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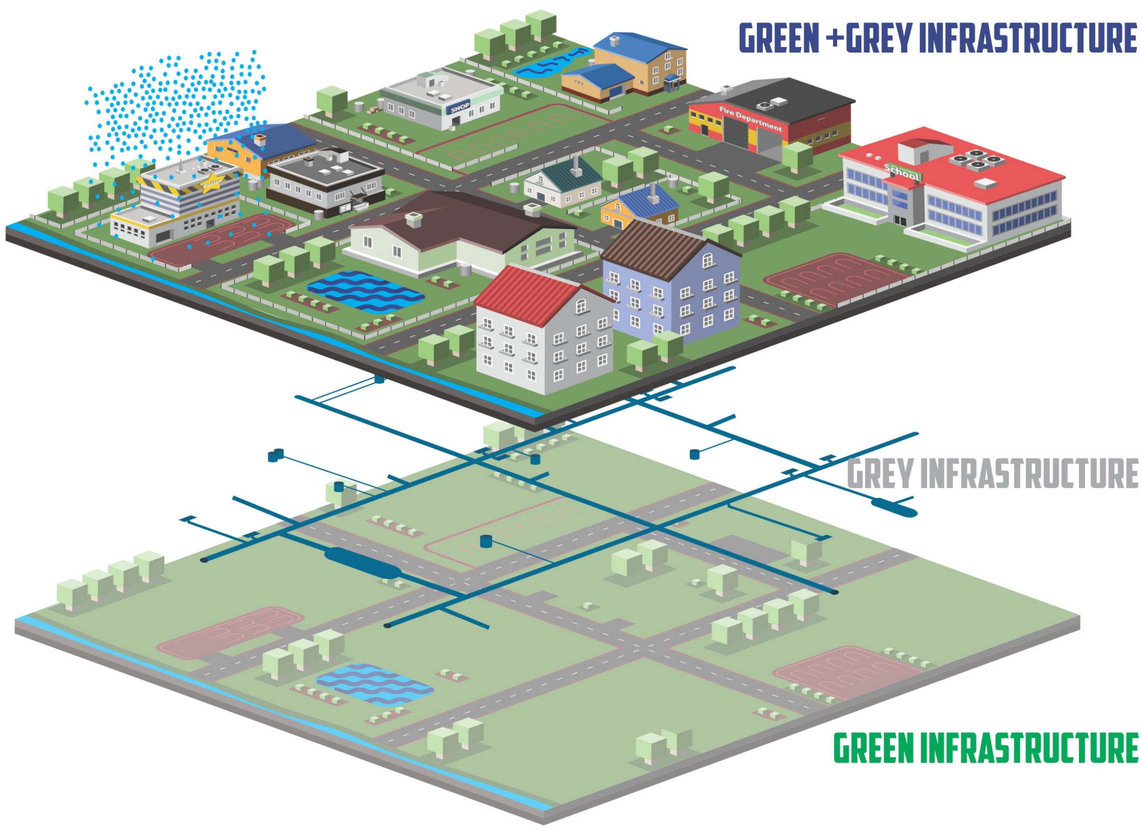

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DISSERTATION

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by

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Dedicated with endless love

to my wife Vanessa

and my son Carlos Daniel

Summary

Nowadays, economic development, urbanisation and heavy rainfall events are present in urban areas. A major change in approaches to the management of flooding is also ongoing in many countries worldwide. It has long been recognized that risk is a central consideration in providing appropriate flood protection and decision making. Urban development has a strong impact on the water cycle such as an increase in flood peaks, volume, hydraulic stress, water pollution and a decrease in base flow. Urban drainage systems (UDS) consist of pipe networks with underground structures; they are a critical link in the urban water cycle and an indispensable infrastructure to cope with floods by conveying water away from urban areas. However, their service level is often considered limited to an acceptable frequency of system overloading and their design capacity cannot cope with the extreme rainfalls and floods expected to occur.

Alternative solutions targeting these causes such as green infrastructure (GI) are well-known. GI provides not only the same drainage comfort as traditional drainage systems (also known as grey infrastructure), but also sustains the urban water cycle and pollution control at the source. Current drainage design practice is still based on the tradition of engineering solutions. In recent years the idea of finding an optimal combination of green and grey infrastructures (also known as "hybrid" solutions) which provide more effective and resilient solutions is being recognized as a promising area of investigation, and therefore is one of the main themes of this research.

Even though there are some decision support system tools available to evaluate green and grey infrastructures across a wide range of conditions as well as to compare alternative options, the performance of UDS that combines different green-grey solutions is still unclear and more work is needed to advance the present knowledge. Model simplification may be necessary to reduce the computational time. However, this will affect the representation of the physical processes which in turn will affect the accuracy of the model prediction. Therefore, there will be a trade-off between the degree of simplification (and the benefits that this brings) on the one hand and the model accuracy on the other hand, and this needs to be carefully addressed.

One-dimensional (1D) hydrodynamic models, either in the form of 1D or 1D-1D, are not capable of adequately representing the flooding over the urban surface, so the estimation of damage will inevitably be simplified. The use of 1D-2D modelling approaches for the purpose of urban flood analysis has already proved to be closer to real-world physics. However, the current modelling practice is largely based on the assumption that the rainfall-runoff and infiltration processes can be calculated within the 1D model and the

surcharged flows are then fed to the 2D model. In some cases this assumption can lead to significant inaccuracies.

The general objective of this research is to develop and test a framework to evaluate the performance of green and grey infrastructures for urban runoff reduction. With this aim, four specific objectives were defined: to assess how different combinations of green infrastructure measures perform within a drainage system in order to reduce runoff and pollution; to evaluate how the interactions between different grey infrastructures can influence the drainage system capacity; to develop and test a 2D infiltration algorithm within an existing 2D surface flood model and couple it with the 1D SWMM 5.1 software based on dynamic link libraries. This is needed for the purpose of modelling rainfall-runoff and representing green infrastructure within a 2D model domain; to propose a multi-objective model-based approach which can be used to evaluate different combinations of green-grey infrastructures.

In this work, a framework that aims to obtain the optimal configuration of green infrastructure (i.e. the optimal number of units distributed within the catchment) for urban runoff reduction is developed and tested. The research includes an assessment of the performance of green solutions dealing with environmental and economic objectives. This framework was applied in a highly urbanized catchment in Cali, Colombia. It could assist water managers and their stakeholders to assess the trade-offs between different GI.

This research studies the interactions between different grey infrastructure (pipes and storage tank sizing) to assess drainage system capacity. The work combines computational tools such as a 1D/2D flood inundation model and optimisation engine in the loop to compute in a 2D domain the potential damage for different rainfall events. The approach of expected annual damage cost (EADC) was also introduced into the evaluation as the probabilistic cost caused by floods for a number of probable flood events. The advantages of this approach are demonstrated in a real-life case study in Dhaka City, Bangladesh.

This work also proposes a new modelling framework which combines the infiltration process, overland flow and sewer system interactions. The performance of an outflow hydrograph generator in a 2D model domain was first investigated. Then, the effect of infiltration losses on the overland flow was evaluated through an infiltration algorithm (Green-Ampt method) added into what is called a Surf-2D model. The surface flow from a surcharge sewer was also investigated by coupling the Surf-2D model with the SWMM 5.1 open source code. An evaluation of two approaches for representing urban floods was carried out based on two main 1D/2D model interactions. Two test cases were implemented to validate the model.

Lastly, a multi-objective model-based approach to assess the optimal combination of green-grey infrastructures for urban flood reduction was developed. This framework was also applied in the case study of Dhaka City (Bangladesh) where different green-grey infrastructures were evaluated in relation to flood damage and investment costs. Including rainfall-runoff and infiltration processes along with the representation of GI within the 2D model domain enhanced the analysis of the optimal combined solutions which in turn allows the drainage system to be assessed holistically.

Clearly there are still limitations in this modelling framework regarding the influence that data has in the application of the methods specifically in terms of its type, quality and availability. However, the initial results look promising and the framework can easily be expanded when including topics such as social aspects, co-benefits obtained from green solutions and data from climate change scenarios.

Samenvatting

Tegenwoordig vindt economische ontwikkeling, verstedelijking en hevige regenval plaats in stedelijke gebieden. In landen over de hele wereld vindt ook een grote verandering plaats in de aanpak overstromingsbeheer. Het is al lang bekend dat risico een belangrijke overweging is bij het bieden van de juiste bescherming tegen overstromingen en tijdens besluitvorming. Stedelijke ontwikkeling heeft een sterke invloed op de watercyclus, zoals een toename van overstromingspieken, volume, hydraulische stress, watervervuiling en een afname van het basisdebiet. Stedelijke drainagesystemen (Urban drainage systems, UDS) bestaan uit leidingnetwerken met ondergrondse constructies; zijn een cruciale schakel in de stedelijke waterkringloop en een onmisbare infrastructuur om overstromingen het hoofd te bieden door water weg te voeren van stedelijke gebieden. Hun serviceniveau wordt echter vaak beschouwd als beperkt tot een acceptabele frequentie van overbelasting van het systeem en hun ontwerpcapaciteit is niet bestand tegen de verwachte extreme regenval en overstromingen.

Alternatieve oplossingen voor deze problemen, zoals groene infrastructuur (GI), zijn algemeen bekend. GI biedt niet alleen hetzelfde drainagecomfort als traditionele drainagesystemen (ook wel grijze infrastructuur genoemd), maar ondersteunt ook de stedelijke waterkringloop en de bestrijding van verontreiniging bij de bron. De huidige ontwerppraktijk voor drainage is nog steeds gebaseerd op traditionele, technische oplossingen. In de afgelopen jaren wordt het idee om een optimale combinatie van groene en grijze infrastructuren (ook wel bekend als "hybride" oplossingen) te vinden die effectievere en veerkrachtigere oplossingen bieden, erkend als een veelbelovend onderzoeksgebied en is daarom een van de belangrijkste thema's van dit onderzoek.

Hoewel er enkele beslissingsondersteunende systeemtools beschikbaar zijn om groene en grijze infrastructuren onder een breed scala van omstandigheden te evalueren en om alternatieve opties te vergelijken, zijn de prestaties van UDS, die verschillende groengrijze oplossingen combineert, nog steeds onduidelijk en is er meer werk nodig om de huidige kennis verder te ontwikkelen. Modelvereenvoudiging kan nodig zijn om de rekentijd te verminderen. Dit zal echter de weergave van de fysieke processen beïnvloeden, wat op zijn beurt de nauwkeurigheid van de modelvoorspelling zal beïnvloeden. Daarom zal er een afweging gemaakt moeten worden tussen de mate van vereenvoudiging (en de voordelen die dat met zich meebrengt) enerzijds en de modelnauwkeurigheid anderzijds, en die afweging moet zorgvuldig worden gemaakt.

Eendimensionale (1D) hydrodynamische modellen, hetzij in de vorm van 1D of 1D-1D, zijn niet in staat om de overstromingen in het stedelijk oppervlak adequaat weer te geven, waardoor de schatting van schade onvermijdelijk zal worden vereenvoudigd. Het gebruik van 1D-2D-modelleringsbenaderingen voor stedelijke overstromingsanalyses is al dichter

bij de echte fysica gebleken. De huidige modelleringspraktijk is echter grotendeelsgebaseerd op de aanname dat de neerslagafvoer- en infiltratieprocessen kunnen worden berekend binnen het 1D-model en de opgewaardeerde stromen vervolgens worden ingevoerd in het 2D-model. In sommige gevallen kan deze aanname tot aanzienlijke onnauwkeurigheden leiden.

De algemene doelstelling van dit onderzoek is het ontwikkelen en testen van een raamwerk om de prestaties van groene en grijze infrastructuren voor het verminderen van stedelijke afvoer te evalueren. Met dit doel werden vier specifieke doelstellingen gedefinieerd: beoordelen hoe verschillende combinaties van groene infrastructuurmaatregelen presteren binnen een drainagesysteem om afspoeling en vervuiling te verminderen; evalueren hoe de interacties tussen verschillende grijze infrastructuren de capaciteit van het drainagesysteem kunnen beïnvloeden; een 2D-infiltratie-algoritme te ontwikkelen en te testen binnen een bestaand 2D-oppervlakte-overstromingsmodel en dit te koppelen met de 1D SWMM 5.1-software op basis van dynamische linkbibliotheken. Dit is nodig voor het modelleren van regenafvoer en het representeren van groene infrastructuur binnen een 2D-modeldomein; een modelgebaseerde benadering met meerdere doelstellingen voor te stellen die kan worden gebruikt om verschillende combinaties van groengrijze infrastructuren te evalueren.

In dit werk wordt een raamwerk ontwikkeld en getest dat gericht is op het verkrijgen van de optimale configuratie van groene infrastructuur (d.w.z. het optimale aantal eenheden verdeeld binnen het stroomgebied) voor vermindering van de stedelijke afvoer. Het onderzoek omvat een beoordeling van de prestaties van groene oplossingen die betrekking hebben op milieu- en economische doelstellingen. Dit raamwerk werd toegepast in een sterk verstedelijkt stroomgebied in Cali, Colombia. Het zou waterbeheerders en hun belanghebbenden kunnen helpen om de afwegingen tussen verschillende GI te beoordelen.

Dit onderzoek bestudeert de interacties tussen verschillende grijze infrastructuur (leidingen en opslagtanks) om de capaciteit van het drainagesysteem te beoordelen. Het werk combineert computationele tools zoals een 1D/2D overstromingsmodel en een optimalisatie-engine in de lus om in een 2D-domein de potentiële schade voor verschillende regenvalgebeurtenissen te berekenen. De benadering van verwachte jaarlijkse schadekosten (expected annual damage cost, EADC) werd ook in de evaluatie geïntroduceerd als de waarschijnlijke kosten veroorzaakt door overstromingen voor een aantal waarschijnlijke overstromingsgebeurtenissen. De voordelen van deze aanpak worden gedemonstreerd in een real-life case study in Dhaka City, Bangladesh.

Dit werk stelt ook een nieuw modelleringskader voor dat het infiltratieproces, het overland stroom en de interacties met het rioolstelsel combineert. De prestaties van een uitstroom-hydrograafgenerator in een 2D-modeldomein werden eerst onderzocht. Vervolgens werd het effect van infiltratieverliezen op het overland stroom geëvalueerd door middel van een infiltratie-algoritme (Green-Ampt-methode) dat werd toegevoegd aan een zogenaamd Surf-2D-model. Ook is de oppervlaktestroom van een toeslagriool onderzocht door het Surf-2D-model te koppelen aan de open source code SWMM 5.1. Een evaluatie van de twee benaderingen voor het weergeven van stedelijke overstromingen werd uitgevoerd op basis van twee belangrijke 1D/2D-modelinteracties. Om het model te valideren zijn twee testgevallen geïmplementeerd.

Ten slotte werd een modelgebaseerde benadering met meerdere doelstellingen ontwikkeld om de optimale combinatie van groengrijze infrastructuren voor stedelijke overstromingsvermindering te beoordelen. Dit kader werd ook toegepast in de case study van Dhaka City (Bangladesh) waar verschillende groengrijze infrastructuren werden geëvalueerd in relatie tot overstromingsschade en investeringskosten. Het opnemen van regenafvoer- en infiltratieprocessen, samen met de weergave van GI binnen het 2D-modeldomein, verbeterde de analyse van de optimale gecombineerde oplossingen, waardoor het drainagesysteem holistisch kan worden beoordeeld.

Het is duidelijk dat er nog steeds beperkingen zijn in dit modelleringskader met betrekking tot de invloed die gegevens hebben op de toepassing van de methoden, specifiek in termen van type, kwaliteit en beschikbaarheid. De eerste resultaten zien er echter veelbelovend uit en het raamwerk kan eenvoudig worden uitgebreid met onderwerpen als sociale aspecten, nevenvoordelen van groene oplossingen en gegevens uit scenario's van klimaatverandering.

CONTENTS

1

INTRODUCTION

1.1 Background

1.1.1 Urban drainage

Small floods play an important role not only in the maintenance of floodplain fertility but also in the importance of regular flows to ecosystems. However, at the same time, floods are worldwide events which cause economic damage and loss of human lives; they are one of the biggest water-related environmental disasters known to society. According to reports (Teegavarapu, 2012), every year floods affect around 530 million people across the world, resulting in up to 25,000 deaths. They cost the world economy about €40 billion a year along with other water-related disasters. In the past decade, 96 percent of deaths in developing countries were due to natural disasters which occurred as a result of their limited capacity to forecast and manage them. In addition, the number of people vulnerable to floods is now rising due to population growth in natural floodplains, large-scale urbanization, deforestation, rising sea levels and climate change. New approaches are still needed to address these challenges and to build the required strategies.

Urban drainage is a critical link in the urban water cycle as an indispensable infrastructure to cope with floods by conveying water away from urban areas. Urban drainage systems (UDS) handle wastewater, water which contains dissolved material, fine and large solids originating from toilets, various sorts of washing, industry and other water uses. UDS also handles stormwater which if not drainage properly would cause damage, flooding and further health risks. Urban water systems conduct these two types of water with the aim of minimizing the problems caused to human life and the environment (Butler and Davies, 2011). Drainage systems consist of pipe networks with underground structures; their service level is often considered limited to an acceptable frequency of system overloading and the design capacity to cope with the extreme rainfalls and floods expected to occur.

1.1.2 Impacts from urban runoff and pollution

The impact of flooding is driven by a combination of natural and man-made factors. The increase in frequency and magnitude of urban flooding and urbanisation results in pollution problems in urban streams and other receiving waters. Discharges of wastewater during dry and wet weather conditions have the effect of cleaning urban surfaces and drainage channels, resulting in significant water deterioration. Floods worldwide have a range of threats to human life, health and well-being. Properties and their contents can be directly and indirectly affected by flooding in different ways. Direct impacts are the physical damage caused to buildings and their contents, while indirect effects include the loss of industrial or business processes.

Although for urban populations this may represent an indirect impact as they tend to be less involved in agriculture than the rural population, floods also cause deaths and injuries to livestock and fish stocks and they damage crops. Large-scale disasters like flooding can reduce food availability in cities. Food shortages lead to rising prices, resulting in economic and financial austerity (IFRC, 2010).

There are also indirect impacts caused by the complex interactions within the natural environment and the human use of resources in cities and towns (Jha *et al.,* 2012). Regarding the natural environment, high rainfall for instance can cause erosion and landslides damaging infrastructure and vegetation, and cause destruction to coral reefs in coastal areas. The impacts on human health as a result of flooding and water pollution can be very serious indeed. In developing countries, the majority of flood deaths have been found to be caused by diarrhoea and other water-borne diseases; also electrocution is the biggest cause of death in the immediate aftermath of flooding, followed by respiratory diseases, pneumonia and exposure to cold (Jha *et al.,* 2012). The provision of adequate non-contaminated water supplies during and after a flood event is critical as there are often problems due to lack of energy or gas to boil water for drinking. Education can also suffer due to malnutrition effects, displacement or closed schools (Bartlett, 2008).

The impacts of flooding and poor drainage are both complex and affect communities in different ways. For this reason, it is necessary to identify the most appropriate structural and non-structural interventions within a holistic framework for urban stormwater management. The organization of communities during flood events is important as it increases the chances of households and neighbourhoods being confident and prepared to cope with flooding. It is therefore important not to assume that drain construction will always be the most appropriate and cost-effective form of intervention to resolve urban drainage problems (Parkinson and Mark, 2005). It is generally agreed that urban drainage management must change to cope with greater urbanisation and climate change. There is also a general agreement on many of the problems with the current systems. Some of the most commonly cited issues are the division of responsibility for flooding, lack of funding, a lack of understanding of the causes of flooding, difficulties in improving drainage in existing developments, and barriers to the wider combined use of measures such as green and grey infrastructures. Beyond these operational issues, some argue that a dramatic change in attitude is necessary (POST, 2007).

1.1.3 Urban flood risk management

A major change in approaches to the management of flooding is now ongoing in many countries worldwide. It has long been recognized that risk is a central consideration in providing appropriate flood protection and decision-making under uncertainty. Flood risk management involves the purposeful choice of suitable plans, strategies and measures that are intended to reduce flood risk (Pender and Faulkner, 2011). The objective of a risk assessment is to provide a quantitative measure of the possible impacts of natural hazards due to rapid urbanization, environmental degradation, development in high-risk areas, movement of populations from rural to urban areas, and the effects of climate change. The results can enhance resilience to disaster and climate change by informing the selection and design of infrastructure and other urban investments.

Resilience enhances the ability to cope with flooding and to recover from flooding. With resilient systems, communities or societies exposed to hazards have the ability to resist, absorb, accommodate to, and recover from their effects efficiently by preserving and restoring essential basic structures (Jha *et al.,* 2013). They can be established in different scales: at property level, neighbourhood level, or city level. Enhancing resilience depends on having enough flexibility to continue providing for essential needs given future risks and uncertainty. Resilience is also related to the strong intent to increase capacity building of human resources, better land use management, increased flood preparedness and emergency measures that are taken mostly during and after flood events (Batica and Gourbesville, 2012).

The idea of combining traditional infrastructure with new ones can be applied through a number of methods which vary from products, material, and technologies to reduce flood risks and contamination. These solutions are used to manage responses (e.g. extreme rainfall) mainly at the local level as part of an overall approach that aims to develop system resilience (Butler *et al.,* 2014). Adaptable approaches should be cumulative and changeable in response to new knowledge, demands and expectations with associated flexibility in standards and practices (Geldof, 2005; Watkinson *et al.,* 2006).

It is well known that urban development has a strong impact on the water cycle such as an increase in flood peaks, volume, hydraulic stress, water pollution and a decrease in base flow. In recent years, alternative measures targeting this causes have become more popular (Barbosa *et al.,* 2012; Elliot and Trowsdale, 2007; Marsalek and Schreier, 2009). Best Management Practices (BMP), also called Sustainable Urban Drainage Systems (SUDS), Low Impact Development (LIDs) or Green Infrastructure (GI), provide not only the same drainage comfort as traditional drainage systems (known as grey infrastructure) but also sustain the urban water cycle and pollution control at the source. However, the current drainage design practice is still based on the tradition of engineering solutions. In recent years the idea to find an optimal combination of green and grey infrastructures which provides more effective and resilient solutions has been recognized as a promising area of investigation, and therefore is one of the main themes of this research.

1.2 Motivation of this research

1.2.1 Urban drainage practice – challenges and limitations

Nowadays, economic development, urbanisation and heavy rainfall events are affecting the possibilities for effective management of stormwater and wastewater systems. Hence, there is a need to continuously advance our knowledge and practice in order to design and implement more effective solutions that can respond to such challenges. Furthermore, the solutions that we need to implement need to be multifunctional, robust and flexible/adaptive. Some of the key limitations of current practice can be summarized as follows:

- Grey infrastructure measures are no longer efficient to respond to the above challenges.
- Most of the current models that are used for assessment of urban drainage systems are based on assumptions that do not necessarily reflect the physics of the catchment processes (i.e. rainfall-runoff, infiltration).
- There are a great deal of uncertainties associated with the above stated challenges and therefore it is difficult to plan expansion of urban drainage systems.
- Information concerning cost-benefit analysis of green infrastructure measures is still inadequate to derive more reliable estimates.

Most of these issues have been addressed within integrated or holistic approaches. However, response strategies attempting to enhance the performance of urban water systems with the use of selected combined solutions (green-grey, known as "hybrid" solutions) are still needed. The present research addresses some of these key issues and it focuses on optimal solutions that can improve the performance of UDS through novel modelling methods.

1.2.2 The need for optimal combination of solutions

Although there are some decision support systems tools available to evaluate green and grey infrastructures across a wide range of conditions as well as to compare alternative options, the performance of an urban drainage system that combines different green-grey solutions is still unclear and more work is needed to advance the present knowledge. Further studies should take the following considerations into account:

1. Green-grey infrastructures have to deal with a wide range of rainfall events which in turn demands better understanding of the interactions between the hydrological processes that occur below ground and above ground.

2. The search for optimal combination of green-grey solutions demands a thorough analysis of trade-offs between different objectives and needs.

1.2.3 Issues concerning numerical models

Numerical models are invaluable for the evaluation of different combinations of green-grey infrastructures. However, there are several issues that need to be taken into account:

1. Model simplification may be necessary to reduce the computational time. However, this will affect the representation of the physical processes which in turn will affect the accuracy of the model prediction. Therefore, there will be a trade-off between the degree of simplification (and the benefits that this brings) on the one hand and the model accuracy on the other hand, and this needs to be carefully addressed.

2. One-dimensional (1D) hydrodynamic models, either in the form of 1D or 1D-1D, are not capable of adequately representing the flooding over the urban surface, so the estimation of damage will inevitably be simplified. The use of 1D-2D modelling approaches for the purpose of urban flood analysis has already proved to be closer to real-world physics. However, current modelling practice is largely based on the assumption that the rainfall-runoff and infiltration processes can be calculated within the 1D model and the surcharged flows are then fed to the 2D model. In some cases this assumption can lead to significant inaccuracies.

1.3 Research objectives

The main objective of the present research is *to develop and test a framework for evaluation of the performance of green and grey infrastructures for runoff and pollutant reduction*. The specific objectives are:

The main objective of the present research is *to develop and test a framework for evaluation of the performance of green and grey infrastructures for runoff and pollutant reduction*. The specific objectives are:

1. To assess how different combinations of green infrastructure measures perform within a drainage system in order to reduce runoff and pollution.

2. To evaluate how the interactions between different grey infrastructures can influence the drainage system capacity.

3. To develop and test a 2D infiltration algorithm within an existing 2D surface flood model and couple it with the 1D SWMM 5.1 software based on dynamic link

libraries. This is needed for the purpose of modelling rainfall-runoff and representing green infrastructure within a 2D model domain.

4. To propose a multi-objective model-based approach which can be used for evaluation of different combinations of green-grey infrastructures.

1.4 Research questions

Given the objectives, the main research question that can be formulated as follows:

How effective are green and grey infrastructures for runoff and pollution reduction?

Specifically, the present research addressed the following specific research questions:

RQ1: ¿How can hydraulic models and optimisation techniques be coupled for the purpose of assessing the effectiveness of green and grey infrastructures for flooding and pollution?

RQ2: ¿How can the infiltration process, overland flow and sewer system interactions be implemented within a coupled 1D/2D model?

RQ3: ¿How can the 1D/2D coupled model addressed in Q2 be used to assess the optimal combination of different infrastructure measures?

1.5 Research approach

Based on the identified limitations of current urban drainage practices, the considerations for optimal combination of solutions and issues concerning numerical models, in this dissertation an evaluation of green and grey infrastructures is developed by providing tools and knowledge to facilitate its achievement. This work is divided into four main parts which address each research question.

An initial selection, placement and costing of green and grey infrastructures is needed based on their characteristics and suitability for implementation in an urban area. With the objective of reducing runoff and pollution in an urban catchment, the optimal number of GI units distributed within the catchment along with a trade-off between pollution load, peak runoff, flood volume and investment are applied. In this work the selection of the type of infrastructure that allows a particular objective for different rainfall events to be reduced is also identified.

This research also studies the interactions between different grey infrastructures to assess a drainage system´s capacity. The work combines computational tools such as 1D/2D flood inundation models and an optimisation engine in the loop to compute in a 2D domain potential damages for different rainfall events. The approach of expected annual damage cost (EADC) was also introduced into the evaluation as the probabilistic cost caused by floods for a number of probable flood events.

A proposed modelling framework which combines the rainfall-runoff and infiltration process on the overland flow and its interaction with a sewer network is presented. An infiltration algorithm which uses the Green-Ampt method was coded into a previous 2D model and then coupled with a 1D sewer model based on dynamic link libraries. Two test cases were implemented to validate the model.

