of results presented by S. Néelz and G. Pender (2013) with full dynamic models. Moreover, these minor differences can be justified by slight deviations on elevation sampling by both models. The *Quasi-2D* model can be more unstable for small water depths, as verified in Point 2. This issue is related with the stability of SWMM and can be addressed by reducing the time step for these situations.

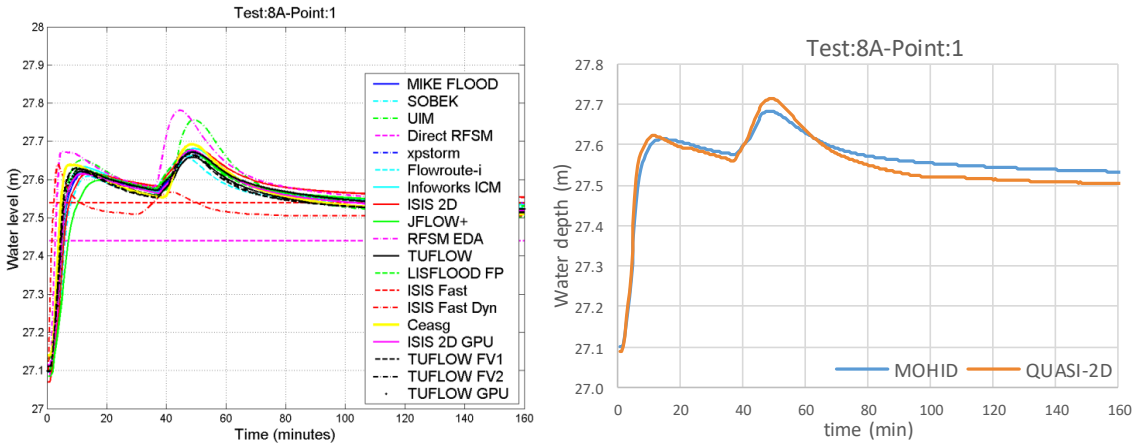


Figure 40. Results for Point 1 of Test 8A (left: from S. Néelz and G. Pender (2013), right: MOHID and *Quasi-2D*).

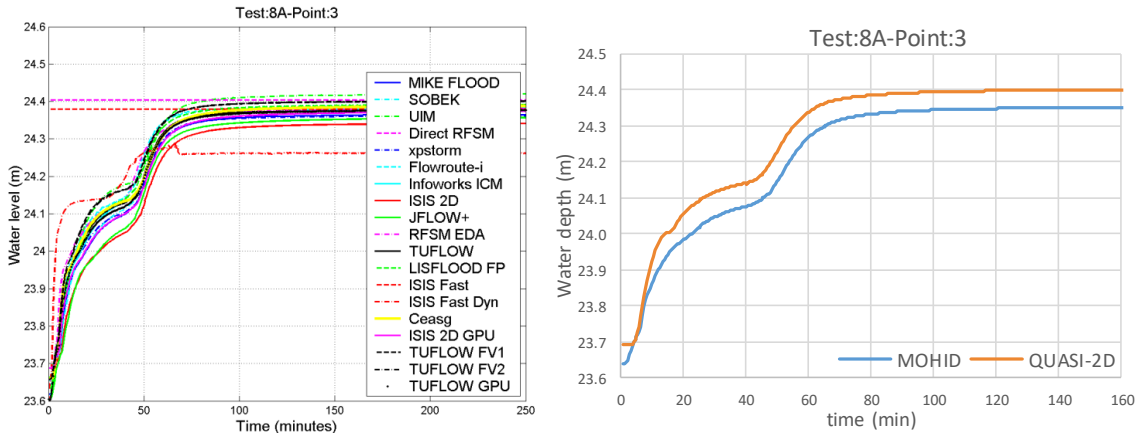


Figure 41. Results for Point 3 of Test 8A (left: from S. Néelz and G. Pender (2013), right: MOHID and Quasi-2D).

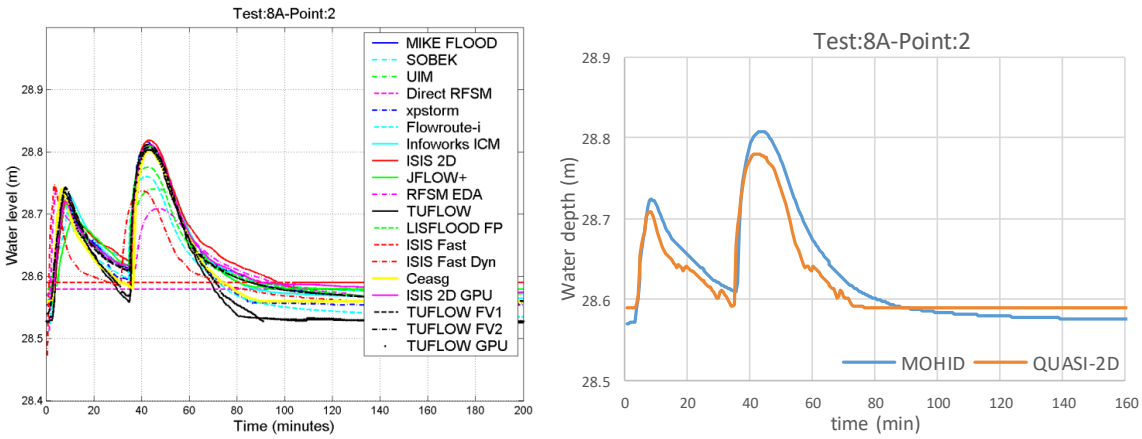


Figure 42. Results for Point 2 of Test 8A (left: from S. Néelz and G. Pender (2013), right: MOHID and *Quasi-2D*).

The next Figure 43 and Figure 44 shows the floodplain predicted by MOHID and *Quasi-2D* models. These results are equivalent for both models which demonstrates that the *Quasi-2D* model can predict flood extents in agreement with a 2D full dynamic model such as MOHID. The floodplain presented by S. Néelz and G. Pender (2013) for part of this area is also in agreement with these results.

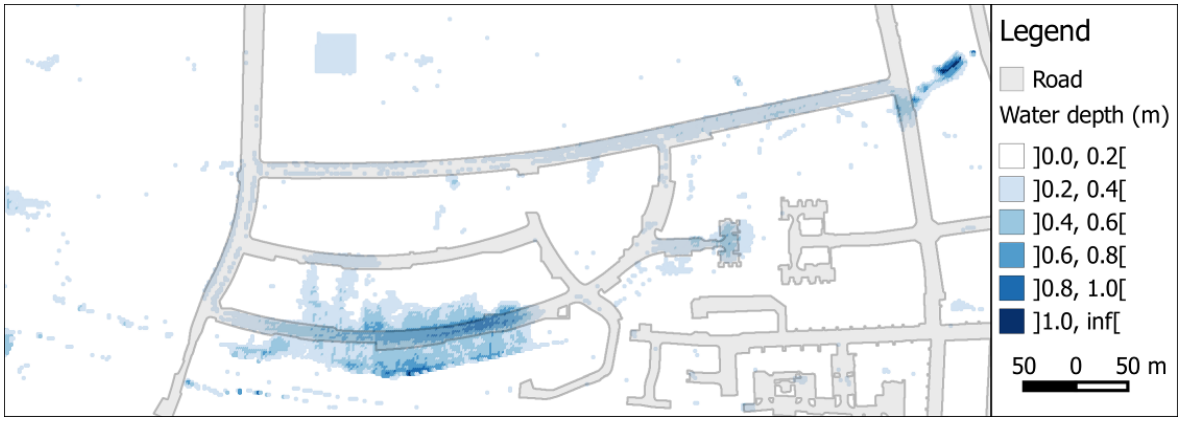


Figure 43. Floodplain predicted for Test 8A with MOHID.