Lastly, a multi-objective model-based approach to assess green-grey infrastructures holistically for urban flood reduction is proposed. This framework suggests three main components to form the structure of a proposed modelling framework. The first two components provide the optimal number of green infrastructure units distributed within the catchment and optimal grey infrastructure such as pipes and storage tank sizing. The third component evaluates the selected combined green-grey solutions based on rainfall-runoff and infiltration computation in a 2D model domain.

These solutions implemented separately have an impact on the reduction of damage and investment costs. However, the combined optimal green-grey solutions have also been evaluated. It was proved that including rainfall-runoff and infiltration processes along with the representation of GI within the 2D model domain enhanced the analysis of the optimal combination of solutions and this in turn allowed a drainage system to be assessed holistically.

1.6 Thesis outline

The thesis is composed of seven chapters presented in Figure 1.1. *Chapter 1* presents as an introduction: the background and motivation of this research. *Chapter 2* presents a review of green and grey infrastructures for urban drainage management. The background of the types and tools for planning and selecting urban drainage infrastructure and its future drivers are part of the items to be reviewed. The next four chapters are focused on the four specific objectives presented in Section 1.3. Each chapter is based on peer-reviewed journal publications. Figure 1.1 also shows the different chapters, the connection between them and how they are related to the research questions.

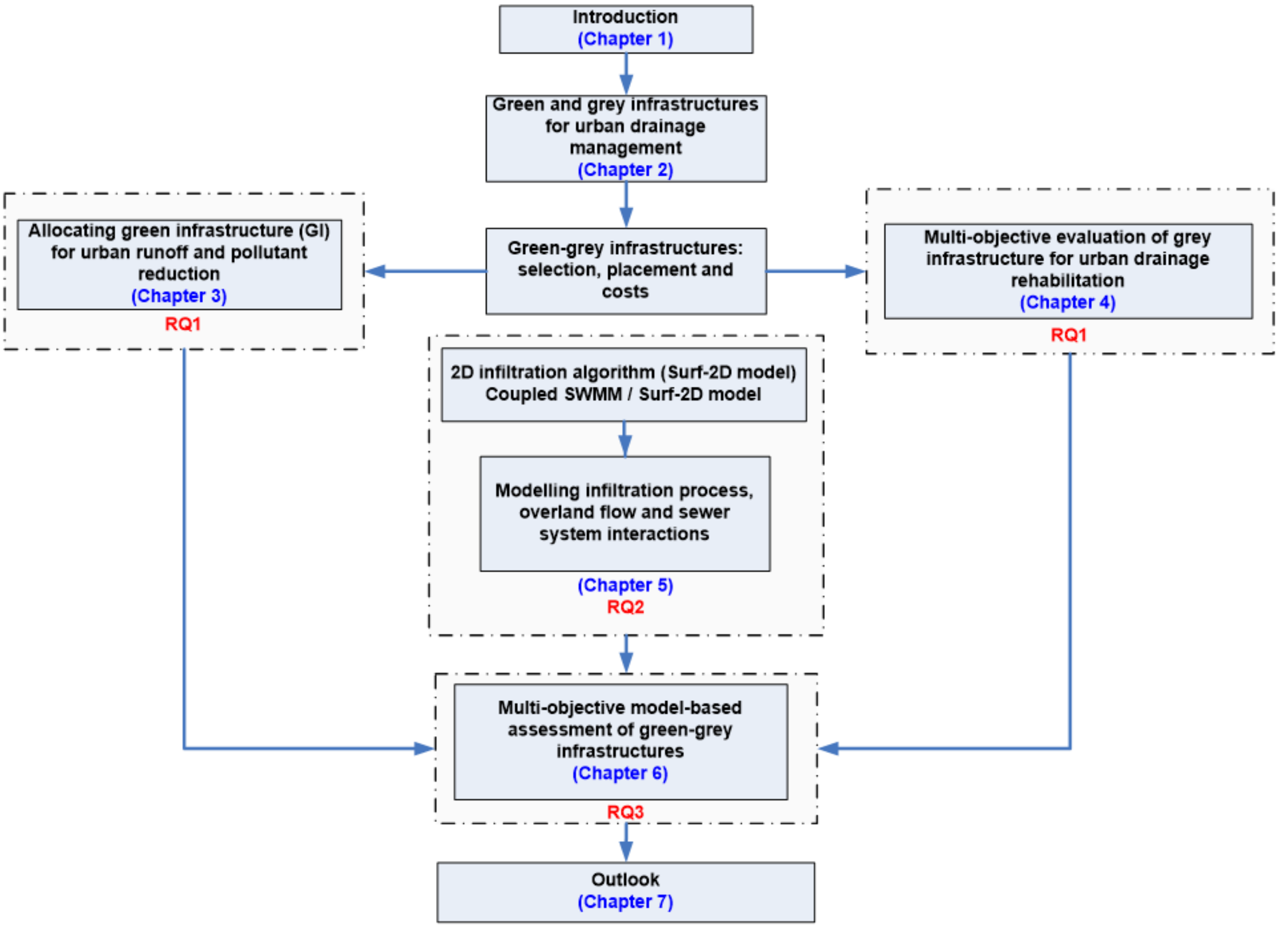


Figure 1.1. Overview of the methodological approach

The performance of Green Infrastructure (GI) for urban runoff and pollutant reduction is explored in *Chapter 3*. The framework obtains the optimal number of GI units distributed within the catchment dealing with environmental and economic objectives. The selection of the type of infrastructure that allows a particular objective for different rainfall events to be reduced is also identified.

An evaluation of a drainage system's capacity to better understand the interactions between different grey infrastructures is presented in C*hapter 4*. It aims to minimise flood damage (computed in a 2D domain) and rehabilitation cost. This evaluation also presents the expected annual damage cost (EADC) concept, as the probabilistic cost caused by floods for a number of probable flood events. It combines computational tools such as a 1D/2D coupled model and an optimisation engine in order to compute potential damage for different rainfall events.

A new modelling framework which combines infiltration process, overland flow and sewer system interactions is investigated in *Chapter 5*. An infiltration algorithm which uses the Green-Ampt method was coded into a previous 2D model and then coupled with a 1D sewer model based on dynamic link libraries. Two test cases were implemented to validate the model. This chapter shows in general that with this model, similar results in terms of peak discharge, water depth and infiltration loss against other dynamic and diffuse models were observed. The results from the two main 1D/2D model interactions show that for some cases significant differences in terms of flood extent, maximum flood depth and inundation volume were found.

In order to assess the combination of green-grey infrastructures for urban runoff reduction, a multi-objective model-based approach is presented in *Chapter 6.* Three main components are proposed to form the structure of the modelling framework. Selected combined green-grey solutions based on rainfall-runoff and infiltration computation accounted in a 2D model domain have been evaluated. It was proved that including rainfall-runoff and infiltration processes along with the representation of GI within the 2D model domain enhanced the analysis of the optimal combined solutions and this in turn allowed a drainage system to be assessed holistically. Finally, *Chapter 7* describes the outlook and reflections of this research.

2

GREEN AND GREY INFRASTRUCTURES FOR URBAN DRAINAGE MANAGEMENT

An evaluation of green and grey infrastructures is a complex matter, in order to develop a sustainable plan, it involves a combination of several subjects. This Chapter presents a review of green and grey infrastructures for urban drainage management. The background of the types and tools for planning and selecting urban drainage infrastructure and its future drivers are part of the items to be reviewed.

2.1 Introduction

A new generation of infrastructure projects is necessary to achieve development goals, including water security, disaster risk reduction, poverty alleviation, and resilience to climate change (World Bank, 2019). Many researches propose different types of infrastructure depending on the objectives to achieve. There are relevant factors affecting drainage systems such as evaluating water quality (Elliot and Trowsdale, 2007; German et al., 2005; Ingvertsen, 2011), technical and economic issues (Davis and Birch, 2009; Zhou et al., 2013), Law and social concerns (Hvitved-Jacobsen et al., 2010) and drainage source control (Chocat et al., 2007; Schroll et al., 2011), among others. As it arises different types of infrastructure can be used as a basis for prioritizing actions to prevent floods and pollution. Priorities can be implemented based on assessments of flooding risks, but also on economic, social, natural, physical and institutional assessment.

Proposed infrastructure should be flexible and resilient with an optimal arrangement to identify which alternative provides a better performance to runoff control. The objectives for the design and dimensioning infrastructure are driven in large part by the social system environment. As such, social values have continuing effects on most objectives (UNESCO, 2017). The general economic development and its consequences for the funding of infrastructure providers will also influence the objectives (Scholes et al., 2006). A change in the legal framework could have consequences for the objectives of infrastructure as well (E.U., 2017).

An important driver for identifying suitable infrastructure is rainfall. The duration, intensity and frequency of rainfall events are affected by long-term fundamental alterations in climate patterns. Description of causes and general consequences of global climate change are presented in IPCC (2021). Another driver considered for selecting infrastructure is pollution load influenced by the land-use of the catchment area (Ingvertsen, 2011). Spatial developments have important influences on the pollution loads of runoff. Future developments of the land-use are characterized by uncertainties. Drivers of the system for hydraulic performance, treatment, maintenance, technological development are also most of the major future uncertainties concerning the implementation of different infrastructure. By defining scenarios, it is possible to reduce the number of possible combinations of different drivers to a limited of possible future states (Eckart, 2012; Scholes and Revitt, 2008).

Due to relatively recent advances in different areas related to computer models, optimisation techniques have been used to address water related problems with the purpose of finding optimal solutions (Maier et al., 2014). Prospects for future development by improving design efficiency, implementing integrated design, handling multiple design objectives, involving feasible design constraints and investigating impacts of cost models among others, have been identifying as new horizons (Basdekas, 2014).

In addition to this, evaluating the performance of urban water systems is also nowadays a concern. There are still challenges and questions related to the suitable characterisation and representation of the spatial and temporal distribution of runoff water in urban catchments. Recently, in urban flood modelling not only the influence of the sewer system in the overland flow is of recognised importance (Mignot et al., 2014; Chen et al., 2015) but also the interaction between surface water and infiltration in order to better estimate inundation extent and water depths (Mallari et al., 2015; Park et al., 2019). The theoretical concepts mentioned above have in common that they provide the foundation to assess a new generation of infrastructure for urban runoff reduction. To approach this idea in this research, a review of the most relevant aspects is presented below:

2.2 Types of urban drainage infrastructure

2.2.1 Grey infrastructure

Grey infrastructure refers to constructed structures such as sewer and stormwater systems, storage and treatment facilities. Part of the following description about grey infrastructure specifically sewer systems as the main infrastructure applied in this research has been taken from the Water Environment Federation – WEF (2017) as follows:

Sewers are a series of connected pipes or pipelines that convey either wastewater or storm water to a designated downstream location for treatment and or disposal. There are three different types of sewers: sanitary, storm or combined. Sanitary sewers and combined sewers convey wastewater from homes, institutions and businesses to a centralized treatment plant. Sanitary sewers carry only wastewater whereas combined sewers carry both wastewater and storm water. Wastewater conveyance and treatment are important because they help to prevent waterborne illnesses and promote general sanitation before safely discharging to receiving waters. Storm sewers convey snowmelt and rainwater from yards, sidewalks and road ways and route it to receiving waters directly or through best management practices facilities to remove certain pollutants.

Sewer defects are pipe system deficiencies resulting from system aging, structural failure, lack of proper maintenance, and/or poor construction and design practices. They can include conditions such as broken pipe, leaking joints, manhole lids with holes, poor sealing and root infested sewer laterals (Figure 2.1) In sanitary sewers, this can lead to excessive *Infiltration and Inflow (I/I)* which can be more noticeable after precipitation conditions.

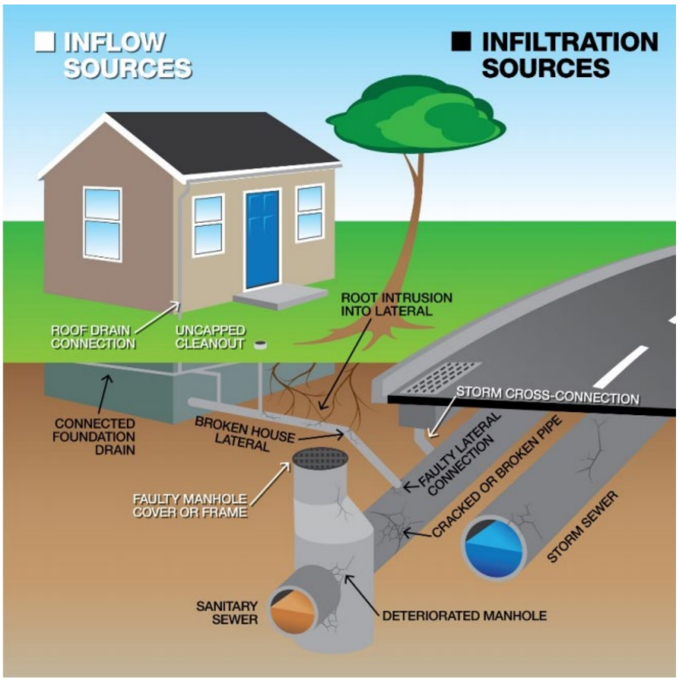


Figure 2.1. Typical Sources of I/I in sanitary sewer systems (WEF, 2017)

The state-of-the-art industry experiences indicate that before investing in sanitary sewer capacity improvements to handle excessive *I/I*, it is prudent to improve sewer system structural conditions to realize practical levels of *I/I* reduction first and then consider supplementing with right-sized conveyance/storage and downstream treatment systems. Rehabilitating the sewer system should be undertaken first to determine the magnitude of *I/I* reduction possible. It may be that partial or comprehensive rehabilitation of the system restores adequate levels of the conveyance capacity. Additional conveyance/storage/treatment capacity should be supplemented as needed.

Sewer rehabilitation can be considered both *repair* and *renewal*, to reduce extraneous flow and address structural defects. *Repair* are generally made to allow the pipe to function to the end of its useful life whereas *renewal* is more comprehensive than *repair* and extends the useful life of the pipe. Sewer rehabilitation projects can include a mixture of *repairs* and *renewal* with a focus on both restoring structural integrity and practical reduction of *I/I*. Each system component is analysed to determine where defective areas allow *I/I* to enter the system and the most cost-effective *repair* or *renewal* method is applied to eliminate that source of *I/I* while ensuring that the extraneous water does not migrate to enter the system through a different defect.

Assessing grey infrastructure

Within the grey infrastructure evaluation three main aspects are taking into account: structural, hydraulic, environmental and social conditions (Butler and Davies, 2011; Barreto, 2012; Stanic, 2016). These conditions require a proper judgment or experience in addition to an objective technical analysis. Since it is not possible to predict the accurate moment of a pipe failure, categorization using different levels of deterioration or risk make it possible to rate the condition of the system. There are a number of sewer condition ratings that have been developed as general guide lines. The Water Research Center (WRc) in Europe, the Water Environment Federation and American Society of Civil Engineering in USA, and the National Research Council in Canada are some examples. Normally, Municipalities develop their rehabilitation plans and adopt a specific condition assessment system, others adapt the mentioned guidelines. Not all the guide lines items can be implemented in all cases, each city has its own requirements and dispositions.

Structural conditions

Different inspection methods can be taken into account for a sewer pipe evaluation taking into account the amount of information supplied, the costs of the application and their availability. Sewer manhole inspections are an important component of the sewer system assessment due to the sensitivity of manholes to structural defects and/or Infiltration and Inflow (*I/I)* which may produce sewer overflows. Manhole inspection not only provides valuable information on the physical condition of the manholes, but also an opportunity to observe pipe diameters, inverts and surcharging within mainline sewers (BSD, 2014).

I/I into sewer systems is not desired as among other things, it decreases the performance of wastewater treatment plants and increases combined sewer overflows. As sewer rehabilitation to reduce *I/I* is expensive, water managers not only need methods to accurately measure *I/I,* but also they need different frameworks to assess the actual performance of grey infrastructure.

Staufer et al. (2012) statistically assess the performance of grey infrastructure to reduce *I/I*. It was possible by using observations in a defined catchment as a control group and assessing the significance of the observed effect by regression analysis. They demonstrate the usefulness of the approach in a case study, where rehabilitation reduced groundwater infiltration by 23.9% and stormwater inflow by 35.7%. Although the results were not statistically significant, investigations into the experimental design of monitoring campaigns confirmed that the variability of the data as well as the number of observations collected before the rehabilitation, impact the detection limit of the effect. Future practical applications should consider a careful experimental design.

Close Circuit Television CCTV inspection is used since decades as industry standard for sewer system inspection and structural performance evaluation. Khan et al. (2010) presents a study which uses artificial neural networks to investigate the importance and influence of certain characteristics of sewer pipes upon their structural performance, expressed in terms of condition rating. Results of sensitivity analysis describe the nature and degree of the influence of each parameter (related sewer pipelines, pipe diameter, buried depth/cover, bedding material, pipe material, pipeline length, age, and closed circuit television - CCTV) on pipe structural condition. The developed models are expected to benefit academics and practitioners (municipal engineers, consultants and contractors) to prioritize inspection and rehabilitation plans for existing sewer mains.

A case study in Bogotá - Colombia, Angarita et al. (2017) identified and quantified physical and environmental explanatory variables for the structural state of urban drainage networks. Within the analysis used information from 2291 CCTV inspections collected by the Water and Sewerage Company of Bogotá using tele-operated equipment during 2008-2010. Linear regression models were used to identify the environmental and physical characteristics of the pipes that are significantly associated with the occurrence, magnitude and type of the failures commonly found. Despite the fact that the correlation levels show that the developed model has a very low predictive capacity. It was found that the process of selecting assets for CCTV inspection can be optimized, increasing the success rate in failure detection.

Another case study in Malaysia has been presented in Safira et al. (2018). The main objective of the study was to develop a prediction tool for the structural condition in open channel sewer pipe in order to facilitate operator in estimating the degradation risk of a certain sewer pipe. Closed-circuit television (CCTV) research was used to observe the structural condition of sewer pipe; therefore, it can be classified using pipeline assessment and certification program (PACP) grading system. The Markov chain model was later used to predict the future structural condition in open channel sewer pipe prior to the development of prediction tool. A total of 37 km length of sewer pipe which covers an estimated 23% of total length of sewer pipe within the study area was evaluated.

The structural resilience theory provides a theoretical basis for establishing uniform system resilience and impact scales or indices across various disciplines in a consistent manner. For instance, Shi et al. (2018) introduce the concept of structural damage energy within the structural resilience of sewer system. Based on the obtained structural behaviours of the renovated pipe specimens, the structural damage energy is introduced.

The structural resilience is theory based on three important equations: the total energy absorbed by the system environment due to a disturbance, the resilience index of a given system derived from the percentage of system damage, and an impact function of the system environment. Based on a clear and conceptual definition of resilience, these scales and indices can be used to develop resilience systems and effective resilience strategies for systems, facilitate consistent policy-making across fields, and reinforce the resilience approach in general, making sustainable development with nature an achievable goal.

Caradot et al. (2017) presents an evaluation of uncertainties in sewer condition assessment. Results indicate that the probability to inspect correctly a pipe in poor condition is close to 80%. The probability to overestimate the condition of a pipe in bad condition (false negative) is 20% whereas the probability to underestimate the condition of a pipe in good condition (false positive) is 15%. Finally, sewer condition evaluation can be used to assess the general condition of the network with an excellent accuracy probably as the respective effects of false positive and false negative are safeguard.

By identifying high failure risk areas, inspections can be implemented based on the system status and thus can significantly increase the sewer network performance. Anbari et al. (2017) developed a new risk assessment model to prioritize sewer pipes inspection using Bayesian Networks (BNs) as a probabilistic approach for computing probability of failure and weighted average method to calculate the consequences of failure values. Results show that majority of sewers (about 62%) has moderate risk, but 12% of sewers are in a critical situation. Regarding the budgetary constraints, the proposed model and resultant risk values are expected to assist wastewater agencies to repair or replace risky sewer pipelines especially in dealing with incomplete and uncertain datasets.

The prompt repairs are a process by which critical repair systems is being done in a timelier and cost effective way. The methodology makes use of the concept that when critical failures promise prompt repair during assessment activities, actions will be taken to tackle the problems by on-call Contractors. Following the standards assessment given in BSD (2014), prompt repairs of sewer infrastructure assets are authorized when critical defects meet an immediate threat to the environment, to the public health and safety, it creates operational problems that may result in sewer overflows or contribute substantial inflow to the system.

Visual inspections can be also conducted during a CCTV operation by the CCTV Contractors. The structural condition, amount of sediment, flow, volume, flow contents, debris and odour can be observed and noted for each manhole. Each manhole can be assigned an overall condition of “excellent”, “good”, “fair”, or “poor” during the visual inspection. The possibility of having different inspection methods to analyse the structural conditions of sewer systems strengthens the information provided to sewer managers in order to take appropriate decisions.

Hydraulic conditions

Hydraulic losses in sewer pipes are caused by wall roughness, blockages and in-pipe sedimentation. Hydraulic resistance is a key parameter that is used to account for the hydraulic energy losses and predict the sewer system propensity to flood. Roughness change over time due to corrosion processes, joint eccentricity and subsidence. Raised hydraulic roughness due to aging of the pipe material decreases the flow capacity of the pipe, resulting in reducing systems hydraulic performance. It is valuable to know the actual condition of the asset pipes such as hydraulic roughness and the precise interior geometry to decide whether or not a given pipe has enough hydraulic capacity.

A common method to estimate the hydraulic resistance of a sewer is to analyse collected CCTV images and then to compare then against a number of suggested hydraulic roughness values published in the sewer rehabilitation manuals. Romanova (2013) reports on the development of a novel, non-invasive acoustic method and instrumentation to measure the hydraulic roughness in partially filled pipes under various structural and operational conditions objectively. Results indicate that for the local roughness the energy content of the reflected acoustic signal is an indicator of the pipe head loss and hydraulic roughness. In the case of the distributed roughness, the variation in the temporal and frequency characteristics of the propagated sound wave can be related empirically to the mean flow depth, mean velocity, wave standard deviation and hydraulic roughness.

Laser scanning offers challenging perspectives for measuring the characteristics of sewer pipes. Improvements in laser technology and digital cameras in theory admit a cost-effective application of laser profiles to measure the interior geometry of sewer pipes. Clemens et al. (2014) built a laboratory set-up to expose based on tests on a new and an 89 years old corroded sewer pipe, that laser scanning is certainly capable of measuring the interior geometry accurately enough to determine wall thickness losses for corroded pipes as long as the position and alignment of the laser and camera are accounted for. The obtained accuracy, however, was not enough to quantify the hydraulic roughness.

Information on the hydraulic conditions of deteriorated sewer pipes will contribute to better understanding of the changes in processes which is essential for reaching effective sewer asset management. Stanic et al. (2016) present the potential of laser scanning methods for accurate, non-invasive and nonintrusive assessment of the hydraulic roughness of concrete sewer pipes. The results show a promising potential of laser scanning approach for a simple and fast quantification of the hydraulic roughness in a sewer system.

Another research related to the risk of hydraulic failure in concrete sewers due to internal corrosion has been done by Kuliczkowska (2016). The purpose of the investigation was to find out the frequency of internal corrosion occurrence in concrete sewers, the thickness of corroded walls and to develop the method of determining the risk of hydraulic failure due to corroded pipes. The proposed method makes it likely to remove or somewhat to decrease structural risk caused by internal corrosion in concrete sewers which can be done by scheduled CCTV surveys of sewer and trenchless renewal performed according to plans established in advance.

Recently, Li et al. (2019) proposed a long-term study to identify the controlling factors of concrete sewer corrosion using well-controlled laboratory-scale corrosion chambers to vary levels of H_2S concentration (hydrogen sulphide), relative humidity, temperature and in-sewer location. Using the results of the long-term study, three different data driven models, i.e. multi linear regression (MLR), artificial neural network (ANN) and adaptive neuro fuzzy inference system (ANFIS), as well as the interaction between environmental parameters, were assessed for predicting the corrosion initiation time (t_i) and corrosion rate (r). It was observed that t_i prediction by these models is quite sensitive. However, they are more robust for predicting r as long as the H_2S concentration is available. Using the H_2S concentration as a single input, all three data driven models can reasonably predict the sewer service life.

Hydraulic reduction performance in sewer and waste-water pipes also appears due to concrete biogenic corrosion (bio-corrosion). It occurs mainly because of the diffusion of aggressive solutions and in situ production of sulfuric acid by sulphur-oxidizing microorganisms. Its prevention commonly needs modification of the concrete mix or the application of a corrosion-resistant, chemical/antimicrobial-coating layer on the inner surface of the pipes. Roghanian and Banthia (2019) investigated three broad families of coating materials, Portland cement-based, geopolymer-based and geopolymer magnesium phosphate-based coatings. Results show that the developed coating materials have significantly superior resistance to biogenic corrosion compared to ordinary cement-based coatings.

Monitoring sewer sediments is essential to understand sedimentation and erosion processes within the hydraulic losses framework. Sonar is one of the available techniques to measure sediments in sewer pipes. Lepot et al. (2016) evaluate a sonar technique, quantify its uncertainties and test it under different circumstances by using both laboratory and field experiments. To this purpose they present a new algorithm to recognize the water-sediment interface. A new gradient algorithm has been introduced and tested to better estimate water-sediment and sewer wall interfaces. The maximum gradient algorithm results in a better accuracy, especially for sewer wall detection. This improves the accuracy of the superimposition of the sewer pipe profile with the sediment profile. Data analysis and validation has been done with the goal of automatically compute sediment areas and volumes.

Sedimentation in sewers also occurs regularly according to the alternating natural flow. Bonakdari et al. (2015) investigate the hydraulic characteristics of flow in channels with a circular cross section with different bed slope and their effects on sediment transport capacity by using a 3D numerical simulation of flow field with ANSYS-CFX software. It studies hydraulic features of the flow passing through a circular channel in a two or three phase conditions. Self-cleansing velocity and volumetric sediment concentration in various Froude numbers were computed by lab outcomes to validate the results of numerical model. Results of numerical simulations indicate a proper adaptation of numerical and experimental models. Longitudinal velocity counters obtained by numerical simulation were compared in two or three phase flows. Fluctuations on bed plate introduced sediment transfer near circular channel bed.

Environmental and social conditions

As the sewer ages, it is required a rehabilitation work plan not only for preserving the sewer system in an optimal level of service but also for mitigating environmental and social impacts. The predominant environmental impact is the overflow discharges to the receiving water bodies. If the pollutant concentration or mass is high, it can yield serious damages to aquatic life and/or users downstream. Additional source of pollution is the exfiltration of sewers to aquifers. Considered problems associated include diseases promoted by vectors proliferation, bad odours and poor aesthetics due to rubbish.

Using low-cost sensors, data can be collected on the occurrence and duration of overflows in each CSO structure in a Combined Sewer System (CSS). The collection and analysis of real data can be used to assess, improve, and maintain CSS in order to reduce the number and impact of overflows. Montserrat et al. (2015) introduced a framework to evaluate the performance of CSS using low-cost monitoring, to assess the capacity of a CSS using overflow duration and rain volume data and to define the performance of CSO structures with statistics. The study also includes the evaluation of a CSS agreement with government guidelines and the generation of decision tree models to provide support to managers for making decisions about system maintenance. Demonstrated with a real case study, the results obtained can greatly support managers and engineers dealing with real-world problems, improvements, and maintenance of CSS.

Methods to monitor and control Combined Sewer Overflow (CSO) still require improvements. Maté-Marín et al. (2018) present a device for stormwater and combined sewer flows monitoring and the control of pollutant fluxes called DSM-flux. It is a channel that provides appropriate hydraulic conditions suitable for measurement of overflow rates and volumes by means of one water level gauge. A stage-discharge relation for the DSM-flux is obtained experimentally and validated for multiple inflow hydraulic configurations. Regarding these results, the DSM-flux appears to be a good alternative compared to current methods in order to reliably monitor CSO structures. The measurement method is independent of the hydraulic conditions upstream from the device and uncertainties associated with the discharge and volume measurements are relatively low, particularly for heavy storm events.

Sewer interception systems have been built along rivers in rapidly urbanizing areas to collect unregulated sewage discharges due to misconnections between storm sewers and sanitary sewers. According to Chen et al. (2019), Interception System Overflows (ISOs) from different orifices in a sewer interception system might interact with each other, therefore pollutants from ISOs show a spatial variation. Their work focus on the understanding the spatial variation of pollutants for ISOs for informed decision making. Applying the framework to a real case study, it is demonstrated that ISO volumes and pollutant increase downstream and spatial variations are influenced by sizes and slopes of interceptors. Also contributions of runoff and sewer to ISO pollution vary from different types of pollutants and different locations. Sewer separation cannot significantly reduce pollutant loads from runoff.

Exfiltration is the leakage of wastewater out of a sewer system. Its impact can harm public health and the environment and require expensive repairs. An approach to locating defects includes supervising all features of the sewer system, there are diverse rehabilitation technologies available to correct exfiltration sources and these include: chemical grouting, cement grouting, sliplining, cured-in-place pipe, fold and form pipe, pipe bursting and point repair. Regular inspection of sewer infrastructure can help to identify exfiltration early so that steps can be taken to stop it and to repair the damage. Timely detection of exfiltration can minimize the environmental impact and scope the required intervention. A comprehensive evaluation of exfiltration from leaky sewer pipes has been done by Rutsch (2006). Several methods for determining exfiltration are thoroughly described and two approaches are considered for extensive real sewer network testing to evaluate their potential application to support cities and operators to define problem-oriented rehabilitation strategies.

Sewer contamination has been found in separated storm drain systems in urban areas during dry-weather flow. To determine whether transmission of sewer is occurring from leaking sanitary sewers directly to leaking separated storm drains, field experiments were performed in three different catchments by Sercu et al. (2011). Areas with high and low risks for sewer exfiltration into storm drains were identified. Rhodamine WT (RWT) were added to the sanitary sewers and monitored in manholes using optical probes set up for unattended continuous monitoring. RWT peaks were detected in high-risk areas and multiple locations of sewer contamination were found. This study provides direct evidence that leaking sanitary sewers can directly contaminate nearby leaking storm drains with untreated sewer during dry weather and suggests that chronic sanitary sewer leakage contributes to downstream faecal contamination.

Performance indicators

Indicators related to the grey infrastructure design are parameters, which are used to describe relevant properties of the system. The introduction of the Water Framework Directive and more specific of the combined approach set the need for new indicators to evaluate the performance of the sewer systems. In this case it is not more enough to consider only the classic indicators for the urban drainage, it is necessary to consider also parameters, which describe the impact of the urban drainage activities on the receiving waters (See, De Toffol, 2006).

According to the Commonwealth Scientific and Industrial Research (Vincent et al., 2007), Performance assessment is an essential part of management. Each organisation should have objectives and means to achieve them (Figure 2.2) and it is essential for it to assess how these objectives are fulfilled (effectiveness) and how it means and resources are utilised (efficiency).

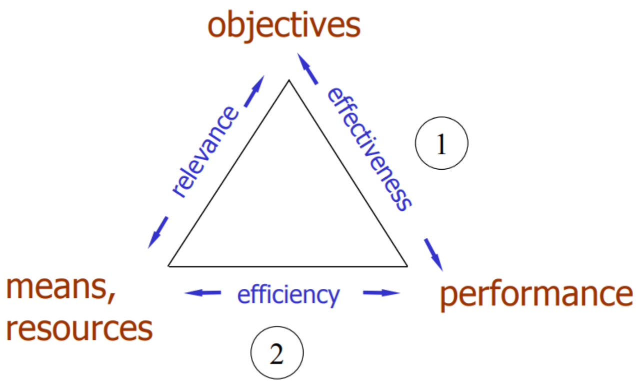


Figure 2.2. The management triangle for performance assessment (Vincent el al., 2007)

Performance can be defined and assessed from a variety of perspectives. In Figure 2.2 (1) Corresponds to the degree to which an organisation's products and services responds to the needs of their customers. (2) Is related to the degree with which the organisation uses the resources at its disposal both strategic and operational. Performance has to be assessed by using a set of indicators associated with criteria and target levels (Figure 2.2). Associations of parameters convert them into indicators. Criteria are bounds of indicator values used to classify the indicator values in acceptable or non-acceptable ranges. A nominal value and a range of deviation of acceptable values around the nominal value define target levels. The major challenge of Performance Assessment is not only defining the indicators but approving the criteria with target levels and ranges.

Environmental indicators are a useful tool to measure its impact. However, careful consideration must be given to develop a set of indicators in order to remove, plan or programme specific impacts. Donnelly et al. (2007) demonstrate the capability of a workshop-based approach to develop suitable criteria for selecting environmental indicators in a Strategic Environmental Assessment (SEA). A multi-disciplinary team was used in the approach with representatives from each of the four environmental fields, biodiversity, water, air and climatic factors, together with SEA experts, planning experts, academics and consultants. The results of this review together with original criteria were applied to the final agreed list. Some of the selected criteria includes, relevance to plan, ability to prioritise, and ability to identify conflict with other plan or SEA objectives.

A large number of human diseases are related to poor access to water and sewer systems, deficient solid waste management and insufficient storm water drainage. Rego et al. (2013) develop environmental sanitation indicators and classify sanitation conditions in specific sewer catchments and their respective neighbourhoods. The database used contains information on the sanitation components within the areas of sewer systems, urban drainage, water supply, building typology, road pavement, and public cleaning. Data was evaluated by using cluster analysis. The key variable of each component was identified and eight sewer catchments and twenty-three neighbourhoods were classified onto the categories of "good", "regular" and "poor". The use of environmental sanitation indicators allows decision makers to find critical areas and define priorities for improving environmental sanitation conditions.

As reported by Santos et al. (2018), over the past two decades, performance assessment based on performance indicators (PI) has been one of the areas showing the greatest advancement in the water sector. However, few projects and initiatives of performance assessment based on PI have been carried out with regards to urban drainage systems. For this reason, they present a state of the art to understand the applicability of the existing methodologies, their limitations and future directions. The main objective of the research was to develop a performance assessment framework with focus on urban drainage system including best practice recommendations. Performance was quantified through the application of performance metrics, such as PI, and classified based on reference values.

The performance assessment also considered different activity contexts, integrating various types of urban drainage systems including conventional, SUDS and its combination. Practical categories such as environmental, ecological, hydraulic, infrastructural, social and economic. The defined characteristics for the performance assessment framework fill the gaps in the field and facilitate its application by water utilities and municipalities.

2.2.2 Green infrastructure

Moving from hard engineering solutions (i.e. conventional piped sewer systems) to some devices in which stormwater will be drainage by natural means and there will be no need for storm sewers are referred to as Sustainable Urban Drainage Systems (SUDS) or Green Infrastructure (GI) (Woods-Ballard et al., 2015). This type of infrastructure has been referred differently in different parts of the world, see Ruangpan, et. al., (2020).

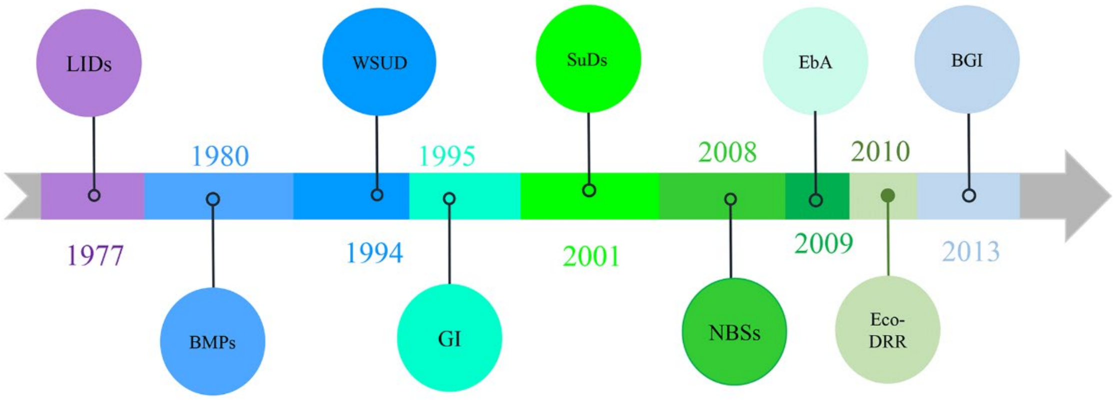


Figure 2.3. Timeline and year of origin of each term Low Impact Developments – LIDS, Best Management Practices – BMPs, Water Sensitive Urban Design (WSUD), Green Infrastructure (GI), Sustainable Urban Drainage Systems (SuDs), Nature-Based Solutions (NBSs), Ecosystem-based adaptation (EbA), Ecosystem-based Disaster Risk Reduction (Eco-DRR), Blue-Green Infrastructure (BGI) based on their appearance in publications (Ruangpan et al., 2020)

Sustainability can exist in the complex network of the urban drainage system. Implementation of sustainable design techniques will serve to produce a long-term viable drainage system. Surface water drainage systems should consider quality, quantity and amenity issues. SUDS are more sustainable than traditional systems because they:

- Control the flow rate of surface runoff, reducing the impact of urbanisation.
- Protect and / or enhance water quality.
- Give consideration to the natural environment and community needs.
- Create new wildlife habitats among the watercourses.
- Promote natural groundwater recharge.

This sustainable approach to urban drainage is an achievement as the systems aim to deal with surface runoff at the point of which it occurs and to manage potential pollution at its source. The introduction of SUDS into an area allows future development to take place in areas where the capacity of the traditional drainage system is full. The principle behind SUDS is to mimic natural drainage processes, remove pollutants and manage flood risk at source, while proving to be a significant contributor to increased biodiversity (Jensen et al., 2010).

SUDS / GI techniques

Techniques of SUDS are different and allow reaching different goals such as water quality improvement, flood protection, biodiversity increase or urban restoration. Depending on which goal need to be maximized, it is possible to use filters trench, vegetated swales, bioretention systems, ponds, and wetlands. Different SUDS allow to propose an approach to various implementation scales, which can be subdivided as follow:

- Open spaces
- Property lots
- Buildings
- Street and parking
- Retrofitting of existing green areas

Contrary to sewers which rely on centralised systems and typically have a lifetime of approximately 100 years, SUDS are primarily based on local or on-site management of urban stormwater runoff and can be implemented according to potentially changing rain patterns with relatively short notice. The systems may basically employ four physical mechanisms to control the quantities of water as follow (Ingvertsen, 2011):

(i) *Storage in ponds or basins*: The water can be slowly released back to the sewer system or to other SUDS. (ii) *Infiltration into the subsoil*: The water percolates to groundwater reservoirs or drain pipes leading to nearby surface water bodies. (iii) *Evaporation:* A fraction of the water from variety of SUDS will potentially leave the system as vapour, but no SUDS rely entirely on evaporation. Plants typically enhance evaporation through evapotranspiration. (iv) *Conveyance:* Transport the runoff between impervious surfaces and SUDS or between individual SUDS.

The SUDS approach to drainage includes a wide range of methods. As a consequence of this, there is no one correct drainage solution for a specific place. In the majority of occasions, a combination of techniques results in best practice. SUDS techniques can be divided into four categories: Control of rainwater at the source, Infiltration trenches and filter drains, swales and basins and ponds and wetlands. These techniques are described by Woods-Ballard et al. (2015) as follows:

Source Control

Green roofs

Green roofs are Multi-layered systems comprising of vegetation cover or landscaping above a drainage layer. The aim is to intercept and retain precipitation which then results in less surface runoff. The advantages and disadvantages of implementing green roofs are presented in Table 2.1.

Table 2.1. Advantages and disadvantages of implementing green roofs

Advantages:	**Disadvantages:**
Effectively remove pollutants.	More expensive than traditional runoff.
Suitable for high density developments.	Not suitable for steep roofs.
Ecological, aesthetic and amenity benefits.	Roof vegetation needs maintenance.
No land take necessary.	Waterproofing vital as is roof acts a sink.
Air quality improvement.	Provide building insulation.
	Sound absorbers.

Rainwater harvesting

Rainwater from impermeable surfaces is stored and used. The purpose of rainwater harvesting is to reuse water and reduce the rates of surface runoff. The advantages and disadvantages of implementing rainwater harvesting are presented in Table 2.2.

Table 2.2. Advantages and disadvantages of implementing rainwater harvesting

Advantages:	**Disadvantages:**
Control the flow of surface runoff.	Pollution risk.
Reduces the demand for mains water.	Underground storage tanks can be complex and costly.
Methods such as water butts are cheap.	Unsightly if storage is above ground.
Easy to install.	

Permeable Pavements

Provide a durable surface which allows surface water to infiltrate through the pavement and into the soil beneath. The water can be temporarily stored in the pavement before it is infiltrated or released into a drainage system. The advantages and disadvantages of implementing permeable pavements are presented in Table 2.3.

Table 2.3. Advantages and disadvantages of implementing permeable pavements

Advantages:	**Disadvantages:**
Remove pollutants.	Cannot be used in areas at risk of being swamped by large sediment loads.
Reduce the rates of runoff.	Untested in areas of high speed and large traffic volumes.
Low maintenance.	Ice prevention.
No land take.	Prevention of surface ponds.

Infiltration and Filtering

Infiltration trenches

An infiltration trench is a shallow, excavated channel that has been filled with stone aggregate to create an underground storage reservoir. The purpose of these trenches is to allow runoff to infiltrate into the ground as it enters the trench.

Filtration trenches / Drains

Filtration trenches or drains are similar in construction comparing to the infiltration trenches apart from a perforated pipe runs through the narrow channel. The purpose of this is to allow water to filtrate into the surrounding soil and into the pipe which then transfers the water to a disposal unit. The advantages and disadvantages of implementing infiltration trenches are presented in Table 2.4.

Table 2.4. Advantages and disadvantages of implementing infiltration trenches

Advantages:	**Disadvantages:**
Infiltration reduces runoff.	Blockages are common and difficult to find
Water pollution is reduced by filtration through the soil	Build-up of pollutants difficult to see.
Trenches can be built into the landscape.	Limited to small catchments.
	High replacement cost.

Swales and Basins

These variations on SUDS can be treated as features within the landscaped areas of a site or incorporated into ornamental pieces. The features can be installed as part of a drainage system connecting to either a pond or a wetland area.

Swales

A swale is a grassed area of depression which guides surface runoff overland from the source area to a storage or discharge system. Swales can be incorporated into the landscape as roadside kerbs avoiding alternative construction methods. Swales are wide, shallow like ditches which provide temporary storage, transport, treatment and the infiltration of surface water.

Basins

Basins are designed to hold back water for a few hours to ensure a settlement of solids. Basins are only temporarily in use, and outside of storm periods they remain dry. They provide short-term water storage and the settlement of solid ensures water filtration reducing contamination.

Ponds and Wetlands

Ponds and wetlands are natural ecosystems and in their original form are valuable parts of the drainage system. The construction of these permanent water bodies will contribute to visual amenity and biodiversity and can form an intrinsic link in a network of sustainable drainage systems. Ponds and wetlands can be designed with the potential to store various levels of water at different times. This makes them features which deal with surface runoff and enhance the flood-storage capacity for a particular drainage system. The use of existing wetlands or ponds for the treatment of surface water is unlikely to be acceptable, but this does not inhibit the creation of new features.

Selection of SUDS / GI

The surface water management train addresses the issue of drainage in stages, in conjunction with the processes occurring in a natural catchment. The process begins with the prevention on an individual basis and progresses to local and regional control. Surface runoff does not have to pass through all stages of the management train. The aim is to deal with the problem locally and then return the water at its source. SUDS are designed using the same principles of hydrology as traditional drainage systems, but different methods of application. Equal consideration must be given to the issues of quality, quantity and amenity resulting in a multi-disciplinary approach to drainage. SUDS are selected in accordance with the surface water management train. The preferred technique is to deal with surface runoff close to the source and to manage it locally (Woods-Ballard et al., 2015).

The management train recommends using a variety of techniques to deal with the issue of drainage. Drainage systems are part of a wider cycle of water and consideration of this is essential in terms of the development process. Site variation identifies the need for the usage of different types of SUDS. Site location, size and urban density all restrict the type of SUDS that can be implemented. The selection of drainage systems is not a clear-cut process and assessments of site capability must be considered in a wider context with a broader geographical focus rather than limited technical details. Source control is the preferred method of water resource management. The key to source control is prevention instead of mitigation, if hazards are not realised then risk does not have to be managed.

Green Infrastructure (GI) practices have been identified as a sustainable method of managing stormwater over the years. Due to the increasing popularity of GI as an integrated urban water management strategy, most of the current catchment modelling tools incorporate these practices, as built-in modules. Jayasooriya and Ng (2014) present a review of a selection of the most recent and popular modelling tools based on their accessibility. This review provides with the fundamental knowledge of different modelling tools which will assist with screening for a model according to the requirements from the number of tools available. Future research directions are presented on developing more comprehensive tools for GI modelling and recommendations.

A flexible framework has been created for modelling multi-dimensional hydrological and water quality processes within stormwater GI practices (See, Massoudieh et al. 2017). This approach conceptualizes GI practices using blocks (spatial features) and connectors (interfaces) representing functional components of a GI. The framework uses an implicit Newton-Raphson algorithm to solve equations representing the hydraulic, particle transport and transformation of water quality constituents. Four demonstration cases were presented showing applications for a variety of common-type GI practices but unique system implementations including a bioretention-based system, a permeable pavement system, a hypothetical infiltration basin, and a hypothetical water quality wet-pond interacting with groundwater. These demonstrations show the ability of the framework to effectively represent various processes affecting the flow as well as pollution transport and transformation within GI systems.

Wang et al. (2017) also presents a new framework for decision making in sustainable drainage system. It integrates resilience, hydraulic performance, pollution control, rainwater usage, energy analysis, greenhouse gas emissions and costs within 12 indicators. A multi-criteria analysis methods of entropy weight were selected to support scheme selection. This framework will help a decision maker to choose an appropriate design scheme for implementation without subjectivity.

Blue-green infrastructure (BGI) provides a wide range of ecosystem services (ES) and other benefits when managing stormwater, beyond flow and pollution control. Ashley et al. (2018) outlines the benefits of SUDS Tool, developed in the UK by the Construction Industry Research and Information Association (CIRIA) for valuing the benefits of BGI for stormwater measures. The tool has been applied to case studies across Europe. It includes a set of benefits based on ecosystem services applied to the use of BGI for stormwater management. Uncertainties in this multiple benefit assessment, detailing the processes used in the tool. The uncertainties when using BGI valuation tools help to inform the delivery of stormwater measures are demonstrated in this work as potentially of sufficient magnitude to justify explicit consideration by professionals and decision makers.

Evaluation of the effectiveness of green infrastructure (GI) practices on improving site hydrology and water quality and their associated cost could provide valuable information for decision makers when creating development/re-development strategies. Chen et al. (2019) present a catchment scale rainfall-runoff model to improve simulation of urban water management practices including GI practices. This model is capable of simulating in more detail impervious surfaces including sidewalks, roads, driveways and parking lots, conducting cost calculations for converting these impervious surfaces to porous pavements and selecting suitable areas for bioretention in the study area.

The effectiveness of GI practices on improving hydrology and water quality in a combined sewer overflow catchment was examined in eleven simulation scenarios using 8 practices. The total cost and the cost effectiveness for each scenario considering a 20-year practice lifetime were calculated. Results showed combined implementation of GI practices performed better than applying individual practices alone. Adoption levels and combinations of GI practices could potentially reduce runoff volume. Adding more practices did not necessarily achieve substantial runoff and pollutant reductions based on site characteristics. This enhanced model can be applied to different locations to support assessing the beneficial uses of GI practices.

2.3 Tools for planning and selecting urban drainage infrastructure

2.3.1 Models

Currently, drainage systems have received special consideration due to the concern in sustainable water resources and the attention on the interrelationships among the entire water cycle, environment and society. The importance of urban flood modelling has been recognized for strengthening human life. Particularly, modelling of urban drainage system focuses on the main and advanced topics including *water quality and water quantity, urban flood modelling, urban flood forecasting, modelling tools*, *risk-based analysis and socio-economic interactions.*

Modelling tools are used to predict the location, likelihood and impact of surface water flooding. As surface water flooding is not a regular occurrence which can be understood completely through observation, predictive models allow to understand where flooding could occur, how this might change under extreme conditions and help test the effectiveness of measures to manage the risk of surface water flooding.

Urban flood modelling and prediction refer to the processes of transformation of rainfall into a flood hydrograph, flood modelling generally involve approximate descriptions of the rainfall-runoff transformation processes. These descriptions are based on empirical, physically-based, or combined conceptual physically-based descriptions of the physical processes involved. In urban drainage, models are useful for various purposes such as overall assessment of floods, pollution, operational management, real time control and analysis of interactions among sub-system. The urban catchment system takes into account a detailed network with the ancillary elements design. The type of model applied depends on the goal of modelling, spatial coverage, data and technology availability.

Models in one dimension (1D) are normally used to simulate flow through defined geometries by computing the Saint-Venant equations (Abbott and Minns, 1998; Price and Vojinovic, 2011). These equations describe the evolution of the water depth and either the discharge or the mean flow velocity, representing the principles of conservation of mass (continuity equation) and momentum. 1D approach presents some limitations since there is no spatial relation to the surface making difficult the estimation of the damages at the buildings (Mark and Hoster, 2004). However, urban surface can be treated as a network of open channels and ponds connected to the pipe system normally associated as 1D-1D approach (Kuiry et al., 2010). The simulation process in the case of coupled 1D-2D modelling is based on numerical solution schemes for the computation of water levels, discharges and velocities. Channel and pipes network are connected with the flood flow model that consider the urban surface as a two-dimensional flow domain. The two domains 1D and 2D are normally coupled at grid cells over the channel computational points through mutual points of the connected cell and the channel section (Seyoum et al., 2012; Vojinovic and Tutulic, 2009).

2D models involve a lower degree of averaging fundamental hydraulic equations compared to 1D models, therefore it can be considered as a more realistic description of flow conditions. 1D-2D modelling is a good choice when it comes to extreme events and the urban surface is covered with excessive flood depths. Most surface water flooding models apply design rainfall profiles of known return period and a Digital Terrain Model (DTM) of the landscape. The rainfall is converted into runoff depending on the land use type (rural or urban) and then routed along flow pathways to low points where it may pond. Through examining the maximum extent and depth of flooding at each point in the modelled area we can determine the homes, businesses and infrastructure which are likely to be exposed to flooding.

DTM which represents the ground surface without any objects like plans and buildings are needed for the analysis of terrain topography, for setting up 2D models, processing model results, delineating flood hazards, producing flood maps, estimating damages, and evaluating various mitigation measures. Besides, a Digital Surface Model (DSM) which represents the earth´s surface and includes all objects on it. It includes buildings, vegetation and roads, as well as natural terrain features. It may be useful for landscape modelling, city modelling and visualization applications (Abdullah et al., 2011) .

The risk of surface water flooding has the potential to increase in the future due to different driving forces such as population growth and urban dynamics. It is not sustainable to depend on traditional sewerage and surface water infrastructure, to manage surface water flooding in UDS in the long term. Designing larger pipes and subsurface storage is expensive and sometimes not adaptable to extreme conditions. In terms of modelling, a holistic approach to drainage that takes account of all aspects to the UDS and produces long-term and sustainable actions should be implemented. This requires examination of the sources, pathways and receptors of flood waters to ensure that a full range of measures can be applied. Across the urban area and during any event, flows should be managed and adaptable in a way that will cause minimum harm to people, environment, buildings, and businesses.

The advance of Hydroinformatics have much to offer the water industry (Abbott, 1991; Abbott, 1999) and particularly urban storm and wastewater drainage. Price (2000) reviews aspects of data mining and knowledge discovery of large asset databases. The complementary nature of both physically based and data-driven modelling of drainage network performance, and the roles of decision support systems and knowledge management. It is claimed that the traditional way of pursuing urban drainage needs to change. The existence of engineering procedures and best management practices within urban drainage offers an effective basis for decision support, enhanced by tacit knowledge of engineers made explicit through the application of such decision support systems.

Over the last two decades, especially with the availability of the computer power and software has had an impact on the possibilities for practising engineers to study the hydrodynamic behaviour of drainage systems. Clemens (2001) propose methods related to the use of computer models in the field of urban drainage. This work discusses the relevant processes when modelling the hydrodynamics in a drainage system by combining different characteristics of a 1D model and measurements, with the aim to obtain an increased modelling accuracy and reliability.

Djordjevic et al. (1999) present an approach to rainfall runoff simulation in which the numerical model takes into account not only the flow through the sewer system, but also the flow on the surface. The numerical model simultaneously handles the full dynamic equations of flow through the sewer system and simplified equations of the surface flow. Chen et al. (2007) also developed an integrated numerical model for simulating the runoff processes in urban areas. A 1D model is used for calculating the rainfall-runoff hydrographs and the flow conditions in drainage networks. A 2D model is employed for routing flow on overland surface. Both models are solved by different numerical schemes and using different time steps with the flow through manholes adopted as model connections. Timing synchronisation between both models is taken onto account to guarantee suitable model linkages.

The development of cost-effective flood management strategies has become important for many cities, Vojinovic and Tutulic (2009) explores the difference analysis across irregular terrains. It is shown that in the case of terrains suited to exclusively 1D models the prediction of flow variables along the channel can be realistic. When it comes to the projection onto a 2D map, the representation of the terrain topography together with the mapping techniques that are employed introduce a limiting factor in their successful application. The results of this study provide users of numerical models with information that can be used to aid them in determining which tool to use and which aspects to consider in order to make more reliable analyses of flooding processes in urban areas.

Siekmann and Siekmann (2013), present options of an optimized area-management through a resilient urban drainage. The results of the study confirm that the combined usage of decentralized facilities for pluvial flooding is a first step to protect urban infrastructure. Decentralized facilities are more flexible than centralized and have a higher adaptation capacity which is needed considering the various effects of climate change. Dual drainage model was used to simulate combined decentralized rainwater management facilities for flooding purposes. A new resilience analysis was carried out by Mugume et al. (2015) to investigate the performance of an urban drainage system during pipe failure scenarios. The results indicate that the design strategy incorporating upstream distributed storage tanks minimises flood volume and mean duration of nodal flooding. When costs associated with failure are considered, resilient design strategies could prove to be more cost-effective over the design life.

As it was stated before, some of the major challenges in modelling rainfall-runoff in urbanised areas are the complex interaction between the sewer system and the overland surface, and the spatial heterogeneity of the urban key features. Leandro et al. (2016b) propose a methodology for considering the variability of building types and the spatial heterogeneity of land surfaces. It was shown that the ability to represent the urban key features spatial heterogeneity is clearly improved by the methodology developed. A rapid assessment of surface water flood management options in urban catchments has been presented by Webber et al. (2018). They propose a new framework for surface-water flood-intervention assessment at high resolution. The framework improves computational efficiency through utilisation of accessible data, simplified representations of interventions and a resource efficient cellular automata flood model. Results from the case study demonstrate that the framework is able to provide quantitative performance values for a range of interventions. The speed of analysis supports the application of the framework as a decision-making tool for urban water planning.