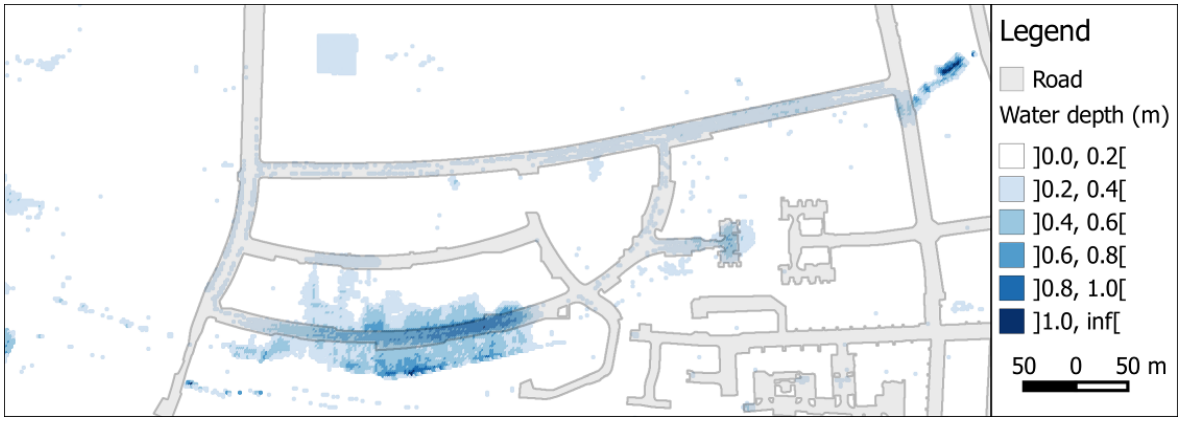


Figure 44. Floodplain predicted for Test 8A with *Quasi-2D*.

5 Conclusions and future work

The research presented in this thesis justified the hypothesis that "Integrated urban stormwater models improve the holistic of the urban water cycle" with the main aim of "improve urban stormwater modelling tools with more accurate representations of 2D overland modules and their interactions with 1D sewer flow modules". This aim was achieved with the detailed objectives which conclusions are presented in the next Sections with suggestions for future work.

5.1 Semi-distributed and fully distributed urban stormwater models

This work presented a comparison between SD and FD models using two real case studies with different characteristics and flooding mechanisms. Innovative concepts were proposed for the model building process and to establish the connections between the modules of SD and FD models.

FD models were generally found to inaccurately retain runoff volumes on the overland surface due to surface depressions, buildings singularities, and the lack of representation of private connections to the sewer network. This has not been observed in the SD model, since the runoff is directly discharged from subcatchments to network nodes. While surface depressions and building singularities are dependent on the resolution of the overland surface module, the lack of connection to the minor system relies on the resolution on the sewer flow module.

In the overland flow module, surface depressions are related with the surface overland definition and building singularities are dependent on the definition of building boundaries. In the Cranbrook case study, surface depressions are the main cause of retaining water on the overland surface and the differences between SD and FD models are reduced for high intensity rainfall events. In the Zona Central case study building singularities accumulate significant runoff volume, leading to significant differences between SD and FD models, even for high intensity rainfall events. This implies that FD models are likely to be inaccurate in highly urbanised areas with dense buildings zones characterised by several singularities and delimited private areas, which could retain runoff volumes.

The resolution of sewer network data defines the connections between the overland flow and sewer flow modules. In addition to the typically available data of the public sewer network, as used in the analysed case studies, FD models should also include information on private networks and connections that drain areas delimited by buildings. However, this data is difficult to obtain for most studies and can make the sewer flow module very complex. An alternative is to define the FD model only for open areas (without buildings - e.g. roads, green areas), combined with SD approach for the other areas in the catchment. In any case, setting up a combined SD and FD model depends on the case study and could require pre-processing to decide which areas should be SD or FD.

FD models are more physically realistic, avoiding the simplifications and spatial data aggregation of hydrological models applied on a subcatchment level in SD models. Nevertheless, the necessary resolution and accuracy of the available data requirements, either to define modules connections, hydrological characterisation, or even to do a proper calibration, are significantly higher for FD models. The increase in data availability and their resolution, as well as data sources will benefit FD models in the future.