2.3.2 Optimisation techniques

Decision making can be regarded as knowledge process resulting in the selection of a course of action among several alternative scenarios. Every decision making process produces a final choice and the output can be an action or an opinion of choice (Reason, 1990). Providing effective decision support to improve planning, decision making and performance of water systems is of critical importance to water managers. There are many examples where the need of decision support for water-related problems has led to high environmental, social and economic costs to societies. Optimisation is a type of modelling which provides solutions to problems that concern themselves with the choice of a "best" configuration or a set of parameters that achieve some objective (Vojinovic et al., 2006; Savic, 2008).

In order to apply one of the optimisation methods, statement of the problem should be defined as well as a clear definition of the objectives and constraints, specified interrelationships between feasible regions for the solution, alternative courses of action. Afterward, a mathematical model has to be formulated: In decision making, preferences are defined using a function called the objective function. The decision maker´s problem can be presented as one of choosing an action among feasible decision variables that maximize or minimize the value of this function. The selection of decision variables can be subject to restrictions or constraints, and therefore, a set of models must also be defined representing such constraints (Price and Vojinovic, 2011).

Evolutionary optimisation

Evolutionary optimisation (EO) is a generic population-based metaheuristic algorithm that uses a population based approach in which more than one solution participates in an iteration and evolves a new population of solutions in each iteration (Goldberg, 1989) . The reasons for their popularity are many: (i) EO does not require any derivative information (ii) EO is relatively simple to implement and (iii) EO is flexible and have a wide-spread applicability (Deb, 2011). Evolutionary optimisation for single-objective is different from classical optimisation methodologies in the following main ways (Goldberg, 1989):

- An EO procedure does not usually use gradient information in its search process. Thus, EO methodologies are direct search procedures, allowing them to be applied to a wide variety of optimisation problems.
- An EO procedure uses more than one solution (a population approach) in an iteration, different from most classical optimisation algorithms which update one solution in each iteration (a point approach). The use of a population has a number of advantages: (i) it provides an EO with a parallel processing power achieving a computationally quick overall search, (ii) it allows an EO to find multiple optimal solutions, thereby facilitating the solution of multi-modal and multi-objective optimisation problems, and (iii) it provides an EO with the ability to normalize decision variables (as well as objective and constraint functions) within an evolving population using the population-best minimum and maximum values.
- An EO procedure uses stochastic operators, unlike deterministic operators used in most classical optimisation methods. The operators tend to achieve a desired effect by using higher probabilities towards desirable outcomes, as opposed to using predetermined and fixed transition rules. This allows an EO algorithm to negotiate multiple optima and other complexities better and provide them with a global perspective in their search.

Multi-objective optimisation problem

A multi-objective optimisation problem involves a number of objective functions which are to be either minimized or maximized. As in a single-objective optimisation problem, the multi-objective optimisation problem may contain a number of constraints which any feasible solution (including all optimal solutions) must satisfy. In the context of multi-objective optimisation, the extremist principle of finding the optimum solution cannot be applied to one objective alone, when the rest of the objectives are also important (Deb, 2011). Different solutions may introduce trade-offs (conflicting outcomes among objectives) among different objectives. A solution that is extreme (in a better sense) with respect to one objective requires a compromise in other objectives. This prohibits one to choose a solution which is optimal with respect to only one objective. This suggests two ideal goals of multi-objective optimisation:

- Find a set of solutions which lie on Pareto-optimal front, and

- Find a set of solutions which are diverse enough to represent the entire range of the Pareto-optimal front.

The multi-objective optimisation problems give increase to a set of Pareto-optimal solutions which need a further processing to arrive at a single preferred solution. To achieve the first task, it becomes quite a natural proposition to use an EO, because the use of population in an iteration helps an EO to simultaneously find multiple non-dominated solutions, which represents a trade-off among objectives, in a single simulation run.

Some applications in urban water systems

Flooding in urbanized areas has become a very important issue as the level of service or performance of urban water systems (UWS) degrades in time. To maintain an acceptable performance, successfully investigations have been carried out to aim at defining a framework to deal with multicriteria decision making in the context of UWS.

Multicriteria analysis methods have been used over the past decade for resolving environmental issues. Martin et al. (2007) presents an application of a multicriteria analysis (MCA) approach to urban drainage management. A French survey was undertaken to assess the performance of different best management practices (BMPs) at the national scale; results highlight the main reasons justifying the use of BMPs. The MCA results obtained allow ranking the various alternatives from best to worst, taking into account the different strategies adopted by the decision-makers involved. The development of a multicriteria approach could, in the future, serve as a supporting decision-aid tool, whose purpose would be to guide users in their choice of stormwater source solution.

Day-to-day water management is challenged by meteorological extremes, causing floods and droughts. Andel (2009), present the use of weather forecasts in operational water management. This work includes continuous simulation of weather forecasts and hydrological predictions with associated management actions for multiple years. This enables comparison and optimisation of alternative anticipatory management strategies. Monitoring networks provide data that is analysed to help managers make informed decisions about their water systems. Alfonso (2010), proposed innovative methods to design and evaluate monitoring networks. The main idea is to maximise the performance of water systems by optimising the information content that can be obtained form monitoring networks. It was done through the combination of models ans two theoretical concepts: Information theory and value of information.

Most sewerage networks are compiled of ageing assets that are becoming increasingly more susceptible to failure. Ward and Savic (2012), presented a methodology for the optimal specification of sewer rehabilitation investment. It builds on previously work which explored the application of multi-objective optimisation tools to assist engineers with the specification of optimal rehabilitation strategies. Through the introduction of a multi-objective optimisation tool to the problem, a unique methodology capable of quantifiably appraising optimal rehabilitation strategies was developed. Mendez et al. (2013) proposed automated parameter optimisation of a water distribution system (WDS). The aim of this study was to determine if model independent parameter estimator can be used to develop a fully calibrated extended-period model for a WDS using a simulation model. This approach is capable of fixing parameters based on a given sensitivity threshold.

Operation of existing flood control facilities is one the efficient method for urban drainage management. Schütze et al. (2002) proposed a new optimization methodology for urban detention pond operation. This work integrates an evolutionary algorithm with a simulation model to effectively manage detention storage capacities during flood periods. Optimal rule curves were compared with the current method of operation and show that the proposed method can decrease network flooding of the smallest and largest extreme rainfall events.

Reducing long computing time could be required during the optimisation processes. Parallel computing in multi-objective optimisation is an alternative to speed up the computation optimisation problem, algorithms specifically designed to run simultaneously on different processors are used. Three parallel strategies have been developed by Burger et al. (2009) in order to distribute the computations on several processor cores. The three strategies are: the flow parallel strategy, the pool pipeline strategy and the ordered pipeline strategy. These strategies have been developed with a view on the structure of urban drainage models and have been implemented within a specific simulation environment, called CityDrain3. This urban drainage software tool was used to demonstrate the runtime and speedup effects of the three strategies. A number of different urban drainage systems were analysed to detect shortcomings and limits of the strategies. The benchmark results reveal that the ordered pipeline strategy is capable of, at times significantly, reducing the runtime of all tested sewer systems.

Barreto (2012) introduce a framework to deal with multicriteria decision making for the rehabilitation of urban drainage systems. This work focuses on several aspects such as the improvement of the performance of the multicriteria optimisation through the inclusion of new features in the NSGA-II algorithm and the proper selection of performance criteria such as small parallelization. Parallel virtual machine (PVM) libraries were used for a newly developed algorithm in a master-slave approach. A small cluster composed by assorted PCs with single and multi-core processors was set up. Two case studies were tested on the parallel framework, one using a small network with 12 pipes and the other for a sub-catchment composed by 168 pipes. The results show a good performance saving between 60% and 80% of the computing time in comparison with a single-computer optimisation.

An alternative to satisfy the demand of computer resources is cloud computing with its inherent ability to exploit parallelism at many levels. Cloud computing has also become a fundamental new enabling technology to facilitate the access to computational capabilities for parallelism users. Vélez (2012), introduces a method named Model Based Design and Control (MoDeCo) for the optimum design of urban wastewater systems. It presents a detailed description of the integration of modelling tools for sewer systems, wastewater treatment plants and riers. This work also presents two alternatives to considerably reduce the computational demand when optimising integrated systems in practical applications such as the use of surrogate models and cloud computing. It provies an excellent tool for designers and managers of urban wastewater infrastructure.

Moya et al. (2013), considered the problem of studying how the flood inundation is influenced by uncertainties in water levels of the reservoirs in the catchment, and uncertainties in the digital elevation model (DEM) used in the 2D hydraulic model. Cloud computing was used through clusters on the basis of a number of office desktop computers. They show their efficiency and the considerable reduction of the required computer time for uncertainty analysis of complex models. The conducted experiments allowed to associate probabilities to different areas likely to flooding. Results offer an effective and efficient technology that makes uncertainty-aware modelling a practical possibility even when using complex models.

A flexible methodology that combines sampling techniques such as Monte Carlo – MC or Latin Hypercube – LH simulations, decision tree analysis and genetic algorithm optimisation is presented in Basupi and Kapelan (2013). The methodology gives flexible and optimal decisions as future water demand unfolds, the problem of optimal water distribution system (WDS) design under uncertain future water demand is formulate here as a multi-objective optimisation problem. The objectives include the minimisation of total intervention cost and maximization of WDS end resilience. The decision variables are the conventional design interventions and the water demand threshold values. The results show that there is value achieved by building flexibility in design when compared to the deterministic approach in the long-term planning of WDSs under uncertainty.

Some researchers have approached computing time reduction by using surrogate models (SM) which mimic the mechanistic model and is computationally less demanding. Ariestiwi (2013); Seyoum and Vojinovic (2008) and Vélez (2012) use SM for pipe network simulations which are validated against a distributed physically based models (DPM). The SM are set up by lumping the DPM network into compartments in which the volume of water is governed by mass balances. Downstream compartments discharge and surcharging are computed from explicit volume-discharge curves. The SM are applied on a 45 km^2 catchment. The number of simulated states and simulation times are reduced by approximately 3 and 6 orders of magnitude, respectively. Uncertainty of SM parameters was examined using the Generalized Likelihood Uncertainty Estimation (GLUE) methodology. Two different sampling methods were applied. Limits of acceptability for real-time control, warning and planning, resulted in many accepted models upstream and few to none in downstream backwater-prone areas. All applications showed SM uncertainty bands within expected uncertainty bands for DPM, supporting the use of a simpler conceptual model in fit-for-purpose modelling in urban water systems when computational demands of DPM are expensive.

2.4 Future drivers

The two main factors affecting runoff from local urban areas are rainfall and impermeable area connected to the drainage system, these influence the flooding source and pathways. In addition, the condition of the drainage system and its propensity to become blocked, either due to local factors or maintenance problems also locally influences the pathways. Urbanisation is dependent on development, either formal (planned) or informal, such as paving over of gardens, which generally falls within the uncontrolled area of permitted development. An example of a detailed review of future drivers of flood risk specifically in terms of SUDS can be found in Eckart (2012).

Rainfall intensity is increasing and the rate at which paved surfaces are being currently introduced in urban areas resulting in similar increases in the stormwater flows and volumes that can impact on existing drainage systems (Ashley et al., 2005; Noor et al., 2018) developed a methodological framework to update the rainfall intensity-duration-frequency (IDF) curves under climate change scenarios. A model output statistics (MOS) method is used to downscale the daily rainfall of general circulation models (GCMs), and an artificial neural network (ANN) is employed for the disaggregation of projected daily rainfall to hourly maximum rainfall, which is then used for the development of IDF curves. Finally, the 1st quartiles, medians, and 3rd quartiles of projected rainfall intensities are estimated for developing IDF curves with uncertainty level. The IDF curves developed in this study can be used for the planning of climate resilient urban water storm water management infrastructure.

According to Jean et al. (2018), combined sewer overflows (CSOs) origin environmental problems and health risks, but poor guidance exists on the use of rainfall data for sizing optimal CSO control solutions. They review available types of rainfall information as input for CSO modelling and assess the impacts of three rainfall data selection methods (continuous simulation, historical rainstorms selected based on rainfall depth or maximum intensity and IDF-derived storms) on the estimation of CSO volume thresholds in order to reach specific seasonal CSO frequency targets. It was found that the overflow structures local characteristics had a marginal influence on results obtained from continuous simulation compared to event-based simulation. The use of design rainfall events should thus be restricted to preliminary assessment of CSO volume thresholds, and the final volume estimation for solution sizing should be reviewed under continuous simulation.

Social values have continuing effects on UDS objectives. Social, ecological and cultural demands could achieve importance as the increasing public knowledge of the sustainability requirements. The understanding on how urbanisation intervenes with flow patterns is necessary to develop strategies for stormwater management, urban floods control and urban development standards (Gomes et al., 2012). Changing legal requirements could have also consequences for the objectives of SUDS, for this reason, urban drainage planning has to consider a broad set of aspects and has to be integrated with land use policy, city planning, building code and all the related legislation (Furlong et al., 2018).

The general economic development and its consequences for the funding of infrastructure providers will also influence the objectives of SUDS. Financial crisis of the public bodies could result in trends toward increased privatization of the infrastructure or in decreased standards for the design and operation of infrastructure in order to reduce costs. The duration, intensity and frequency of rainfall events are affected by long-term fundamental alterations in climate patterns. The availability of rainfall information with high spatial resolution is of fundamental importance in many applications in the field of water resources (Corzo et al., 2009; Rodriguez et al., 2012; Li et al., 2013; Alvis et al., 2016). There is a significant difference between what can be provided by climate science and what is required for the design of infrastructure systems (Hallegatte, 2009).

Different drivers of the social system environment contribute to the influences of pollution loads on drainage systems, the pollution load of runoff is influenced by the land-use of the catchment area (Martínez et al., 2014a). Increases in the volume of traffic as well as developments in traffic technologies will also influence the pollution loads of urban runoff in considerable ways. The pollution load of urban runoff is also affected by general air pollution, to prevent this, the future developments of air pollution should be considered. In addition, the pollution load is influenced variously by medium to long-term developments, these structural uncertainties cannot be reduced, and they can be described using future scenarios (Galvis et al., 2014; Martínez-Cano et al. 2014a, Galvis, 2019).

Spatial developments in cities also have experienced extensive urban land expansion that has created various negative impacts on the urban ecosystem. The drivers behind the suburbanization are social processes like the growth of population, changing requirements on housing and the increase of the mean living space demand per person. Local developments in single development sites also affect the size of paved surfaces and in the local level is then associated with a number of its own uncertainties. The interactions of the different global and local spatial developments are associated with uncertainties, in most cities trends can be derived by population decrease in one site and population increase in another site. The spatial development trends are associated with medium to long-term uncertainties and could be described by future scenarios (Sanchez, 2013; Sanchez et al., 2014).

Driving forces in relation of the hydraulic performance can occur during the operation process of the system. During operation, there is a danger of failure – bottlenecks in ditches caused by waste, plants, sediments, etc. such bottlenecks occur accidentally, but the danger could be reduced through regular control and maintenance. Furthermore, there are problems with the siltation of infiltration facilities. These problems could be reduced by constructional measures like sedimentation facilities or filtration via the vegetated topsoil or with management measures like restricting the use of the infiltration areas to prevent the compaction of the soil and regular maintenance. Uncertainties for hydraulic performance can therefore be reduced significantly by regular control and maintenance. The standards for maintenance depend on the business model for the operation of the urban drainage system (EPA, 2016b; Lepot et al., 2016).

3

ALLOCATING GREEN INFRASTRUCTURE FOR URBAN RUNOFF AND POLLUTANT REDUCTION

Green infrastructure (GI) has been regarded as an effective intervention for urban runoff reduction. Despite the growing interest in GI, the technical knowledge needed to demonstrate their advantages, cost and performance in reducing runoff and pollutants is still under research. The present Chapter describes a framework that aims to obtain the optimal configuration of GI (i.e. the optimal number of units distributed within the catchment) for urban runoff reduction. The research includes an assessment of the performance of GI measures dealing with pollution load, peak runoff and flood volume reduction. The methodological framework developed includes: 1) data input, 2) GI selection and placement, 3) hydraulic and water quality modelling, and 4) assessing optimal GI measures. The framework was applied in a highly urbanized catchment in Cali, Colombia. The results suggest that if the type of GI measure and its number of units are taken into account within the optimisation process, it is possible to achieve optimal solutions to reduce the proposed reduction objectives with a lower investment cost. In addition, the results also indicate a pollution load, peak runoff and flood volume reduction for different return periods of at least 33%, 28% and 60%, respectively. This approach could assist water managers and their stakeholders to assess the trade-offs between different GI.

Based on: Martínez, C., Sanchez, A., Galindo, R., Mulugeta, A., Vojinovic, Z., Galvis A. (2018). Configuring Green Infrastructure for Urban Runoff and Pollutant Reduction Using an Optimal Number of Units. *Water* 10(11) 1528 – 1548. doi: 10.3390/w10111528

3.1 Introduction

There is a growing interest in urban runoff processes since more than half of the world's population live in urban areas. As a result, urban water researchers and practitioners are increasingly concerned with how to manage urban runoff with the effort to maintain more water on-site and to replicate natural hydrological processes. Urban runoff is generated when rainfall flows over land or impervious surfaces, such as paved streets, parking lots and rooftops and does not seep into the ground. Apart from direct damage, heavy rainfall can also lead to a sequence of cascading events such as power interruptions, traffic congestion, business interruptions and pollution of water bodies (Hilly et al., 2018).

In the past, the urban runoff control was focused on efficient surface drainage and flood control for a given return period rainfall event that was often of a larger magnitude (Ashley et al., 2005). However, researchers and practitioners are becoming increasingly concerned with the runoff resulting from smaller and more frequently occurring rainfall events that can cause a sequence of negative effects in urban areas and receiving waters (Mailhot and Duchensne, 2010).

The main pollutants found in runoff come from fertilizers (nutrients), humans and animals (bacteria), chemicals (pesticides), roofs and roads (metals) and from vehicles (hydrocarbons) (Department of Environmental Conservation – NY, 2018) . Suspended sediments constitute the largest mass of pollutant loadings to receiving waters from urban areas and is generally conveyed by urban drainage as non-point pollution (Novotny, 2003). Polluted urban drainage runoff can be harmful to plants, animals and people and its quality was largely ignored in the design of urban drainage systems until approximately 1980 (EPA, 2016a).

Green Infrastructure (GI) is an attractive option for urban water managers as it has the potential to provide a range of benefits and co-benefits. If carefully designed and implemented, GI can be effective in dealing with problems associated with floods and droughts as well as with poor urban runoff quality (Yang and Li, 2013; Ozgun et al., 2017). However, retrofitting GIs in long-established urban areas can be a technically very challenging and costly task (Stovin et al., 2007). Previous studies aiming to evaluate the performance of GI in urban water systems have been carried out by: (i) modelling tools for stormwater management and the economics of GI practices (Jayasooriya, 2014), (ii) evaluating the importance of GI in small and medium-sized towns (Shackleton et al., 2016), (iii) examining the performance of vegetative swales to improve runoff in an urban area with moderate traffic (Leroy et al., 2016), (iv) proposing a flexible modelling framework for hydraulic and water quality performance assessment of stormwater GI (Massoudieh et al., 2017), (v) combining ecosystem services with a cost-benefit analysis for selection of green and grey infrastructures for flood protection in a cultural heritage (Vojinovic et al., 2017), and (vi) combining co-benefits and stakeholders' perceptions into the GI selection for flood risk reduction (Alves et al., 2018). The results from these studies have shown a good potential for application of GI in urban water management.

Numerical models have proved to be invaluable for modelling flows in urban areas (Vojinovic et al., 2006; Vojinovic et al., 2012), while multi-objective optimisation can provide useful support in decision-making processes. In addition, the combination of numerical models and optimisation tools such as the NSGA-II optimiser has proved to be particularly useful for dealing with stormwater-related issues (Barreto et al., 2010; Vojinovic et al., 2014; Martínez et al., 2018). The objective of the NSGA-II algorithm is to improve the adaptive fit of a population of candidate solutions to a Pareto front constrained by a set of objective functions. The algorithm uses an evolutionary process with surrogates for evolutionary operators including selection, genetic crossover, and genetic mutation. The population is sorted into a hierarchy of sub-populations based on the ordering of Pareto dominance. Similarity between members of each sub-group is evaluated on the Pareto front, and the resulting groups and similarity measures are used to promote a diverse front of non-dominated solutions (Deb et al., 2002).

Investigations where NSGA-II have been used to optimise GI have focused on: (i) multiobjective optimisation for combined quality-quantity urban runoff control (Oraei et al., 2012), (ii) selecting an optimal sustainable drainage design for urban runoff reduction (Galindo-Calderón et al., 2015), (iii) proposing an evolutionary and holistic assessment of green-grey infrastructure for CSO reduction (Alves et al., 2016). Other multiobjective evolutionary algorithms are also used to optimise GI as in the case of (iv) minimization of cost, sediment load and sensitivity to climate change in a watershed management application (Chichakly et al., 2013) , (v) optimal selection and placement of green infrastructure to reduce the impacts of land use change and climate change on hydrology and water quality (Liu et al., 2016), (vi) optimal sizing of GI treatment trains (i.e. a sequence of multiple stormwater treatments) for stormwater management (Jayasooriya et al., 2016) and (vii) a quantitative modelling framework to support decision making in Sustainable urban Drainage Systems (SuDS) design alternatives (Wang et al., 2017). The above approaches have produced promising results and may become a useful tool for planning and decision making of drainage systems.

Based on the previous studies, the benefits of applying GI measures (or practices) are well known. However, currently available methodologies are more focused on the optimal coverage area of GI instead of a GI-type preference. The key advantage of the present approach is that the number of equal GI-size units redistributed within the subcatchment has been taken into account within the optimisation process and this is presented in more detail in the subsequent sections.

The present chapter provides a novel approach which aims to configure GI for urban runoff and pollutant reduction using the optimal number of units. The research includes an assessment of the performance of GI measures when dealing with two main objectives: environmental (i.e. pollution load, peak runoff, flood volume) and economic (i.e. investment costs). This proposed framework has been implemented in the coding environment LAZARUS (a free source Delphi compatible with cross-platform IDE for rapid application development) in order to couple a hydrodynamic model with an optimisation algorithm. This coupled model searches for optimal GI units that can achieve runoff reduction, better runoff quality and least investment costs. The potential of this method has been demonstrated in a real-life case where different GI units were evaluated considering the environmental and economic objectives.

3.2 Case Study

An urban area within the Meléndez catchment in Cali (Colombia) has been used to demonstrate the proposed framework. The catchment area encompasses 46.2 km^2 and the river network has a length of 7 km which receives discharges from both sewer pipes and open channels. Since the 1990s there have been floods and pollution due to river overflows along its entire length that have affected highly urbanized sectors. The area that is divided into the south and southwest has a higher population density. The total area can be delineated into 25 subcatchments, and the drainage system is formed by one river, 22 open channel sections, 21 box culvers and 51 circular pipes. The surface runoff flows by gravity from west to east of the catchment to reach the outfall in the southern channel. According to CVC (2000), the estimated time of concentration is approximately 84 minutes. The urban catchment has one outflow point near to the Cauca river, which is the main source of drinking water for Cali. The river receives an average discharge of 0.51 m3/s per day from the Meléndez catchment (EMCALI, 2003). Figure 3.1 shows the urban drainage catchment of the study area.

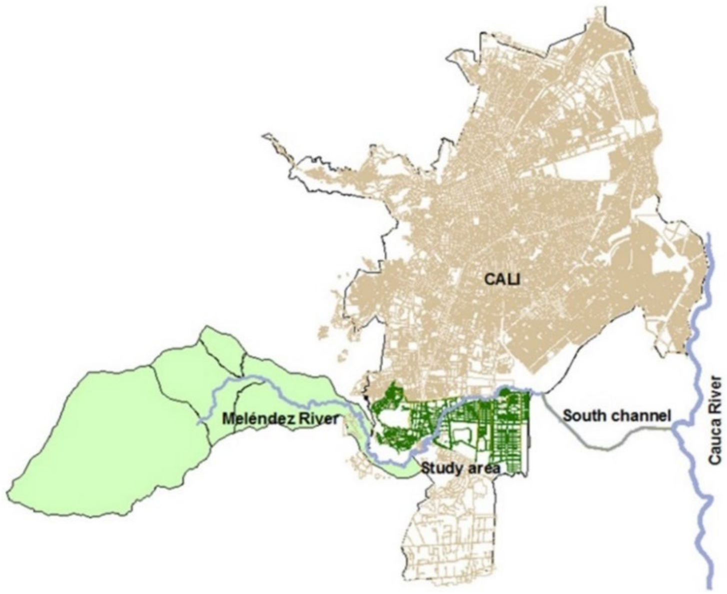


Figure 3.1. Location of the Meléndez catchment area within the city of Cali.

3.3 Methodological framework

The methodological framework for optimal system configuration of GIs consists of four main steps: 1) data input, 2) GI selection and placement, 3) hydraulic and water quality modelling, and 4) assessing optimal GI measures. These steps are described in detail in the following subsections.

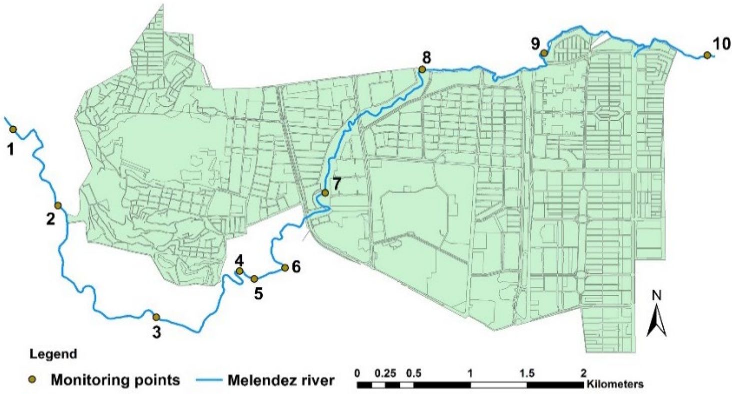
3.3.1 Data input

The data used in this study includes the network layout, conduits, canals, manholes, outflow and subcatchments with their hydrological parameters and dry weather flow characteristics. Subcatchment parameters include outlet nodes, percentages of previous and impervious areas, terrain slope, depression storage and soil infiltration characteristics. The present work also used contour maps, a Digital Elevation Model (DEM), land use, the percentage of impervious areas, soil type, urban land use, roads, streams, the groundwater table depth, and land ownership. Rainfall is introduced into the model with hyetographs for 2, 5, 10, 20 and 50-year return period events based on intensity, duration and frequency. Water quality data obtained from the DAGMA project (DAGMA and Univalle, 2004) was used for build-up and wash-off model simulations.

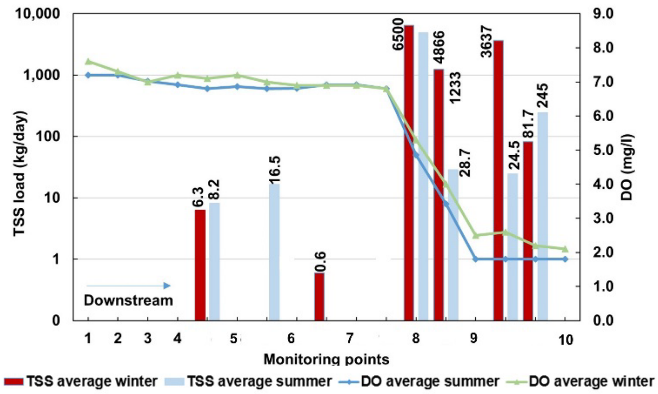
Figure 3.2a depicts the geographical location of the monitoring points. Point 1 was selected as a reference point in order to determine the status of the water quality of the river before it reaches the urban area. Two stations were taken into account in the rural area (i.e. points 2 and 3) with the purpose of evaluating the impact from coal mines. In total 10 monitoring points were monitored to evaluate the water quality of the river. The maximum daily Total Suspended Solids (TSS) measured was approximately 6,500 kg/day and a minimum Dissolved Oxygen (DO) of 2 mg/l (see Figure 3.2b).