The more and detailed data is available, the more tendency for FD models will be in the modelling and academic community. The increase in data available will lead to more tests similar to the comparison presented in this Thesis and the methodology proposed for model building and results comparison can be followed for different catchments. Interesting future work is to set criteria for the applicability of SD and FD models and study combinations of SD and FD models.

5.2 1D-2D interactions with stochastic sewer inlets' capacity

In this work, a methodology is proposed to stochastically model the consequences of a storm event, assessing the impact of sewer inlet operational conditions (i.e. clogging due to debris accumulation) on urban pluvial flooding. The proposed methodology comprises three main steps: (i) identification of sewer inlets most prone to clogging based upon a spatial analysis of their proximity to trees and evaluation of their location in relation to terrain depressions; (ii) Monte Carlo simulation of the capacity of inlets prone to clogging and subsequent simulation of flooding for each scenario, and (iii) the delineation of stochastic flood hazard maps. The methodology was demonstrated using as case study design storms as well as two storm events observed in the city of Coimbra, which reportedly led to flooding in different areas of the catchment.

The results show that variations in sewer inlet capacity can largely affect flooding occurrence and extent, and that considering such variations can lead to more accurate representation of urban pluvial flooding, as compared to traditional deterministic methods. For example, accounting for variations in inlet capacity can explain why, in some cases, relatively small rainfall events can lead to flooding, whereas larger storms may not. In addition, the stochastic flood evaluation can identify flood prone areas which traditional deterministic methods fail to identify.

The stochastic approach proposed in this study establishes a useful tool for dealing with uncertainties related with operational conditions of sewer inlet and allows a more comprehensive assessment of urban pluvial flood hazard, which in turn enables better informed flood risk management decisions.

The work presented can be explored further and the methodology proposed can be applied to access uncertainty of other modelling parameters. To explore the work presented, different catchment with operational problems of sewer inlets can be tested with the methodology proposed. Furthermore, the probability distribution used (Beta-PERT) can be tested against others available. This methodology was applied to the capacity of sewer inlets but can also be applied to access the uncertainty of other modelling parameters and will be interesting to see it applied for sensitivity analysis, such as to quantify infiltration parameters. Another interesting application of this methodology is also for conditions that might change during the storm event, such as to model the opening of a manhole cover due to the pressure of a surcharged sewer conduit.

5.3 Quasi-2D: solving 2D with 1D models

The proposed *Quasi-2D* concept was tested with models implemented in EPA SWMM. The mathematical formulation of this concept is demonstrated, and results were compared against 2D full dynamic models. The *Quasi-2D* concept proposed differ from other overland schemes applied with 1D schemes, because it covers continuous surfaces with a grid of regular elements similar to 2D schemes. This concept enables 1D models such as SWMM to perform analysis on 2D surfaces.

The *Quasi-2D* models tested achieved results in very good agreement with 2D full dynamic models but required higher simulation times. This is as a limitation of this concept; nevertheless, it can be minimized making use of more powerful CPUs.

The proposed *Quasi-2D* concept is based on 1D schemes to represent continuous overland surfaces commonly analysed with 2D models. The simplicity of 1D models added to the popularity of models such as SWMM makes the *Quasi-2D* modelling concept appealing for urban drainage modellers.

Future work is suggested to develop further the *Quasi-2D* and couple it with 1D sewer flow models. The mathematical formulation of the concept can be developed further with other grid types in addition to the rectangular grid presented. Different grid types can be regular grids based on geometric base elements, such

as triangles or hexagons, or irregular grids based on the aggregation of geometric elements, such as the aggregation of triangles to generate irregular polygons. The numerical approach of the *Quasi-2D* can also include new concepts to make it faster, such as the Quadtree method already applied to other 2D models. The *Quasi-2D* can be coupled with 1D sewer flow models, which should be an advantage because the same model used for the sewer network can also be applied to the overland surface.

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