In order to represent the TSS loads for each subcatchment for a 10-year return period and for validation of the model purposes, different input concentrations were specified at different nodes of the drainage network by a trial-and-error procedure in accordance with the data reported in (Galvis et al., 2008; Galvis et al., 2018). TSS was also introduced into the model at specific nodes representing the combined sewer in the conveyance system. This makes it possible to represent the pollutant concentration coming from the dry weather flow – DWF (i.e. assumed DWF diurnal pattern). After assigning different TSS concentrations into the system, the modelled Meléndez river produced a TSS-based flow concentration of 10 mg/l comparable to the TSS profile measured and presented in Figure 3.2c. This value was placed as a boundary condition in the model.

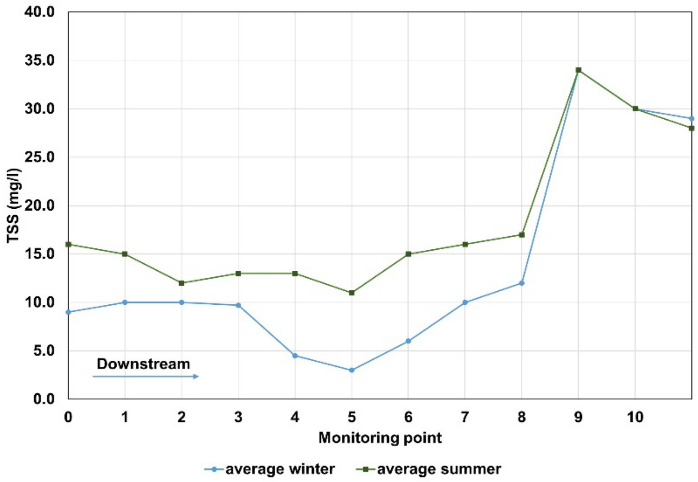
In terms of the costs concerning different GI measures, a catalogue with different costs was compiled. This catalogue contains unit costs for different GI types (see also the summary table of GI in Table 3.3).



(a) WQ monitoring points



(b) Measured TSS (kg/day) and DO (mg/l)



(c) TSS concentration profile

(DAGMA and Univalle, 2004).

Figure 3.2. Water quality data in the Meléndez river

3.3.2 GI selection and placement

GI Selection

The different GIs were selected based on their characteristics and suitability for implementation in the urban area. The selection is based on the requirements concerning land use, applicability on the impervious area and treatment capacity. The GIs were implemented within each subcatchment by defining the number of units and surface area for each type. The selected GI includes Bio-Retention cells (BR) for expanding the green space in the subcatchments, Infiltration Trenches (IT) for using around playing fields and recreational areas, Porous Pavement (PP) to reduce storm water runoff and Vegetative Swales (VS) with the purpose of improving the water quality.

Table 3.1 presents the characteristics of the GI design following the recommended values given in (Shoemaker et al., 2013). The BR were selected with the minimum recommended dimensions (4.6 m wide by 12.2 meters in length). The minimum width allows the BR to control the effects of runoff pollutants while the minimum length enables the cell to accommodate the distributed flow by decreasing the changes of concentrated flow. The ponding depth was set to 120 millimetres to give a sufficient water storage capacity and a vegetative fraction was set to 0.05. Three types of BR (BR01, BR02 and BR03) were used according to the soil characteristics and either to drain out by infiltration or underdrains. Two types of IT (IT01 and IT02) were used to cover from 2 to 4 ha drainage area, draining within 24 hours and with a storage depth of 1,400 mm. The seepage rate was set depending on the soil class. PP (PP01) was chosen to infiltrate water through the soil so that there is no vegetative cover on top. VS (VS01) was also selected based on the 2 year-return period with high vegetative cover. The swale side slope was set to be 5% and covered by dense vegetation, usually grass, to slow down flows and to trap pollutants (Rossman and Huber, 2016). Table 3.2 presents a summary table with the detailed information of the selected GI for the study area.

Table 3.1. Characteristics of the GI design (configuration parameters)

Layer	Property	Units	BR01	BR02	BR03	IT01	IT02	VS01	PP01
Surface	Berm height	mm	120	120	120	200	200	900	5
	Vegetation volun	fraction	0.05	0.05	0.05	0	0	0.15	0
	Surface roughnes	Manning n	0	0.001	0.001	0.25	0.25	0.40	0.012
	Surface slope		0	0.5	0.5	0.5	0.5	2	1
	Swale side slope	(run / rise)						5	
Soil	Thickness	mm	800	900	600	-	-	-	0
	Porosity	Volume fracti	0.453	0.453	0.43	-	-	-	0.5
	Field capacity	Volume fracti	0.212	0.144	0.1	-	-	-	0.2
	Wilting point	Volume fracti	0.109	0.058	0.047	-	-	-	0.1
	Conductivity	mm/h	14.54	3.42	2.7	-	-	-	0.5
	Conductivity slop		7	7	5	-	-	-	10
	Suction head	mm	4.33	4	2	-	-	-	3.5
Storage	Thickness	mm	800	300	300	1400	1400	-	150
	Void ratio	Voids / solids	0.47	0.75	0.75	0.47	0.47	-	0.47
	Seepage rate	mm/h	7.27	18.79	0	7.27	18.79	-	18.79
	Clogging factor		0	0	0	0	0	-	0
Drain	Flow coefficient	mm/hour	0	0	2.66	0	0	-	1.02
	Flow exponent	fraction	0.5	0	0.5	0	0	-	0.5
	Offset height	mm	6	0	50	0	0	-	10
	Thickness	mm	-	-	-	-	-	-	150
	Void ratio	Voids / solids	-	-	-	-	-	-	0.15
Pavement	Impervious Surf.	Fraction	-	-	-	-	-	-	0
	Permeability	mm/h	-	-	-	-	-	-	3400
	Clogging factor		-	-	-	-	-	-	0

Table 3.2. Summary table of the implemented GI for Meléndez Catchment, Cali - Colombia.

Sub Catchment	GI Type	Area (Ha)	% Imper	Drainage Area (Ha)	Flow (m^3/hr)	Volume (m^3)	Size Depth (m)	Width (m)	Unit Area (m^2)	GI # Units	% Imper Area Treated	GI Unit Cost	GI Total Cost
1		126.58	0.72										
	BR03			2	202	4858	1.6		3036	25	54.76	$27,439	$685,726
	IT02			2	202	4858	1.4	7.5	3470	16	35.05	$30,812	$308,119
	VS01				51	1214	1.6	10	759	16	8.76	$23,966	$383,448
2		65.08	0.76										
	BR02			2	212	5099	1.6		3187	10	40.59	$32,122	$321,224
	IT02			2	212	5099	1.4	7.5	3642	10	40.59	$30,812	$308,119
	VS01			0.5	53	1275	1.6	10	797	13	13.19	$23,966	$311,552
	PP01			0.054				6	540	16	1.75	$122,812	$1,964,990
3		39.58	0.72										
	BR02			2	202	4849	1.6		3030	5	35.09	$32,122	$160,612
	IT02			2	202	4849	1.4	7.5	3463	5	35.09	$30,812	$154,059
	VS01			0.5	51	1212	1.6	10	758	15	26.32	$23,966	$359,483
4		29.03	0.76										
	BR02			2	212	5095	1.6		3185	5	45.53	$32,122	$160,612
	IT01			2	212	5095	1.4	7.5	3639	4	36.42	$30,182	$123,248
	VS01			0.5	53	1274	1.6	10	796	7	15.94	$23,966	$167,759
	PP01			0.054				6	540	8	1.97	$122,182	$982,495
5		97.57	0.11										
	BR03			2	31	736	1.6		460	2	37.51	$27,429	$54,858
	IT01			2	31	736	1.4	7.5	526	2	37.51	$30,812	$61,624
	VS01			0.5	8	184	1.6	10	115	5	23.44	$23,966	$119,828
6		41.5	0.67										
	BR02			2	188	4508	1.6		2818	4	28.80	$32,122	$128,490
	IT02			2	188	4508	1.4	7.5	3220	6	43.20	$30,812	$184,871
	VS01			0.5	47	1127	1.6	10	704	13	23.40	$23,966	$311,552
	PP01			0.054				6	540	16	3.11	$122,812	$1,964,990
7		58.65	0.76										
	BR02			2	214	5125	1.6		3203	12	53.77	$32,122	$385,469
	IT02			2	214	5125	1.4	7.5	3661	8	35.85	$29,508	$246,495
	VS01			0.5	53	1281	1.6	10	801	8	8.96	$23,965	$191,724

Table 3.2 Cont.

Sub Catchment	GI Type	Area (Ha)	% Imper	Drainage Area (Ha)	Flow (m^3/hr)	Volume (m^3)	Size Depth (m)	Width (m)	Unit Area (m^2)	GI # Units	% Imper Area Treated	GI Unit Cost	GI Total Cost
8		31.57	0.76										
	BR02			2	212	5090	1.6		3181	5	41.91	$32,122	$160,612
	IT02			2	212	5090	1.4	7.5	3636	4	33.53	$30,812	$118,033
	VS01			0.5	53	1272	1.6	10	795	11	23.05	$23,966	$263,615
9		61.32	0.81										
	BR01			2	228	5474	1.6		3421	11	44.14	$32,122	$353,347
	IT01			2	228	5474	1.4	7.5	3910	8	32.10	$30,812	$246,495
	VS01			0.5	57	1369	1.6	10	855	23	23.07	$23,966	$551,207
10		87.57	0.80										
	BR01			2	224	5388	1.6		3367	8	22.84	$31,122	$256,979
	IT01			2	224	5388	1.4	7.5	3848	18	51.39	$30,811	$554,602
	VS01			0.5	56	1347	1.6	10	842	13	9.28	$23,966	$311,552
	PP01			0.054				6	540	14	1.08	$122,812	$1,719,366
11		68.76	0.10										
	BR01			2	29	694	1.6		434	1	28.21	$32,122	$32,122
	IT01			2	29	694	1.4	7.5	496	2	56.42	$30,812	$61,624
	VS01			0.5	7	174	1.6	10	108	2	14.11	$23,966	$47,931
12		34.91	0.30										
	BR02			2	84	2020	1.6		1263	2	38.19	$32,122	$64,245
	IT02			2	84	2020	1.4	7.5	1443	2	38.19	$30,812	$61,624
	VS01			0.5	21	505	1.6	10	316	4	19.10	$23,966	$95,862
	PP01			0.054				6	540	6	3.09	$122,812	$736,871
13		32.45	0.77										
	BR02			2	215	5159	1.6		3225	5	40.23	$32,122	$160,612
	IT02			2	215	5159	1.4	7.5	3685	6	48.27	$30,812	$184,871
	VS01			0.5	54	1290	1.6	10	806	5	10.06	$23,966	$119,828
14		37.56	0.67										
	BR02			2	189	4526	1.6		2828	5	39.62	$32,122	$160,612
	IT02			2	189	4526	1.4	7.5	3233	5	39.62	$30,812	$154,059
	VS01			0.5	47	1131	1.6	10	707	10	19.81	$23,966	$239,655
	PP01			0.054				6	540	4	0.86	$122,812	$491,248

Table 3.2. Cont.

Sub Catchment	GI Type	Area (Ha)	% Imper	Drainage Area (Ha)	Flow (m^3/hr)	Volume (m^3)	Size Depth (m)	Width (m)	Unit Area (m^2)	GI # Units	% Imper Area Treated	GI Unit Cost	GI Total Cost
15		8.5	0.82										
	BR01			2	231	5546	1.6		3466	2	57.14	$32,122	$64,245
	IT01			2	231	5546	1.4	7.5	3961	1	28.57	$30,812	$30,812
	VS01			0.5	58	1386	1.6	10	867	2	14.29	$23,966	$47,931
16		27.2	0.75										
	BR02			2	210	5039	1.6		3150	4	39.30	$32,122	$128,490
	IT02			2	210	5039	1.4	7.5	3600	5	49.13	$30,812	$30,812
	VS01			0.5	52	1260	1.6	10	787	4	9.83	$23,966	$47,931
17		9.32	0.48										
	BR02			2	135	3237	1.6		2023	1	44.64	$32,122	$32,122
	IT02			2	135	3237	1.4	7.5	2312	1	44.64	$30,812	$30,812
18		25.7	0.77										
	BR02			2	217	5217	1.6		3261	4	40.18	$32,122	$128,490
	IT02			2	217	5217	1.4	7.5	3727	5	50.23	$30,812	$154,059
	VS01			0.5	54	1304	1.6	10	815	3	7.53	$23,966	$71,897
19		40.72	0.78										
	BR02			2	218	5225	1.6		3265	5	31.66	$32,122	$160,612
	IT02			2	218	5225	1.4	7.5	3732	8	50.65	$30,812	$246,495
	VS01			0.5	54	1306	1.6	10	816	10	15.83	$23,966	$239,655
	PP01			0.054				6	540	8	1.37	$122,812	$982,495

Total: $19,941,227

GI Placement

Suitable locations for GI placement were obtained by using the best management practices tool, i.e., Siting tool (Shoemaker et al., 2013). This tool identifies potential suitable locations/areas for implementing all types of GIs proposed. It supports users with selecting suitable locations that meet the defined site by considering urban land use, location of streams, soil classification, land ownership and impervious layers. The Siting tool does not consider other constraints such as geological appearance, or the socio-economic or political situation. The main output from this tool is the identification of suitable locations for different GIs.

3.3.3 Hydraulic and water quality modelling

The next step involved setting up a hydrodynamic model to simulate quantity and quality as well as the hydrologic and hydraulic routing of urban runoff. This was done in the Storm Water Management Model - SWMM. SWMM solves the Saint-Venant equations which govern the unsteady flow of water through a drainage network of channels and pipes by converting the equations into an explicit set of finite-difference equations (Rossman, 2017). The quantity model calibration was originally undertaken in the studies of (Martínez-Cano et al., 2014a; Alvis et al., 2016). The mass of a pollutant transported during a storm event has also been modelled in SWMM as a coupled build-up and wash-off process providing stormwater pollutant load generated from the urban catchment. In the present work, build-up was computed using Equation 3.1 in order to describe pollutant build-up over time (Rossman and Huber, 2016).

$$b = Min(B_{max}, K_B\ t^{N_B}) \tag{3.1}$$

Where, b is the pollutant build-up (kg m^{-1}), t is the build-up time interval in days, B_{max} is the maximum build-up possible (kg m^{-1}), K_B is the build-up rate constant (kg/mday^{-NB})

and N_B is the build-up time exponent (dimensionless). The time exponent, N_B, should be ≤ 1 so that a decreasing rate of build-up occurs as time increases. When N_B is set equal to 1, a linear build-up function is obtained. Wash-off is the process of dissolving the constituents from catchment surface during the period of runoff. In this work, an event mean concentration (EMC) wash-off function has been applied according to Rossman and Huber (2016) and presented in Equation 3.2.

$$w = Kw\, q\, fluA \tag{3.2}$$

Where, Kw is the EMC expressed in the same volumetric units as flow rate. $qfluA$ is the fraction of the total runoff rate that applies to the land use. Most of the urbanized areas for this case study are covered by highly populated residential land use followed by commercial, industry, park and road zones. Total Suspended Solids (TSS) has been chosen as an indicator of the runoff quality. TSS is one of the basic indicators of urban runoff pollution as some of the nutrients and metals are transported attached to the particles.

3.3.4 Assessing optimal green infrastructure

Optimisation Procedure

The optimisation procedure consists of adjusting the number of GI units, taking into account their location within the catchment area. For this purpose, the NSGA-II optimiser algorithm developed by Deb et al. (2002) was used with the goal of finding a representative set of optimal Pareto solutions and to quantify the trade-offs between pollution load, peak runoff, flood volume and investment cost. Figure 3.3 illustrates the proposed optimisation procedure. The optimisation steps include: (1) computation of the initial value of variables, in this case the maximum number of GI units and its maximum costs, (2) the initial simulation of the hydraulic and quality model, (3) computation of the maximum values of pollution load, peak runoff and flooding volume for different return periods of rainfall, (4) computation of the objective functions, (5) running the optimiser NSGA-II according to the number of populations and generations, and (6) updating the hydrodynamic input file by changing the number of equal size units of the GI (e.g., the number of bio-retention cells) deployed in each subcatchment if the number of populations and generations has not been reached.

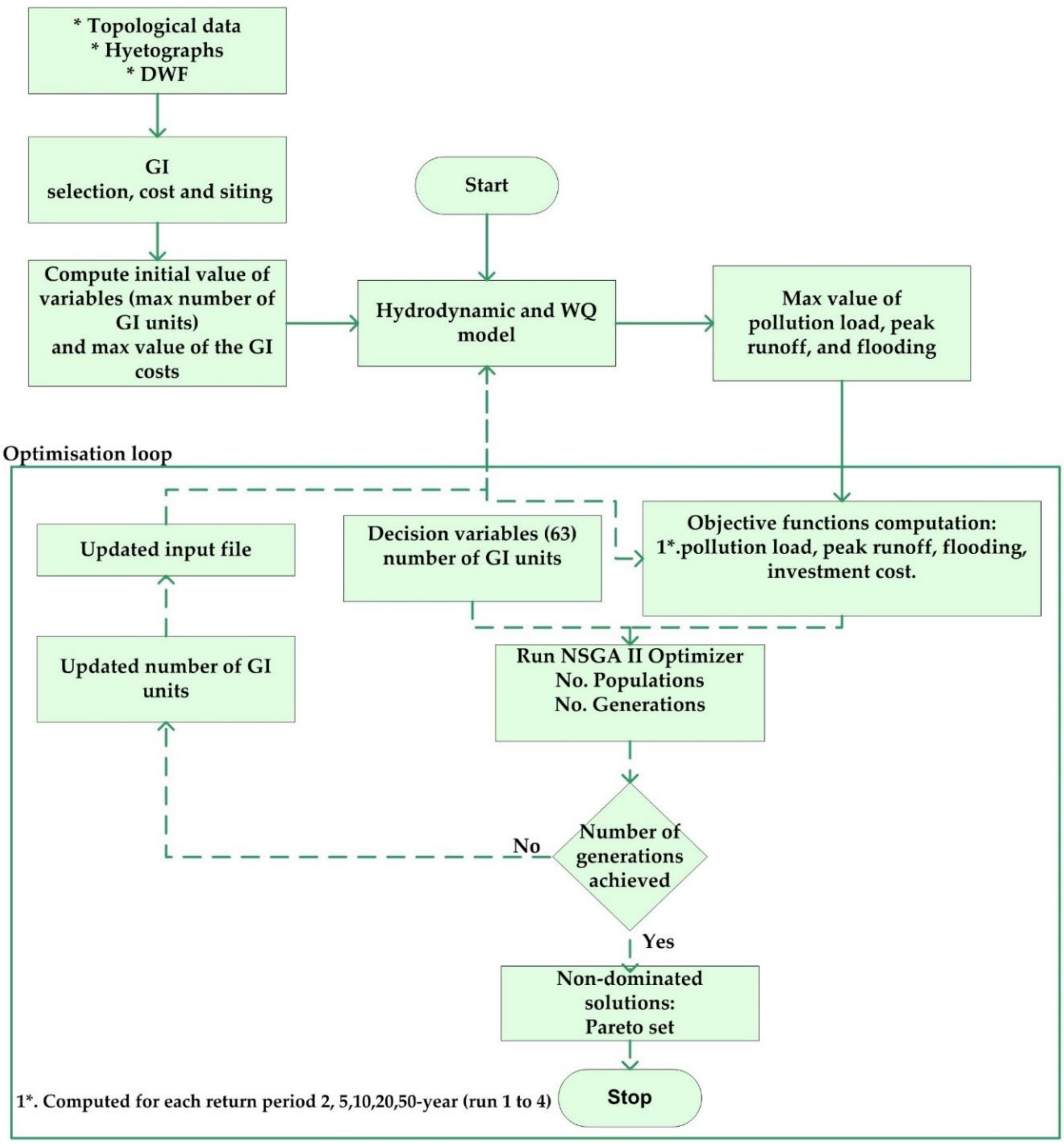


Figure 3.3. The optimisation procedure.

Linkage between the hydrodynamic model and NSGA-II optimiser

Two interfacing routines were developed and coded using LAZARUS, a free-source Delphi compatible cross-platform IDE for rapid application development to join the NSGA-II optimiser with the hydrodynamic model. The hydrodynamic input file specifies how a particular GI is deployed. The data entry fields include the GI type and its number of units (i.e. decision variables) for each subcatchment. The first routine was coded to run the hydrodynamic model, set the initial value of variables and compute the normalized values of the objective functions. The maximum value of GI investment cost is also computed from a catalogue file which contains the GI unit cost.

The NSGA-II optimiser generates a file with the lower and upper range values of the decision variables. The second routine was also coded to run the model and update the hydrodynamic input file. This routine uses the generated file with the decision variables range to modify the number of GI units by overwritten its value in the hydrodynamic input file for each iteration. With this procedure a new objective function value is obtained. Optimisation parameters for the NSGA-II algorithm were set to a population size of 200 and 30 generations for a total number of function evaluations of 6,000. The probability of crossover was set to 0.90, the probability of mutation to 0.09, the distribution index for crossover to 15 and the distribution index for mutation to 20.

Objective Functions

In order to quantify the trade-offs between the reductions objectives, three objective functions were coded: pollution load, peak runoff and flood volume with their corresponding reductions in investment costs. The objective functions are presented as follows:

Pollution Load Reduction

The maximum wash-off of TSS in the urban drainage system is used for the evaluation of percentage reduction of TSS. The objective function calculates the average of TSS wash-off for each subcatchment using Equation (3.3).

$$f_1(x_i) = \frac{1}{n} \cdot \sum_{j=1}^{n} \frac{wf_j}{wf_{j,max}} \tag{3.3}$$

Where, $f_1(\, x_i)$ is the fitness function 1 of chromosome i, n is the number of subcatchments, j is the subcatchment number, wf_j is the TSS wash-off (kg/day) of the subcatchment j, and $wf_{j,max}$ is the maximum TSS wash-off load (kg/day) in the system without using any GI type in each subcatchment j.

Peak Runoff Reduction

In the model, surface runoff occurs when the depth of water exceeds the maximum depression storage so that the peak runoff value is taken from the model system response through its hydrograph (Rossman, 2017). The objective function consists of calculating the average of each normalized runoff peak of each subcatchment as follows:

$$f_2(x_i) = \frac{1}{n} \cdot \sum_{j=1}^{n} \frac{pr_j}{pr_{j,max}} \tag{3.4}$$

Where, $f_2(x_i)$ is the fitness function 2 of chromosome i, n is the number of subcatchments, j is the subcatchment number, pr_j is the peak runoff (m^3/s) of subcatchment j, and $pr_{j,max}$ is the maximum runoff in the system without using any GI type in each subcatchment j.

Flood Volume Reduction

In the model, flooding occurs when the water depth at a node exceeds the maximum available depth, and the excess flow is either lost from the system or can pond on top of the node and re-enter the drainage system (Rossman, 2017). This objective function focuses on evaluating the flooding volume reduction in terms of the maximum flooding volume obtained from the model simulations. It consists of the sum of the volume of all the nodes divided by the sum of flood volume coming from the system without using any GI type in each subcatchment j.

$$f_3(x_i) = \frac{\sum_{j=1}^{n} nfv_j}{nfv_{j,max}} \tag{3.5}$$

Where, $f_3(x_i)$ is the fitness function 3 of chromosome i, n is the number of flood conflicting nodes, j is the node number, nfv_j is the flood volume (m^3) of node j, and $nfv_{j,max}$ is the maximum flooding volume coming from the system without using any GI type in each subcatchment j.

Investment Cost Function

The investment cost of each GI configuration system is related to the total number of GI units implemented in each subcatchment multiplied by their implementation cost. The number of GIs results from the optimisation process while the implementation cost is calculated from the catalogue that contains unit costs for different GI. This is presented in the following Equation 3.6.

$$f_4(x_i) = \frac{\sum_{j=1}^{n}\left(GI.cost_j \cdot GI.number_j\right)}{cost_{max}} \tag{3.6}$$

Where, $f_4(x_i)$ is the fitness function 4 of solution i, $GI.cost_j$ is the cost (US dollars / m^2) of GI type j, $GI.number_j$ is the number of GI type j and $cost_{max}$ is the maximum implementation cost.

The output of the optimisation procedure includes non-dominated solutions with the number of GI units to be implemented for lowest possible cost and for return period events of 2, 5, 10, 20 and 50 years. The percentage of reduction reached has been computed taking into account de objective function value (O.F) obtained from the optimisation process and the maximum objective function value without using any GI type as follows:

$$100 - \frac{[O.F\ value(opt.solution) * 100]}{Max\ O.F\ value\ (present\ state)} \tag{3.7}$$

Maximum GI investment cost

The maximum GI investment cost was calculated by adding the investment cost to the operation and maintenance cost per square meter of each GI type for 20 years. A unit cost was computed based on the cost categories according to the layers of the GI (i.e., surface, soil, storage, underdrain). An inflation rate of 2.8% was used in order to calculate the net present value of each unit (BCC, 2014) and the base prices were taken from the city price index (GVC, 2017). The investment cost for all types of BR included less cost for the surface layer due to the lower costs of local grass. Investment costs for different types of IT may vary from one place to another due to the variation in local gravel costs. The overall project investment cost using this maximum number of GI units was found to be \$ 19.9 million dollars. This value was obtained according to the maximum number of units of each GI type and its corresponding unit cost (see the summary of GI in Table 3.3).

3.4 Results and discussion

Initial Performance of the Drainage System

The hydrodynamic model was run for the selected return period events of 2, 5, 10, 20 and 50 years without implementing any of the GI measures (i.e. present state). Simulation results indicate a peak runoff of 147, 171, 185, 200, and 227 m^3/s, respectively. The TSS loading at the outfall of the system was found to be 37,348; 40,388; 42,635; 45,184 and 49,113 kg/day, respectively.

GI Placement

Potential suitable location/areas for different types of GI were identified from the analysis of urban land use, stream location, soil classification, land ownership and impervious layers. The placement of GIs was carried out by finding the available space for GI in each subcatchment. The minimum percentage of available area (ha) was found to be 2.8 % and the maximum 32 %. Figure 3.4 presents the maximum number of GI units for each subcatchment. With the criteria presented in Section 2.3.2, the maximum number of GI units found was 468 divided as follows: 116 units of BR, 116 units of IT, 164 units of VS and 72 units of PP. Figure 3.5 depicts an example of the TSS loading in each subcatchment for a 5-year return period event before and after GI placement (applying the maximum number of units). On average, a reduction of 40% of TSS could be potentially obtained after implementing GI measures.

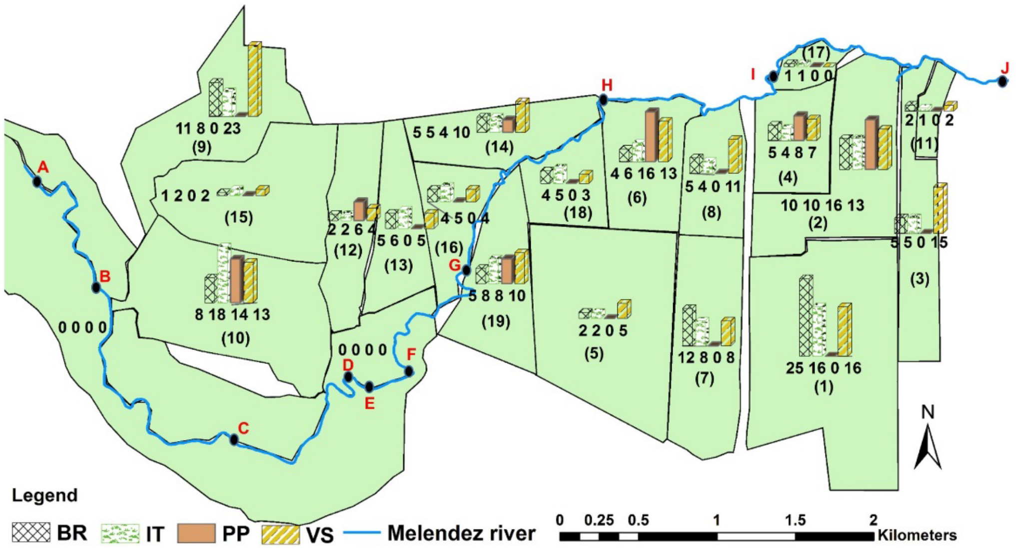


Figure 3.4. Maximum number of GI units in the Melendez catchment

(subcatchment numbers in brackets).

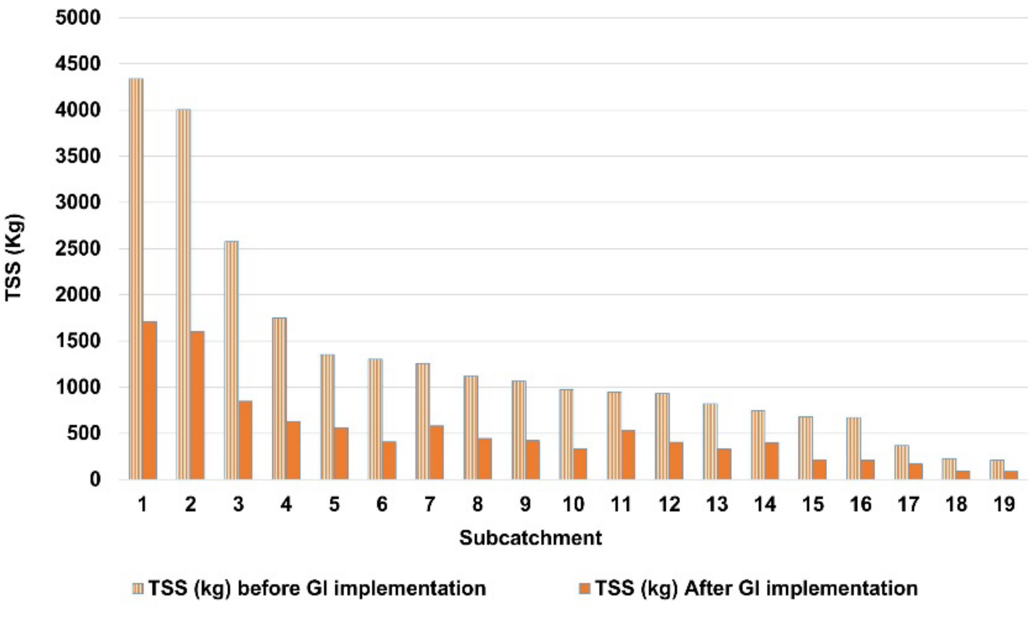


Figure 3.5. TSS (kg/day) for each subcatchment

The second hydrodynamic model run was carried out for 2, 5, 10, 20 and 50-year return period events with the maximum number of GI units (i.e. 468 units). Simulation results indicate a peak runoff reduction to 114, 131, 140, 153, and 172 m^3/s, respectively. Also, the reduction of TSS at the outfall of the system was found to be 22,737; 24,955; 26,717; 28,585 and 31,471 kg/day, respectively.

Assessing optimal GI measures

In order to obtain optimal GI solutions distributed within the catchment, a trade-off between each optimisation model (i.e. pollution load, peak runoff, flooding volume) and investment costs was introduced as an optimisation problem. As described above, four different GI measures were evaluated (BR, IT, VS and PP) and the selection of these measures were described in Section 3.3.2. These measures were evaluated by running simulations for 2, 5, 10, 20 and 50-year return period events. The maximum number of GI units was a subject of the optimisation process. Figure 3.6 shows the non-dominated solutions obtained for the mentioned objectives.

Figure 3.6a shows that for smaller events, with solution s-2 an investment of \$7.5 million can achieve a pollution load reduction of 43%. For larger return period events (up to 50 years) a \$7 million investment suggests a pollution load reduction of 40% (solution s-10). In terms of peak runoff reduction, Figure 3.6b presents solution s-12 with a peak runoff reduction of 30% by investing \$6.4 million for a 2-year event. With solution s-20 it is possible to reduce peak runoff by 27% for a 50-year event and \$6 million investment. Figure 3.6c presents optimal solutions for flood volume reduction. Solution s-22 shows a reduction level of 80% for a 2-year event with an investment cost of \$7.3 million. Solution s-30 demonstrates a flooding volume reduction of 68% by investing \$7.4 million (up to 50 years).

Table 3.3 presents the comparison of the selected solutions for each objective reduction. The catchment points (letters in red colour) presented in Figure 3.4 have been used for comparison purposes between the present state (no GI placement) and the computed optimal solutions.

Solutions s-2 and s-10 indicate an important pollution decrease especially where the water quality deterioration in points G, H, I and J of the Meléndez catchment is very significant. Solution s-12 and solution s-20 are able to regulate the flow of the river when a rainfall event occurs in the upper part of the catchment (points A, B, and C) and thus reduce the river flow at the entrance of the city (between points G and H). Solution s-22 and solution s-30 indicate the possibility of reducing the risk of flooding particularly in points H and I where the highest flood volumes occur in a mostly residential area. Figure 3.7 presents the optimal number of units with the aim of identifying the GI type that better reduce the three objectives for the study area.

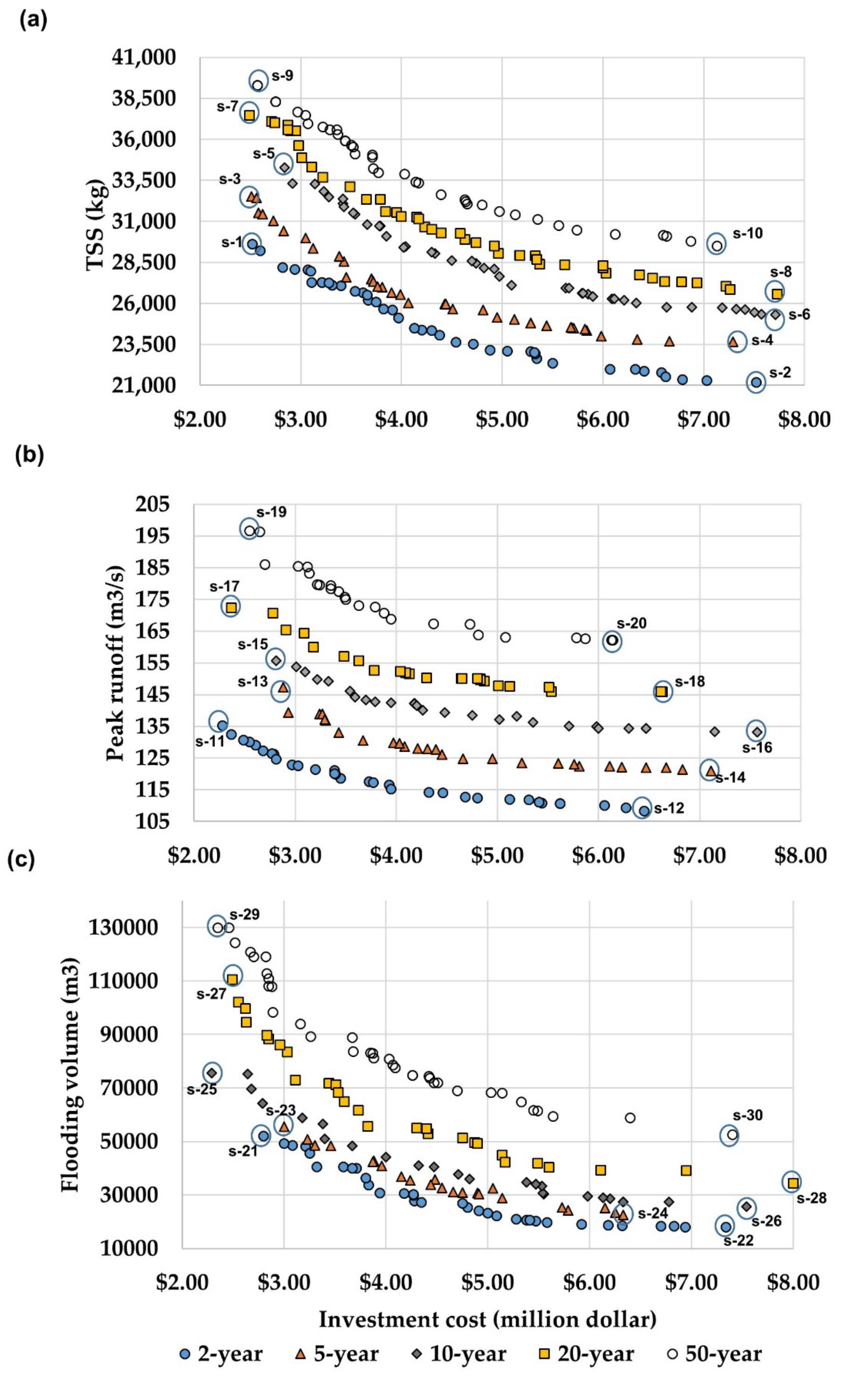


Figure 3.6. The non-dominated solutions obtained for different objectives (a) pollution load; (b) peak runoff; (c) flood volume. Two solutions were initially selected from each return period; the first solution corresponds to a very low objective reduction and minimum investment cost. The second solution corresponds to a very high investment cost and maximum objective reduction.

Table 3.3. Comparison of the selected solutions

Catchment point	Return period	TSS (kg)			Peak runoff (m^3/s)			Flooding volume (m^3)		
		Present state (no GI)	Optimal solution s-2	Optimal solution s-10	Present state (no GI)	Optimal solution s-12	Optimal solution s-20	Present state (no GI)	Optimal solution s-22	Optimal solution s-30
	2	2,433	924	-	3.72	1.88	-	5,654	933	-
G	50	2,976	-	1,286	4.59	-	1.71	8,111	-	2,470
	2	659	276	-	5.02	2.96	-	6,279	1,364	-
H	50	799	-	281	6.28	-	3.71	37,802	-	9,914
	2	195	83	-	0.94	0.55	-	27,964	12,455	
I	50	244	-	114	1.18	-	0.75	34,177	-	21,656
	2	215	89	-	1.47	0.79	-	-	-	-
J	50	262	-	110	1.83	-	1.34	-	-	-

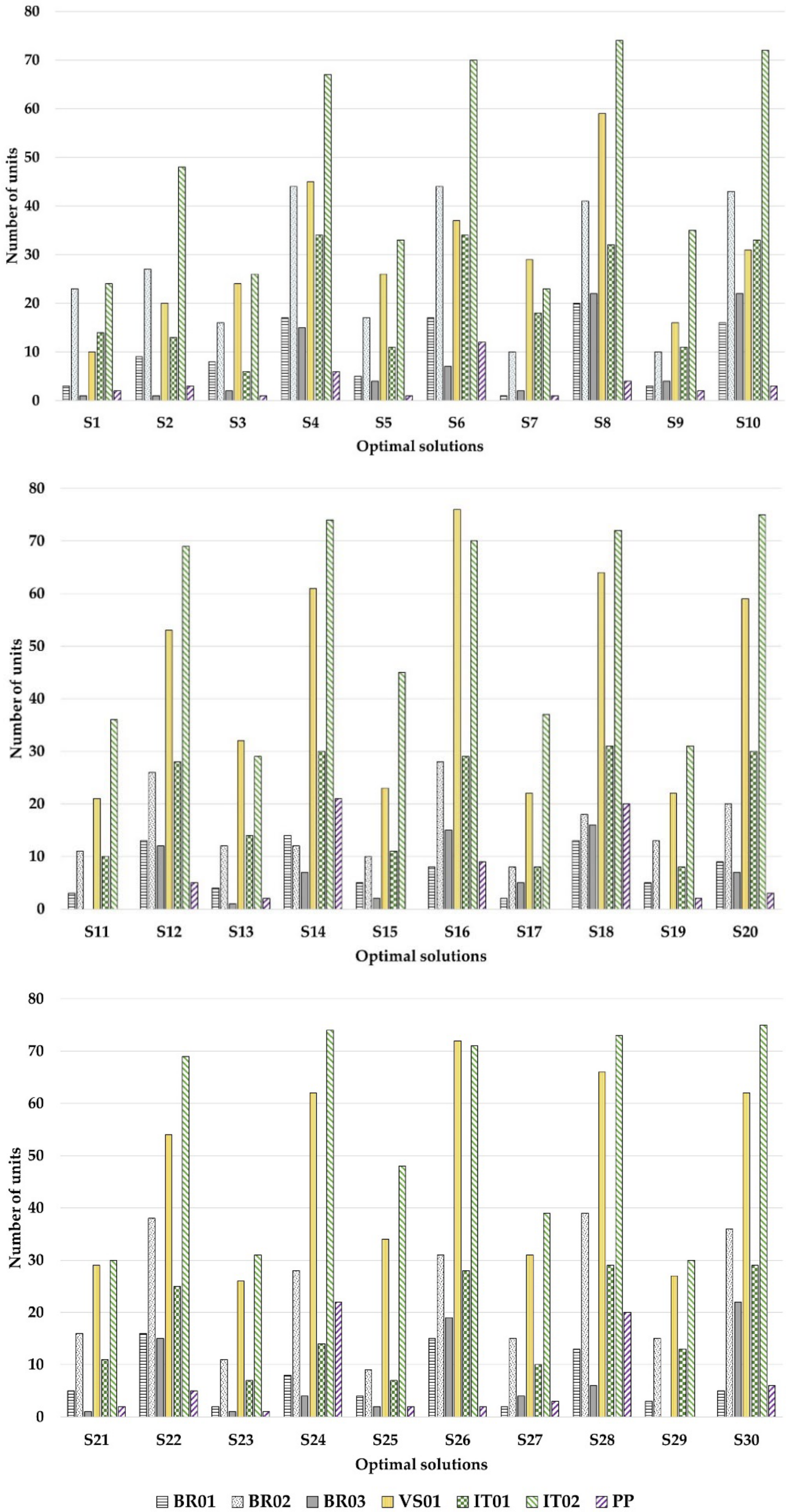


Figure 3.7. Optimal number of GI units associated with solutions.

As it can be seen from Figure 3.7, GI types such as infiltration trench (IT02) and vegetative swale (VS01) for small and large events present the largest number of GI units deployed in the catchment reducing the three objectives. Comparing these numbers with the results obtained in Figure 3.6 and Table 3.2, a larger number of IT02 and VS01 would have an effect on improving runoff quality and quantity compared to bio-retention cells (BR) and porous pavement (PP) despite the small differences in the investment costs. According to the characteristics of the GI design shown in Table 3.1, IT02 could have been more placed due to the rate value at which water seeps into the native soil below the layer (greater than IT01).

Through the analysis of these results, it can be observed that BR02 has been mainly used with the purpose of reducing pollution load (solutions s-1 to s-10) compared to the other two objectives. From the three types of BR, BR02 differs from the other two due to the thickness of the soil layer, the rate value at which water seeps into the native soil below the layer (both greater than BR01 and BR03) and the draining type (it uses infiltration rather than underdrains). In line with the design characteristics, for larger events the use of PP01 in the catchment increases in solutions s-8 and s-10 for pollution load reduction and solutions s-18 and s-30 for peak runoff and flooding volume reduction respectively.

The main constraint of these optimal solutions is the amount of financial resources that is required to the initial GI implementation which is $ 19.9 million dollars. Within this framework important investment cost reductions were obtained in terms of pollution load, peak runoff and flooding volume to $7 million, $6 million and $7.4 million respectively for events up to 50 years. In terms of the effect of a rapid increase in rainfall intensity and similar to the work presented in (Mugume and Butler, 2017) different return periods (2, 5, 10, 20 and 50-year) enable not only a better understanding to achieve optimal solutions but also an effective GI placement minimizing erroneous and costly intervention for urban runoff reduction.

According to other researches, although we manually determined subcatchment parameters and GI process layers much of this work could be automated. However, it is important to consider the stakeholders input into the final design decisions for each subcatchment, possibly iteratively applying this framework with additional constrains until a satisfactory solution is found (See, Oraei et al., 2012; Chichakly et al., 2013; Jayasooriya et al., 2016). The methodological framework presented here demonstrates a possible way to select one solution from different alternatives. For the case study area of Cali, these solutions that can maximise environmental and economic objectives for up to a 50-year return period event could be considered as preferred. However, since different GI type units can produce a similar performance, the preferred combination of GIs would depend on the objectives that need to be achieved. In future work such issues should be addressed by incorporating a preference-based multi-objective model within the present methodological framework.

3.5 Conclusions

The present Chapter describes a novel methodological framework that aims to configure GI for urban runoff and pollutant reduction using the optimal number of units. This work addressed the assessment of the performance of GI measures dealing with environmental and economic objectives. The proposed methodological framework has been implemented in the coding environment LAZARUS which is a free-source Delphi compatible cross-platform. The code combines a hydrodynamic model and an optimisation algorithm. Simulations of hydraulic, hydrologic and quality aspects were performed within the SWMM package while the NSGA-II model was used for process optimisation. The mass of a pollutant transported during a storm event has been modelled as a coupled build-up and wash-off process, providing the stormwater pollutant load generated from the urban catchment. The work was demonstrated in a real-life case study of Cali (Colombia) where bio-retention-cells (BR), infiltration trenches (IT), vegetative swales (VS) and porous pavement (PP) were evaluated considering pollution load, peak runoff and flood volume objectives at the lowest possible investment cost.

There are currently actions that are aiming to reduce the pollution load in the Meléndez river but its water quality is still continuing to decline. Similarly, in spite of substantial investment in flood control structures in the catchment, there is still the risk of flooding as the investments are not executed according to their priority and their true impact in the catchment. The results of this study show that by investing an amount of $7.7 million with a higher number of BR units (up to 83 units) within a specific configuration, a pollution load reduction for larger events can be obtained with solution s-10. The solutions also show that an increase in the number of VS units (up to 76 units) with the same investment can yield a reduction in peak runoff for both smaller and larger events (s-16 and s-18). Similarly, with the same level of investment and with a larger number of PP units (up to 22 units), solution s-30 would help to reduce flood volume for shorter and larger events.

The application of multi-objective optimisation processes for GI configuration may become a good choice in terms of reducing investment cost without compromising the efficiency of the drainage system. The results show an advantage of having an optimal number of GIs as the GI types mainly reflect the impact on the reduction of the three objectives. This suggests that if the type of GI measure and its number of units are taken into account within the optimisation process, it is possible to achieve optimal solutions to reduce the proposed reduction objectives with a lower investment cost.

In terms of disadvantages, one of the key disadvantages is that this approach is not able to incorporate surface water infiltration process into the hydrodynamic model based on a given infiltration equation (i.e. modified Horton method) in a 1D-2D modelling approach taking into account the expensive computational time which limits its application for real-life purposes. The present work also demonstrates how different performance can be used to address different objectives and to identify a solution that can be suitable for the study area.

4

MULTI-OBJECTIVE EVALUATION OF GREY INFRASTRUCTURE FOR URBAN DRAINAGE REHABILITATION

This Chapter describes a framework for the evaluation of a drainage system's capacity in order to get a better understanding of the interactions between three rehabilitation measures: The upgrading of pipes (UP), distributed storage (DS) and the combination of both (UP+DS). It is posed as a multi-objective optimisation problem with the aim of minimising rehabilitation costs and flood damage. The approach of Expected annual damage cost (EADC) was also introduced as the probabilistic cost caused by floods for a number of probable flood events (i.e. the accumulation of damage during a timeframe). The study combines computational tools such as the coupled 1D/2D flood inundation model and an optimisation engine in the loop to compute potential damages for different rainfall events and to optimise combinations of rehabilitation measures. The advantages of this approach are demonstrated also on a real-life case. The optimal solutions confirm the usefulness and effectiveness of the proposed approach where both rehabilitation and damage costs are reduced by the optimal implementation of the UP and DS measures. In addition, the results of the proposed EADC approach indicate a damage cost reduction of at least 56% by implementing UP and of 27% by implementing DS, and both measures have lower rehabilitation costs. The proposed approach can be found appealing to water/wastewater utilities who are often challenged to achieve optimal design and rehabilitation of urban drainage systems.

Based on: Martínez, C., Sanchez, A., Toloh, B., Vojinovic, Z. (2018). Multi-objective evaluation of urban drainage networks using a 1D/2D flood inundation model. *Water Resources Manag.* 2018, 32 (13), 4329-4343. Doi: 10.1007/s11269-018-2054-x

4.1 Introduction

Dealing with floods in urban areas has become an important and growing issue for urban flood managers. In many cities around the world, urban drainage systems (UDS) are reaching the end of their expected useful lives and timely rehabilitation of such systems is imperative for proactive asset management. Experience has shown that the main constraint of conventional drainage systems is related to their maintenance and operational activities, along with the financial, social and adaptive limitations of the local context (Dominguez, 2011; Schellekens and Ballard, 2015). The recent progress in sustainable drainage development across different disciplinary fields suggests a new goal related to the 'sustainability' of urban drainage systems which takes into account the urban drainage management as a component of the urban water cycle (Krebs and Larsen, 1997; Zhou, 2013).

Investigations attempting to enhance the performance of urban water systems have seen significant improvements (e.g. Ten Veldhuis and Clemens, 2011; Yazdani et al., 2011; Mugume et al., 2015; Marques et al., 2015; Diao et al., 2016). The results of these studies have brought about tools and techniques that can enable the development and implementation of more effective and resilient solutions. The use of numerical models and optimisation techniques have also proved to be invaluable for dealing with various system rehabilitation issues.

With models and optimisation techniques, it is possible to explore the performance of drainage networks and evaluate the effectiveness of different intervention measures. However, there are several issues that need careful consideration in order to use numerical models more effectively for this task (e.g. Vojinovic et al., 2006; Vojinovic et al., 2006a; Abdullah et al., 2009; Vojinovic et al., 2014). Park et al. (2012) and Cunha et al. (2016) implement evolutionary algorithms for design of detention pond geometry. Artita et al. (2013) find the optimal location of best management practices for an integrated watershed-scale management problem. Sanchez et al. (2014) describe and demonstrate an integrated cellular automata evolutionary-based approach for evaluating future scenarios including the expansion of UDS. Yazdi et al. (2014) present a new risk-based optimisation approach for determining rehabilitation plans in urban drainage systems by integrating the copula method, MCS, Multiobjective EAs, and hydrodynamic models.

Multi-objective optimisation of UDS including the 1D/2D modelling approach has also seen some significant advantages (e.g. Vojinovic et al., 2014; Martínez-Cano et al., 2014b) and the results obtained demonstrate their potential for solving some of the greatest challenges that water/wastewater utilities face nowadays. Despite the remarkable progress achieved over the past two decades in the area of sustainable urban water management, there is still the remaining challenge of how to secure and manage infrastructure investments so that systems meet continuously increasing service standards and challenges posed by climate change. The present chapter provides contribution in this direction by presenting a novel approach that combines 1D/2D models with an optimisation algorithm in order to search for optimal solutions for a drainage system that needs rehabilitation. The key advantage of this approach is that we are able to retain the necessary physics (and interactions) in computations between pipe network systems (i.e., below ground system) and urban surface (i.e., above ground system) while searching for an optimal set of rehabilitation measures. In terms of the disadvantages, one of the key disadvantages is that this approach requires rather extensive computational time which limits its application for real-time purposes.

This proposed approach has been implemented in the code of EMBARCADERO Delphi integrated environment. The potential of the proposed approach has been demonstrated on the real-life case study of Dhaka City. Different rehabilitation measures were evaluated in relation to investment and flood damage costs. The results obtained are promising and confirm that the proposed approach has a good potential to deal with one of the greatest challenges that water/wastewater utilities face nowadays.

4.2 Study approach

4.2.1 Problem formulation

The assessment of flood damage is done using the 1D/2D model flood inundation model. These two models have been coupled dynamically to exchange information in each time step. The output of the coupled model is a flood map depicting the water depth onto a 2D grid. This map represents spatial distribution of flood water depths within the model domain (i.e., an urban area containing buildings, residential areas, commercial areas). This approach enables a more realistic computation of damages across the model domain. The estimation of flood damage is traditionally computed for different levels of risk or return periods. In this Chapter, the concept of Expected annual damage cost (EADC) was also introduced as the probabilistic cost caused by floods for a number of probable flood events (i.e. the accumulation of damage during a timeframe). This can be considered as yet another novelty in the present work.

To explore different solutions for the rehabilitation of an UDS a trade-off between rehabilitation costs and direct flood damages was formulated as an optimisation problem. Three different rehabilitation measures were tested (pipe diameters, storage and their combination). Each of these measures were evaluated by running simulations for 2, 10, 20 and 50-year return period rainfall events. After that, calculation of EADC was undertaken as an objective function instead of the individual flood damage calculation done for each return period. The total expected cost (TEC) is then obtained by summing up of all costs (i.e., rehabilitation costs) and benefits (EADC). To demonstrate the potential of the proposed approach an area known as Segunbagicha located in Dhaka (Bangladesh) has been used as a case study site.

4.2.2 Data requirements

To set up a 1D model to simulate the minor system (pipes), the data such as network layout, conduits, manholes, outlet, subcatchments with their hydrological parameters and dry weather flow characteristics were utilised. Rainfall is applied to the model surface using hyetographs to describe different events based on intensity, duration and frequency. Sub-catchment parameters include outlets nodes, percentages of previous and impervious areas, terrain slope, depression storages and soil infiltration characteristics.

The data used to build the 2D model includes a digital elevation model (DEM), road network and buildings. To assess damages a method based on depth-damage curves was used and applied for different sectors (i.e., land uses). For the assessment of rehabilitation costs, two catalogues with infrastructure costs were applied. The first catalogue contains unit costs of pipes with the associated costs of excavation and reinstatement works. The second catalogue contains the storage area (m^2) costs.

4.2.3 Hydraulic modelling

As mentioned earlier, for the hydrodynamic modelling purpose, the proposed approach combines the sewer network model (SWMM) with the 2D surface water model. The output of the 1D/2D flood inundation model is a matrix with values of flood depths across the surface which is in turn used to compute flood damage costs as part of the optimisation process.

4.2.4 Optimisation framework

The NSGA-II algorithm, developed by Deb et al. (2002), was used within the proposed approach to search for a set of Pareto optimal solutions and for quantifying trade-offs between flood damage costs, rehabilitation cost and the EADC. The flowchart of the proposed approach is depicted in Figure 4.1.

The optimisation steps include the following: (1) Initial simulation of the hydraulic 1D/2D flood inundation model, (2) Computation of the maximum value of damage costs for each return period and the initial EADC, (3) Computation of the objective functions, (4) Running optimiser NSGA-II according to the number of populations and generations, (5) Updating pipe diameters or size of storage if the number of populations and generations have not been reached and (6) Updating of 1D/2D model input file.

With the purpose of executing the optimisation framework described above, two interfacing routines were developed and coded using the EMBARCADERO environment (former Borland Delphi). A first routine was coded to run the 1D SWMM model to compute the initial value of the variables, objective functions and maximum value of the original costs. A second routine was also coded to run the coupled 1D/2D model, select from the range of decision variables, change the pipe diameters of the selected elements to be rehabilitated, resize the selected storage, and compute the objective functions.

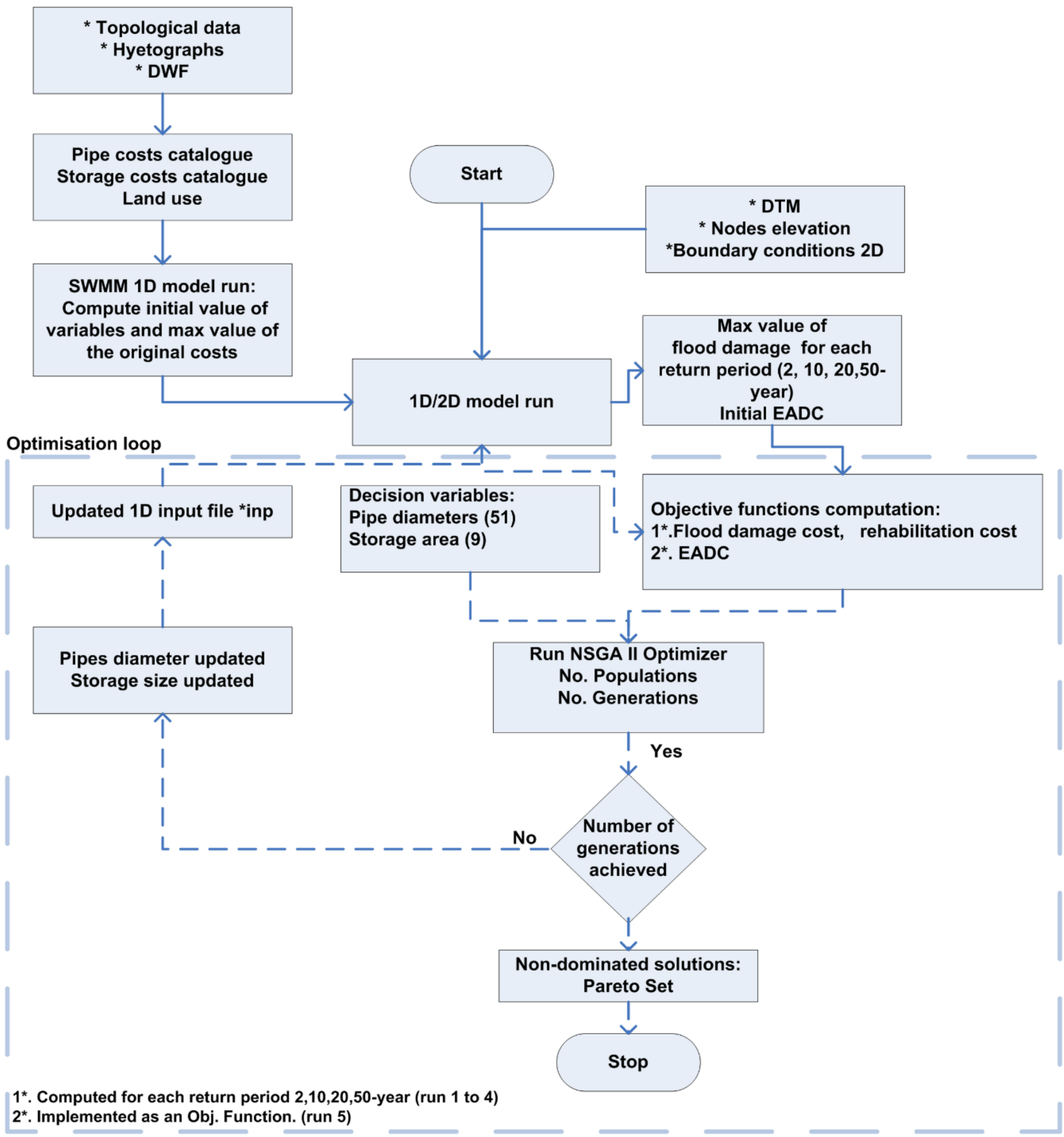


Figure 4.1. Flowchart of the applied framework

Initially, it was found that the 1D/2D model simulations are rather computationally expensive. For example, a single run of the 1D/2D model can take up to one hour and the estimated time for one run of the optimisation framework to assess 2000 evaluations took 7 days. Therefore, it was decided to use the NSGA-XP, which is a version of the NSGA-II that can run in parallel using a cluster of computers. Based on the work of Barreto (2012), two clusters of four laptops were set-up and run using the master-slave approach and executed into a Parallel Virtual Machine environment. The laptops had the following architecture: Intel (R) Core (TM) i5-2410M CPU@ 2.30 GHz, 4.0 GB RAM, 64 bits O.S and Windows 7 platform. The total computational time for one optimisation run was reduced from 7 days to 3.5 days for a single rainfall event.

The optimisation parameters of the NSGA-II algorithm for each rehabilitation measure and for the EADC were set to a population size of 100 and 20 generations for a total number of function evaluations of 2,000. The probability of crossover was set to 0.90, the probability of mutation to 0.09, the distribution index for crossover to 15 and the distribution index for mutation to 20. The decision variables consisted of 51 pipe diameters ranging from 0.3 meter to 2.0 meter diameters based on a catalogue of commercially available sizes. For storage options, 9 possible sites were identified and the decision variable was set to be the surface area at these sites. The surface area at these sites ranges from 0 (No storage) to 1200 m^2. The depth was kept constant at 5m. The objective functions used in the experiments were rehabilitation cost, flood damage and EADC.

4.2.5 Rehabilitation cost function

The objective function for pipe rehabilitation is a function of pipe lengths (Barreto, 2012). Equation 4.1 presents the rehabilitation cost function for upgrading of pipes (UP).

$$RCost = \sum_{i=1}^{n} (C(P)_i) * L_i \tag{4.1}$$

Where *RCost* is the pipe rehabilitation cost (US dollars), *i* is the index of pipes ith, *n* is the number of pipes to be upgraded, $C(P)_i$ is the cost of the pipe ith (US dollars/m) based on the catalogue of commercially available sizes and L_i is the length of the pipe ith (m). For storage tanks, the costs are also based on the cost/area of possible storage in the defined catalogue.

4.2.6 Flood damage cost function

The estimation of the flood damage costs was done based on the maximum flood depth at the overland surface. To assess damages, depth-damage curves need to be developed taking into account different water depth ranges, land uses categories (e.g. residential, commercial, governmental, educational sectors) and a relationship by fitting a linear equation. Damage costs in each grid cell of the 2D model was computed using the Equation 4.2 given by:

$$DamageCost[i,j] = (\propto + \beta) * MaxWdpth[i,j] \tag{4.2}$$

Where α is the slope and β is the intercept of each linear regression and *MaxWdpth [i, j]* is the maximum water depth of the flood at the cells [i, j].

4.2.7 EADC function

EADC is the probabilistic cost caused by floods for a number of rainfall events. In this study, it was computed as the integration of damage costs for four different rainfall events of different magnitude (Olsen et al., 2015). Four rainfall events with return periods of 2, 10, 20 and 50 years were used to assess the performance of drainage infrastructure. The EADC represents the expected cost in any year during the time interval of the analysis. See for example the work of Barreto (2012). The EADC can be derived by using Equation 4.3.

$$\mathrm{EADC} = \sum_{i=1}^{Tr-1} [\left(\frac{\mathrm{Cost(i)} + \mathrm{Cost(i+1)}}{2}\right) * \ (\mathrm{Pi} - \mathrm{P(i+1)})] * \mathrm{f} \tag{4.3}$$

Where *Tr* is the return period event, *P* is the exceedance probability, 1/Tr and *f* is given by:

$$\mathrm{f} = \frac{(1+\mathrm{r})^{\mathrm{N}} - 1}{\mathrm{r} * (1+\mathrm{r})^{\mathrm{N}}} \tag{4.4}$$

Where *f* is the present worth factor, *r* is the interest rate and *N* is the service life of the assets (Olsen et al., 2015).

4.3 Case study

To demonstrate the potential of the proposed approach an area known as Segunbagicha located in Dhaka (Bangladesh) has been used as a case study site. This area has been experiencing frequent flood-related problems for many years. The system has a drainage area of 8.3 square kilometres and it includes the most important business and government office areas of Dhaka City. It encompasses 74 subcatchments, 88 conduits with a total length of 13,635 m, which is a combination of 75 circular pipes with a total length of 11,308 m, and 13 box culverts with a total length of 2,327 m. The circular sewer pipe diameters range from 450 to 5,500 mm and the box culvert sizes are between 2.5 by 2 m and 5.5 by 4.3 m. The system also includes 88 nodes (junctions), two pump stations and 1 outfall. The degree of impervious area was estimated for each subcatchment and the time of concentration in the outfall section was calculated to be in the order of 20 minutes. Figure 4.2 depicts the layout of the local drainage system.

The rainfall-runoff from sub-catchments is drained by pipe network system towards two basins from which sewage is pumped to the Tongi Khal river system. The digital elevation model (DEM) has 10 m resolution and it was used to set up the 2D model domain. The sub-catchment parameters (i.e., width, slope and percentage of imperviousness) were adjusted taking into account the previously calibrated 1D model of the sewer network system described in Ahmed (2008).

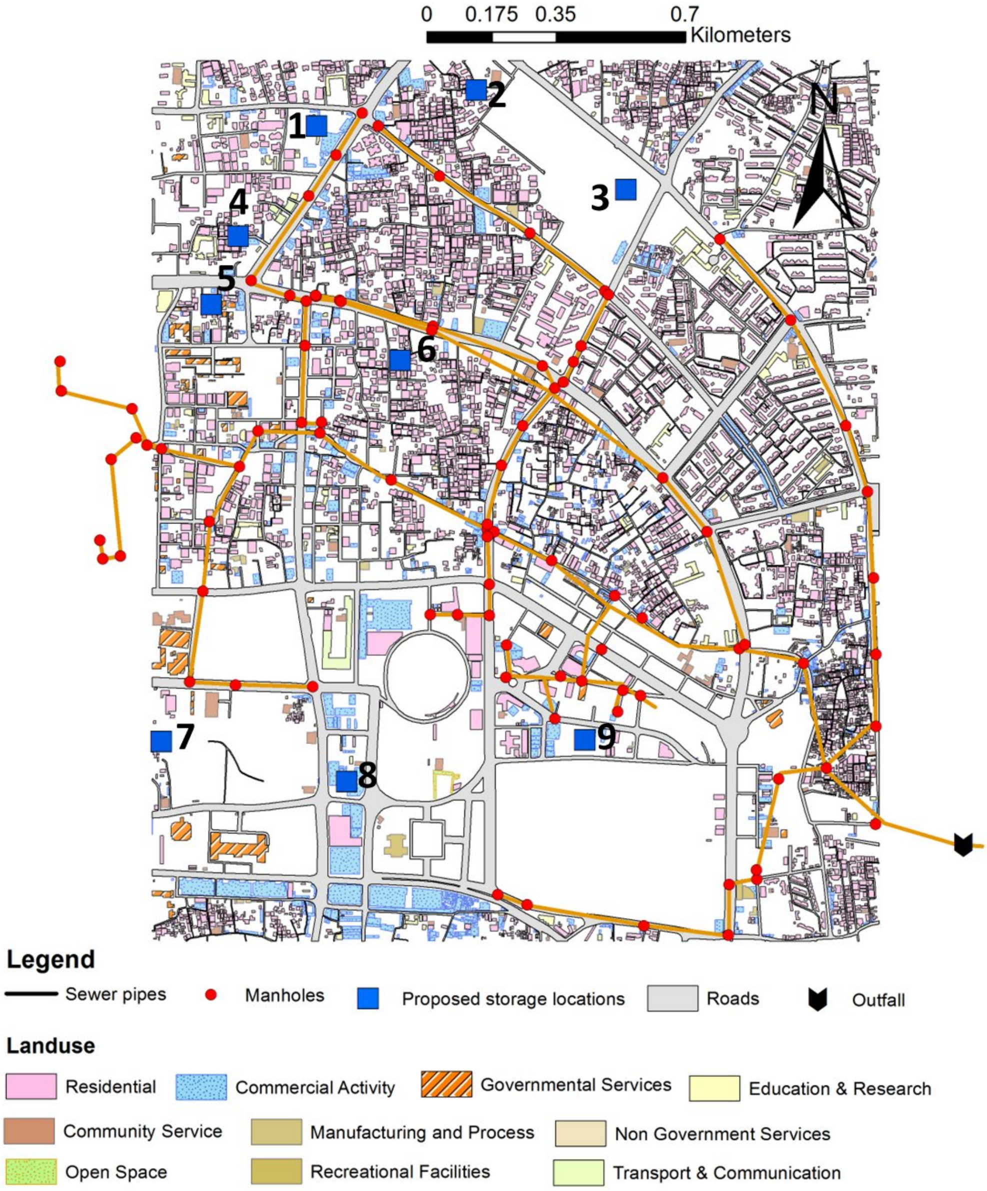


Figure 4.2. Drainage network layout for the Segunbagicha Catchment, Dhaka

4.4 Grey infrastructure

Three types of grey infrastructure are used in the present work to test their effects on the system's capacity: (a) upgrading of pipes (UP): 19 diameters of 51 pipes were found to surcharge (i.e. when the ratio between the maximum flow in the conduit and the full conduit area was greater than 0.99) and these pipes were identified as variables for optimisation. (b) distributed storage (DS): Nine possible sites for storage tanks were selected based on the availability of space and the performance of the system. Their location is illustrated in Figure 4.2. Storage tanks are defined through an elevation – storage curve with a maximum depth of 5 meters. The depth is governed by a weir and a control rule. This is handled inside the code developed to interface with the optimisation algorithm. (c) Combination of both measures (UP+DS).

4.5 Maximum rehabilitation cost

The rehabilitation cost of the drainage system infrastructure was calculated from the financial and economic analysis of the stormwater drainage master plan for Dhaka City (DWSA, 2015). The maximum value of the infrastructure for UP and DS measures was found to be in the order of \$44.6 and \$17.3 million respectively. These values were obtained by using the maximum value for each decision variable.

4.6 Estimated initial damage

The flood damage estimation is based on depth-damage curves that relate to an inundation depth (m) in a grid cell (obtained from the 1D/2D model result) and the land use class for each grid. For this purpose, nine land use damage curves (expressed in US dollars) and five water depth ranges (0.3 m, 0.61 m, 0.91 m, 1.22 m, 1.52 m) were applied based on the average damage/loss dataset developed for Dhaka City by Islam (2005). Land use classes for residential, commercial, governmental, educational and religious institutes, business, non-governmental utilities and industrial were used (Dutta et al., 2001). The damage costs in each grid cell of the 2D model were computed using the Equation 4.2. The nine land use damage curves and five water depth ranges led to a 45 damage cost functions. It is important to note that the present work addressed only estimation of tangible direct damages. The damage cost computed without implementing rehabilitation measures for rainfall events of 2, 10, 20 and 50 years was found to be \$3.7, \$7.4, \$9.2 and \$11.8 million respectively and the EADC computed initially was found to be \$46.6 million.

4.7 Results and discussion

4.7.1 Initial performance of the UDS

In order to assess the initial performance of the UDS, the 1D/2D model was run without implementing any of rehabilitation measures discussed earlier. Simulation results indicate that there is a substantial hydraulic overloading in the system that leads to flooding. Figure 4.3 depicts the inundation maps of the initial state system for four rainfall events used. Five critical branches that cause most of the flooding are branches marked as 1, 2, 3, 4, and 5 in Figure 4.3d. The total flood volume was found to be 6,040 m^3, 10,740 m^3, 13,620 m^3 and 18,650 m^3 for 2, 10, 20 and 50-year rainfall events respectively.

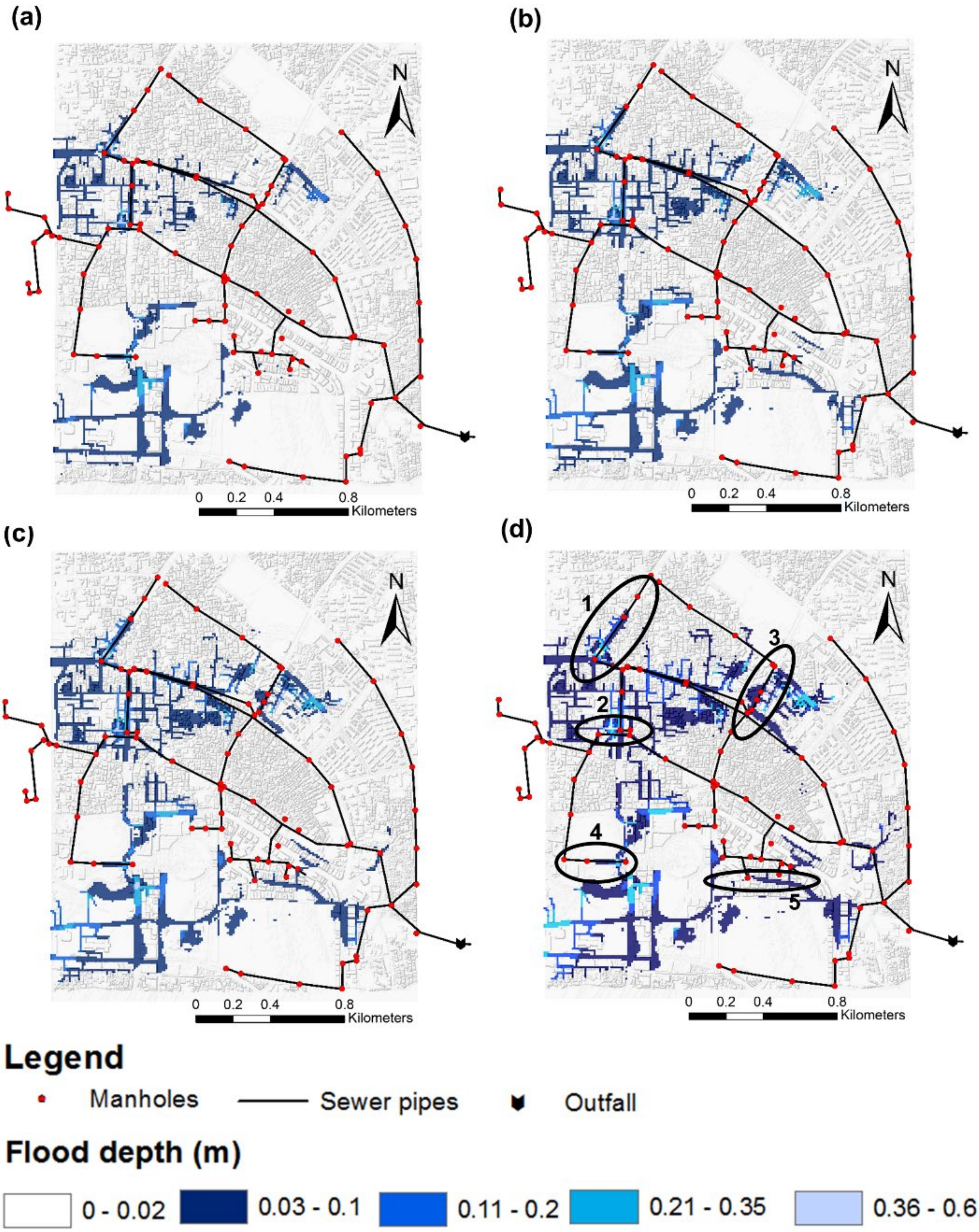
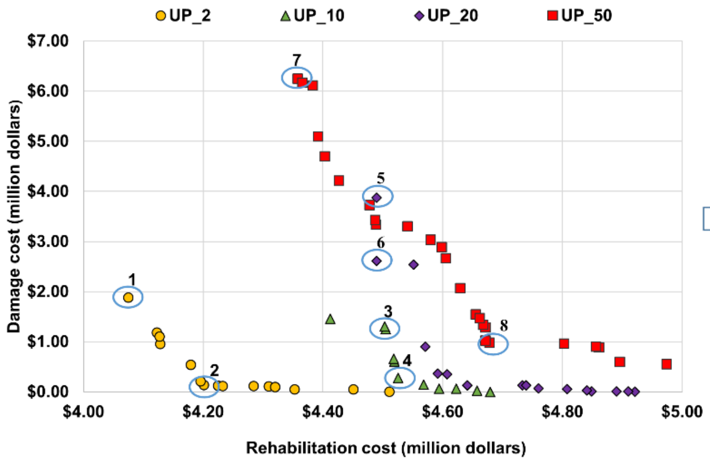


Figure 4.3. Inundation maps for a) 2-years b) 10-years c) 20-years d) 50-years - initial states of the UDS (DEM resolution 10 meters).

4.7.2 Assessing grey infrastructure

The first grey infrastructure evaluated was the upgrading of pipes (UP). The 1D/2D model was run for each rainfall event and diameters of 51 pipes were modified during the optimisation process (51 variables). The non-dominated solutions obtained and its summary for UP measure are presented in Figure 4.4.



Summary of the optimal solutions

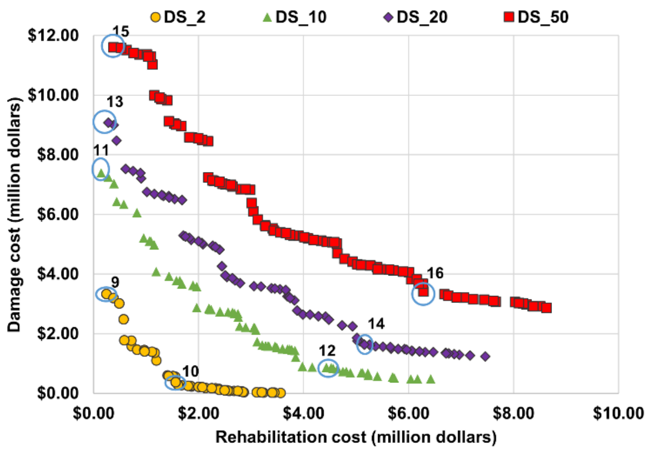
Return Period	Optimal solution #	Damage cost ($ million dollars)	Rehabilitation cost ($ million dollars)
UP_2	1	1.88	4.08
	2	0.13	4.20
UP_10	3	1.25	4.50
	4	0.28	4.53
UP_20	5	3.88	4.49
	6	2.61	4.49
UP_50	7	6.16	4.37
	8	0.98	4.68

Figure 4.4. The non-dominated solutions obtained for UP measure

The optimal solutions compared to the initial performance indicate a reduction to zero in flood damage cost for all rainfall events. The maximum damage cost is $6.1 million obtained with solution 7. The 2-year Pareto set dominates the others as less impacts require less money to be invested. From the analysis of a 10-year event, when compared to the 2-year event, it was found that this event also results in less damage but requires more substantial investment. Also, for this event 77% damage reduction is obtained from solutions 3 to 4. Similarly, the 20-year event Pareto set suggests that with the same rehabilitation cost of $4.5 million, it will be possible to achieve a decrease of 23% in damage from solutions 5 and 6. For a 50-year event there is a drastic drop from solutions 7 and 8 in terms of the damage by investing the amount of $4.7 million for infrastructure works leading to 68% of damage reduction. Solutions 3, 4, 5 and 6 also indicate a reduction in flood damage cost from $ 3.88 million to $ 0.88 million for 10, 20 and 50-year events with $ 4.50 million in rehabilitation cost. Solution 8 indicates that it would be necessary to invest $4.7 million as this solution does not only minimises the expected damage costs but it also achieves a total protection of up to a 50-year event.

The Pareto set for UP measure was capable of finding several solutions although there is no a wide range of values in rehabilitation costs due to the extensive computational time (i.e. the number of populations were set to a size of 100 and this value should be two or three times greater than the number of variables, 51 in this case). It the investment would exceed $ 4.7 million the rehabilitation cost becomes less efficient as it does not have an effect in reduction of damage costs.

The second grey infrastructure evaluated was a DS measure. The 1D/2D model was also run for each rainfall event and 9 storage tanks were placed in the UDS based on the availability of space. The storage area of each tank was modified during the optimisation process (9 variables). The non-dominated solutions obtained and its summary for DS measure are presented in Figure 4.5.



Summary of the optimal solutions

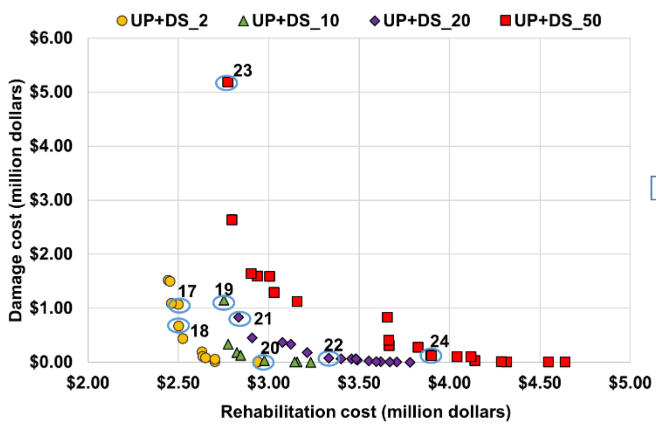
Return Period	Optimal solution #	Damage cost ($ million dollars)	Rehabilitation cost ($ million dollars)
DS_2	9	3.19	0.39
	10	0.26	1.68
DS_10	11	7.40	0.14
	12	0.87	4.44
DS_20	13	9.08	0.28
	14	1.75	5.07
DS_50	15	11.58	0.39
	16	3.40	6.29

Figure 4.5. The non-dominated solutions obtained for DS measure

The optimal solutions are found to achieve damage reduction of zero with solutions 10 and 12 for 2 and 10-year events respectively. With the available location of storage tanks, flooding was not reduced to zero for 20 and 50-year rainfall events. However, with DS measure it is possible to further reduce damage cost for all rainfall events (this would require an investment of $2.8 million). The total flood damage cost has been reduced by 81% by employing the solution 14 and 72% by employing the solution 16 when compared to damage costs obtained with solutions 13 and 15 respectively. With a DS measure, there is a clear trade-off between rehabilitation and damage costs for all rainfall events, and the flood damage is found to decrease as the investment increases.

The analysis of optimal results also shows that for smaller events (e.g. 2 years), optimal solution 10 suggests the implementation of three storage tanks, number 2, 4 and 5 (see, Figure 4.2). This would reduce flood damage by $0.2 million with an investment of $1.7 million (see the summary in Figure 4.5). For larger events (e.g. 50 years), the optimal solution 16 indicates the implementation of seven storage tanks, number 1, 2, 4, 5, 6, 7 and 8 to decrease flood damage by $3.4 million with an investment of $6 million. With an investment of more than $2, $5.4, $6 and $8 million the rehabilitation cost becomes less efficient as it does not have an effect in damage cost reduction.

According to other researchers, it is important to consider the combination of different measures under the specific local design conditions (Duan et al., 2016). For the combination of UP+DS measures, the investment cost was computed including the construction cost of the storage plus a summation of each pipe length multiplied by the cost of that particular pipe based on its diameter. Within the optimisation process, 51 pipes and 9 storage tanks as variables were included. The results of the optimisation process when UP and DS were implemented altogether and the summary of optimal solutions are displayed in Figure 4.6.



Summary of the optimal solutions

Return Period	Optimal solution #	Damage cost ($ million dollars)	Rehabilitation cost ($ million dollars)
UP+DS_2	17	1.06	2.50
	18	0.67	2.50
UP+DS_10	19	1.15	2.75
	20	0.03	2.98
UP+DS_20	21	0.83	2.83
	22	0.07	3.40
UP+DS_50	23	5.18	2.77
	24	0.11	3.90

Figure 4.6. The non-dominated solutions obtained for UP+DS measure

The results obtained show that solutions are able to reduce damage cost to zero for different levels of investment with less rehabilitation cost compared with UP and DS measures. It can be observed that for a total protection against flood damage an investment of around $3.5 and $4.0 million needs to be made. A maximum damage cost of $5.2 million is achieved compared to those obtained with the UP ($6.2 million) and DS ($11.6 million) measures. There are two solution points for a 2 years' event (17 and 18) showing a 37% drop for damage cost from one solution to another with the same rehabilitation cost of $2.5 million. For 10 and 20-year events solutions 20 and 22 suggest an investment of $2.9 and $3.4 million in order to reduce damages to zero compared to solution 19 and 21 with $2.7 and $ 2.8 million invested to reduce damages to $1.1 and $0.8 million respectively. The results from a 50-year event show a drastic drop (solutions 23 and 24) in terms of the reduction in damage from $5.2 to $0.1 million and by investing $3.9 million these levels of investments would be required to get protection for this return period event.

4.7.3 Expected annual damage assessment

The optimisation using a single return period does not provide much information about damages for other return periods and the accumulation of damages during a time frame.

In order to address this issue, the EADC was used to calculate the expected damages. Equations 3.3 and 3.4 were added to the algorithm for multi-objective optimisation. The interest rate *(r)* for this case was set to 6% according to the Bangladesh Bank and the service life of the assets *(N)* was assumed to be 50 years. The calculation of the EADC requires 1D/2D model simulations for 2, 10, 20 and 50-year rainfall events simultaneously (i.e. four simultaneous runs). Figure 4.7 depicts Pareto sets of EADC for the UP and DS measures. It also shows the total expected cost - TEC (damages + investment) for the measures.

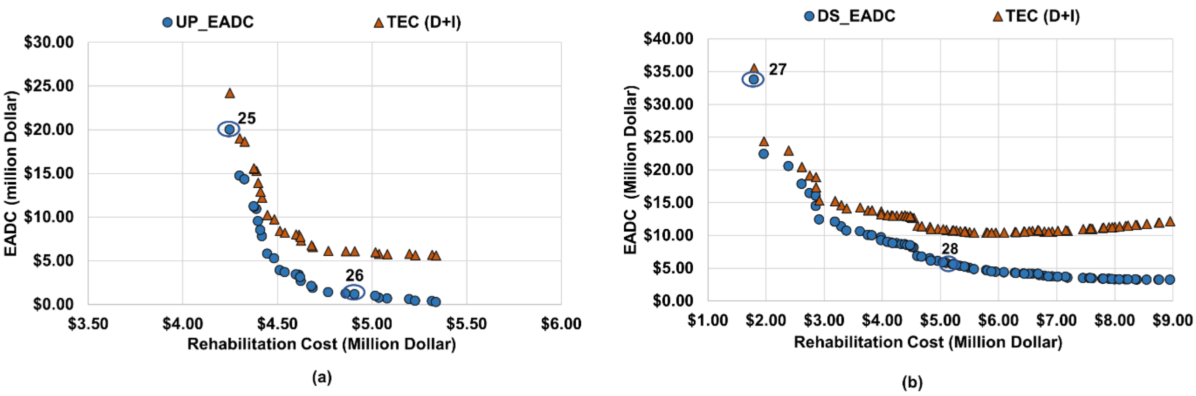


Figure 4.7. EADC Pareto sets and total expected cost (D+I) (a) UP (b) DS

The EADC for UP measure was computed with 51 variables (pipes). It increases the value of the damage to around \$20 million (solution 25), Figure 4.7a. This value is 3 times larger than the value for a single return period optimised for a 50-year event (\$6 million). Besides, there is a damage cost reduction of 56% compared to the actual performance of the system without implementing any measure (initial EADC cost = \$46 million). The TEC curve is obtained from the total cost by adding the damages and investment costs. The TEC curve indicates a maximum damage cost of \$24 million and a minimum value of \$5.6 million. Solution 26 refers to an investment of \$4.9 million and it presents a minimum damage of \$0.26 million. For this case there is no return period that exceeds the capacity of the drainage system. From Figure 4.4, it can be observed that this corresponds to a damage cost less than \$0.5 million for a 50-year event. It can be argued that the selection of solution 26 is a good choice as it minimises flood damage substantially and it also ensures the protection for up to 50-year event.

Figure 4.7b shows EADC and TEC for storage tanks computed with 9 variables (storage). For this measure, the damages increase to around \$33.7 million with solution 27. However, there is a damage cost reduction of 27% compared to the actual performance of the system (initial EADC). This value is also 3 times larger than the value for a single return period optimised with different storage tanks for a 50-year event (\$11.6 million). Similarly, when the TEC curve is obtained, the results show a minimum value of \$11.2

million with an investment of \$5.0 million. From Figure 4.5 it can be observed that this corresponds to a damage cost of \$0.5 million for a 10-year event. By implementing DS, the optimal solution 28 suggests the implementation of seven storage tanks # 1, 2, 4, 5, 6, 7 and 8, in order to reduce flood damage and to contribute to a total protection for up to a 10-year event.

Although EADC for the combination of UP+DS measures was not executed, from Figure 4.7 it can be inferred that a UP measure may have a global effect on flooding control for a total protection up to 50-year (see solutions 8 and 26) and a DS measure can be locally efficient to reduce flooding for a total protection up to 10-year (see solutions 12 and 28). This is also the case of (Duan et al., 2016) where LID devices and detention tanks were analysed. The EADC Pareto sets confirm the significance of including the benefits (reduced rehabilitation cost) over the design life.

4.8 Conclusions

The present Chapter describes a novel approach for evaluation of a drainage system's capacity which combines 1D/2D models with an optimisation algorithm. This proposed approach has been implemented in the code of EMBARCADERO Delphi integrated environment and demonstrated on the real-life case study of Dhaka, Bangladesh. Three different rehabilitation measures subject to different rainfall events were used with the aim of minimising flood damage and rehabilitation costs.

The results obtained indicate that for larger design event (up to 50 year return period events), UP as a measure would be a good option as it requires an investment of \$4.7 million to minimise the damage cost to \$1 million (see solution 8). With the same investment, if smaller events (up to 10 years) are selected for design purposes, a DS measure can reduce damage down to \$0.7 million (Figure 4.4), indicating the importance and efficiency of the UP and the practicality and economy of constructing DS along the urban catchment area. However, with the locations available to implement storage tanks, it was not possible to reduce flooding to zero for 20 or 50-year rainfall events. This suggests that for the present case study area the DS measure can become less effective beyond certain design events and the additional damage reduction would depend on their location instead on their volume.

The combined UP+DS measures show that solutions are able to reduce damage cost to zero for different levels of investment with less rehabilitation costs when compared to UP and DS measures implemented separately. In order to have a total protection for a 50-year event, between \$3.5 and \$4.0 million have to be invested for a total protection against flood damage. The concept of EADC was applied to calculate the accumulation of damages during a time frame. 1D/2D models were simulated for 2, 10, 20 and 50-year rainfall events simultaneously. The EADC results show that there is no return period that exceeds the capacity of the drainage system for an investment of \$4.9 million. This confirms that UP measure is capable of minimising flood damages substantially and it can ensure the protection of up to 50-year design event. Similarly, with this investment (i.e., $4.9 million) by implementing DS seven storage tanks can reduce flood damage and contribute to a total protection of up to a 10-year event.

The results obtained through the case study work indicate a promising potential of the proposed approach to achieve optimal solutions (e.g. less damage and lower rehabilitation cost) for different rainfall events. Although the proposed approach does not specifically include aspects such as social and environmental concerns the results demonstrate its usefulness for planning of measures once they have passed evaluation of those concerns. In future research the present approach should be expanded by taking into consideration a variety of structural and non-structural mitigation measures (e.g. green/grey/blue infrastructure measures) within the optimisation process.

5

MODELLING INFILTRATION PROCESS, OVERLAND FLOW AND SEWER SYSTEM INTERACTIONS

Rainfall-runoff transformation on urban catchments involves physical processes governing runoff production in urban areas (e.g., interception, evaporation, depression, infiltration). Some previous 1D/2D coupled models do not include these processes. Adequate representation of rainfall-runoff-infiltration within a dual drainage model is still needed for practical applications. In this Chapter a new modelling setup which includes the rainfall-runoff-infiltration process on overland flow and its interaction with a sewer network is proposed. The performance of an outflow hydrograph generator in a 2D model domain is investigated. The effect of infiltration losses on the overland flow is evaluated through an infiltration algorithm added in a so-called Surf-2D model. Then, the surface flow from a surcharge sewer was also investigated by coupling the Surf-2D model with the SWMM 5.1 (Storm Water Management Model). An evaluation of two approaches for representing urban floods is carried out based on two 1D/2D model interactions. Two test cases are implemented to validate the model. In general, similar results in terms of peak discharge, water depths and infiltration losses against other 1D/2D models are observed. The results from two 1D/2D model interactions show significant differences in terms of flood extent, maximum flood depths and inundation volume.

Based on: Martínez, C., Vojinovic, Z., Price, R., Sanchez, A. (2021). Modelling infiltration process, overland flow and sewer system interactions for urban flood mitigation. *Water* 13 (15) 2028. Doi: 10.3390/w13152028

5.1 Introduction

Hydrological water losses are an important issue within the spatial and temporal distribution of the runoff water in urban catchments. An important component of these losses is the infiltration. Although much of a typical urban area is paved, there has been a growing concern to restore natural infiltration functions and reduce impacts to the catchment by allowing rainwater to gradually infiltrate into the ground.

In urban flood modelling not only the influence of the sewer system in the overland flow is of recognised importance (Mignot et al., 2014; Chen et al., 2015) but also the interaction between surface water and the infiltration losses, in order to better estimate inundation extent and water depths (Mallari et al., 2015; Park et al., 2019; Martínez et al., 2019). It is necessary to provide infiltration input in overland flow models as it plays as a water volume loss that can be defined using empirical laws (e.g. Horton or Green-Ampt equations).

Some of the current included infiltration approaches focus on: (i) hydraulic models for the simulation of flow routing in drainage canals taking into account the infiltration effect with Green-Ampt method (Pantelakis et al. 2013); (ii) estimating the parameters of the Green-Ampt infiltration equation from rainfall simulation data (Van den Putte et al., 2013); (iii) flood routing model incorporating intensive streambed infiltration (Cheng et al., 2015); (iv) rainfall-runoff simulation with 2D full shallow water equations (Fernandez-Pato et al., 2016); (v) modelling two-dimensional infiltration with constant and time-variable water depth (Castanedo et al., 2019) and (vi) investigation of overland flow by incorporating different infiltration methods into flood routing equations (Gülbaz et al., 2020).

New approaches including the influence of the sewer system in the overland flow (coupled 1D/2D model) have also been proposed and applied. Some of the current selection approaches focus on: (i) influence of sewer network models on urban flood damage assessment based on coupled 1D/2D models (Martins et al., 2018), (ii) Multi-objective evaluation of urban drainage networks using a 1D/2D flood inundation model (Martínez et al., 2014b; Martínez et al., 2018a), (iii) the influence of modelling parameters in a coupled 1D/2D hydrodynamic inundation model for sewer overflow (Ganiyu et al., 2015), and (iv) a coupled 1D/2D hydrodynamic model for urban flood inundation (Fan et al., 2017).

Recent progress in urban flood modelling reveals that the above mentioned coupled models are accurate and efficient in simulating floods for practical applications. However, the rainfall-runoff transformation on urban catchments involves physical processes governing runoff production in urban areas, such as interception (on rooftops and on trees), evaporation, depression storage and infiltration. Rainfall-runoff models for urban catchments do not usually include these processes. Previous 1D/2D coupled models compute rainfall-runoff in the 1D sewer network (Seyoum et al., 2012) and, although this is not real world physics, it is a good approximation. Better approaches compute rainfall-runoff into the 2D model domain without considering infiltration losses (Martins et al., 2018; Fan et al., 2017; Leandro and Martins, 2016a; Chen et al., 2007). Adequate representation of rainfall-runoff-infiltration within dual drainage models is still needed within a surface water assessment.

This Chapter aims to develop a new modelling setup which includes rainfall-runoff and infiltration process on the overland flow and its interaction with a sewer system. The key point is to evaluate the proposed model performance when rainfall–runoff and infiltration losses are included in a dual drainage approach, crucial for proper planning and design of urban drainage systems. For this purpose, the performance of an outflow hydrograph generator in a so-called Surf-2D model and used it as an inflow boundary condition is investigated. Its results are compared with the nonlinear reservoir method computed in SWMM 5.1 (Storm Water Management Model). The Surf-2D model is then coupled with SWMM in order to analyse the variation in water depths when overland flow originates not only from rainfall-runoff but also from a surcharge sewer. A benchmark test in Greenfield, Glasgow (UK) produced by the UK Environmental Agency (Néelz and Pender, 2010) is used to examine water depth predictions and flood extents.

The effect of infiltration losses on the overland flow is evaluated through an infiltration algorithm (Green-Ampt method) added to the proposed Surf-2D model. Infiltration is computed in a grid cell using the Green-Ampt method. In order to show the ability to simulate infiltration from a point source direct runoff resulting from a given excess rainfall hyetograph, a validated FullSWOF_2D open source (Delestre et al., 2018) is used to show the performance of the model, computing water depths for different infiltration parameter combinations in a hypothetical case. Finally, an evaluation of two approaches for representing urban floods is carried out based on two main 1D/2D model interactions (e.g., rainfall-runoff computed in 1D sewer model vs. rainfall-runoff-infiltration computed in a 2D model domain) to study differences in terms of flood extent, water depths and flood volumes.

5.2 Methodology

Previous 1D-2D coupled models are combined to simulate the flow dynamics in sewer networks and on the aboveground surface (Martins et al., 2018; Fan et al., 2017; Chen at al., 2007). Approaches to representing urban floods are based on two main 1D/2D model interactions as shown in Figure 5.1.

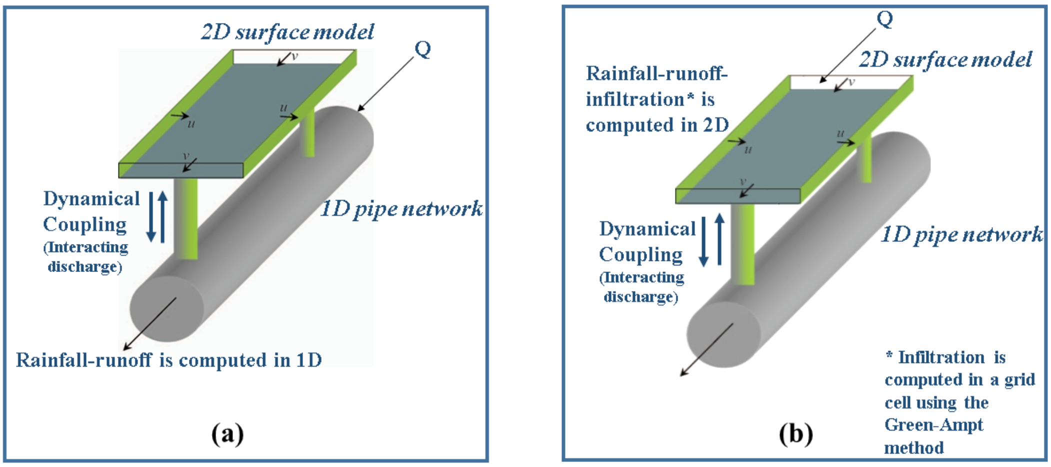


Figure 5.1. Illustration of the 1D/2D model interactions: (a) approach where the rainfall-runoff is computed in a 1D sewer network model (b) an approach based on the effect of rainfall-runoff and infiltration process on the overland flow and its interaction with a sewer network. u and v are the fluxes across cell boundaries of the 2D model.

The 1D rainfall-runoff and 1D pipe network coupling are presented in Figure 1a when the hydrological rainfall-runoff process and routing of flows in drainage pipes are performed using the 1D sewer network. When the capacity of the pipe network is exceeded, flow spills into the 2D model domain from manholes and is then routed by a surface 2D model. The surface infiltration 2D and 1D pipe network coupling are presented in Figure 1b. The model uses a rainfall-runoff infiltration dynamically coupled with a sewer network model. In this study a new modelling setup based on this second interaction has been developed. Figure 5.2 summarizes the whole methodological process.

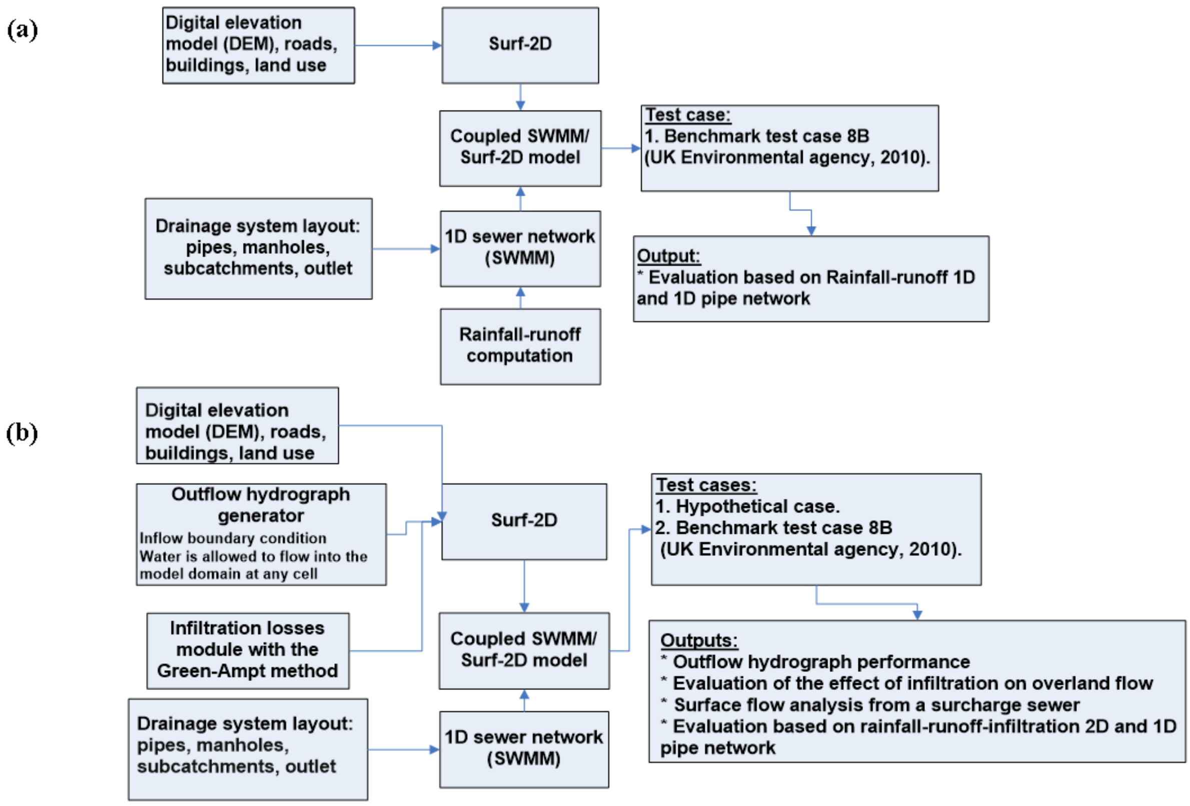


Figure 5.2. Methodological approach

This framework includes the following main components: (1) an outflow hydrograph generator to estimate the direct runoff within the 2D domain, (2) a proposed Surf-2D model to represent the overland flow, (3) an infiltration module based on the Green-Ampt method, (4) a 1D sewer network simulated using SWMM 5.1, and (5) a coupled SWMM/Surf-2D model for representing urban floods. The details of each component are presented through the following sub-sections:

5.2.1 Surf-2D model

In this study, surface water flood simulation builds on the work started in Seyoum et al. (2012). A noninertia model system named subsequently in this work as Surf-2D, was implemented to represent the urban topography, with the ground elevations at the centres and boundaries of cells on a rectangular cartesian grid. This determines the water levels at the cell centres and the discharges (velocities) at the cell boundaries. The system of 2D shallow water equations was obtained by integrating the Navier Stokes equations over depth and replacing the bed stress by a velocity squared resistance term in the two orthogonal directions. The continuity equation for the 2D flood plain flows is expressed as follows:

$$\frac{\partial h}{\partial t}+\frac{\partial (hu)}{\partial x}+\frac{\partial (hv)}{\partial y}=0 \tag{5.1}$$

where h is the water depth and u and v are the velocities in the directions of the two orthogonal axes (the x and y directions), neglecting eddy losses, Coriolis force, variations in atmospheric pressure, the wind shear effect, and lateral inflow; the momentum equation is expressed as in Equation (5.2) for the x direction and Equation (5.3) for the y direction:

$$\frac{\partial (hu)}{\partial t}+\frac{\partial (hu^2)}{\partial x}+\frac{\partial (huv)}{\partial y}+gh\frac{\partial H}{\partial x}+gC_f u\sqrt{u^2+v^2}=0 \tag{5.1}$$

$$\frac{\partial (hv)}{\partial t}+\frac{\partial (huv)}{\partial x}+\frac{\partial (hv^2)}{\partial y}+gh\frac{\partial H}{\partial y}+gC_f v\sqrt{u^2+v^2}=0 \tag{5.2}$$

where H is the water level, g is the acceleration due to gravity and the coefficient C_f appearing in the friction terms is expressed in terms of Chézy roughness. It is known that two-dimensional flow over an inundated urban flood plain is assumed to be a slow, shallow phenomenon (Hunter et al., 2007) and therefore the convective acceleration terms (the second and third terms in Equations (2) and (3)) can be assumed to be small compared to the other terms, and therefore they can be ignored. Expressing the velocities in terms of the discharges and using the Chézy roughness factor, the simplified momentum equations are expressed as Equations (5.4) (x direction) and (5.5) (y direction).

$$\frac{\partial}{\partial t}\left(\frac{Q}{Z_Q}\right)+\Delta Y g\frac{\partial H}{\partial x}+g\frac{Q}{C^2 Z_Q^2}\left(\left(\frac{1}{\Delta Y}\frac{Q}{Z_Q}\right)^2+\left(\frac{1}{\Delta X}\frac{R}{Z_R}\right)^2\right)^{0.5}=0 \tag{5.3}$$

$$\frac{\partial}{\partial t}\left(\frac{R}{Z_R}\right)+\Delta X g\frac{\partial H}{\partial y}+g\frac{Q}{C^2 Z_R^2}\left(\left(\frac{1}{\Delta Y}\frac{Q}{Z_Q}\right)^2+\left(\frac{1}{\Delta X}\frac{R}{Z_R}\right)^2\right)^{0.5}=0 \tag{5.4}$$

where H is the water level, Q and R are the discharges in the directions of the two orthogonal axes (the x and y directions), ΔX and ΔY are the grid spacings in the x and y directions, Z_Q and Z_R are the water depths at the cell boundaries, g is the acceleration due to gravity, and C is the Chézy friction factor.

Numerical solution

The above conservation of mass and momentum equations (the Saint-Venant equations) given in discretized form were solved using the alternating direction implicit scheme (ADI algorithm). In the ADI algorithm, the solution procedure includes the computation of conservation of mass and conservation of momentum in the corresponding direction and, in the other direction, the conservation of mass is once more computed, but now with the conservation of momentum for that direction (See, Seyoum et al., 2012). The main features of the Surf-2D model includes a two-point forward spatial and temporal difference scheme adopted on the basis of a uniform time step $\Delta t = t_n + 1 - t_n$, in which n is the time step counter

.

Wetting and drying

The water depth of a grid cell is then calculated as the average depth over the whole cell. When the cell first receives water, the wetting front edge usually lies within the cell. In most cases, only part of the cell will be wetted at that time step. When the flow volume leaving a cell is more than that entering the cell, the cell dries and there is the possibility that the water depth may be reduced to zero or a negative value (Seyoum et al., 2012).

In order to avoid negative depth values, the wetting process is controlled by a wetting parameter. When the cell is wetting, the water should not be allowed to flow out of the cell until the wetting front has crossed the cell (Yu and Lane, 2006); each cell has a property called percentage wet when the cell is first wetted, as follows:

$$percentage\ wet = min\left(1, \frac{\sum(v\Delta t)}{\Delta x}\right) \tag{5.5}$$

where v is the velocity computed from the discharge crossing the cell boundary divided by the cell width and the cell flow depth; Δx is the cell width and Δt is the current time step. Water is not allowed to flow out of the cell until the wetting parameter reaches unity. The wetting parameter is updated at each time step to describe the water traveling across a cell. The whole surface of the cell is used as active infiltration surface, even if rainfall intensity is zero and the cell is only partially wet. In terms of the numerical scheme, the model has the ability to halve or double the time step, halving to meet the convergence criterion, and doubling after a certain number of time steps without halving.

5.2.2 Infiltration algorithm

A proposed infiltration algorithm was incorporated into the Surf-2D model code. In this case, infiltration is computed in a grid cell using the Green-Ampt method (Green and Ampt, 1911; Mein and Larson, 1973). The cumulative depth of water infiltrated from the soil and the infiltration rate form of the Green-Ampt equation for the one-stage case of initially ponded conditions, and assuming the ponded water depth is shallow, is given in Equations 5.7 and 5.8.

$$f(t) = k_e \left[1 + \frac{\Psi\theta_d}{F(t)}\right] \tag{5.6}$$

$$F(t) = k_e t + \Psi\theta_d ln\left[1 + \frac{F(t)}{\Psi\theta_d}\right] \tag{5.7}$$

where $f(t)$ is the infiltration rate (mm/h), *F(t)* is the cumulative infiltration depth (mm), k_e is the effective saturated conductivity (mm/h), θ_d is the moisture deficit (mm/mm), t = time and Ψ depends on the soil and represents the suction head at the wetting front (mm). The ponded water depth h_o computed at the surface of the cell, as described in the previous section and now also available, is assumed to be negligible compared to Ψ as it becomes surface runoff. However, in the case when the ponded depth is not negligible, the value of Ψ-h_o is substituted for Ψ for infiltration computation at time t_n in Equations 5.7 and 5.8 (Delestre et al., 2018; Chow, 1988).

Equations 5.7 and 5.8 have been solved within the Surf-2D model from a quadratic approximation of the Green-Ampt equation based on the power series expansion presented first by (Li et al., 1976). Stone et al. (1994) derived their approximation based on two first terms in a Taylor series expansion and it was presented as the modified Equation as follows:

$$F_q^*(t_c^*) = 0.5\left(t_c^* + \sqrt{t_c^*(t_c^* + 8)}\right) \tag{5.8}$$

where $F_q^*(t_c^*)$ is the quadratic approximation of infiltrated depth for the case of the initial ponded conditions and t_c^* is the corrected time (dimensionless). The Taylor series expansion is given as follows (Kale and Sahoo, 2011):

$$\begin{gathered} F_{pr}^*(t_c^*) = t_c^* + (2t_c^*)^{1/2} \text{-}\ 0.2987(t_c^*)^{0.7913} \\ t_c^* = \frac{k_e(t + t_s - t_p)}{\Psi\theta_d} \\ f_{pr}^*(t_c^*) = 1 + \frac{1}{F_{pr}^*(t_c^*)} = \frac{f(t)}{k_e} \\ F_{pr}^*(t_c^*) = \frac{F(t)}{\Psi\theta_d} \end{gathered} \tag{5.9}$$

where $F_{pr}^*(t_c^*)$ is the resulting approximation of the Taylor series for cumulative infiltrated depth; t_s is the time shift, termed as "pseudo time" used as a correction for considering the cumulative infiltrated depth of water at the time of ponding during an unsteady rainfall event; and t_p is the time to ponding (Stone et al., 1994) to the quadratic approximation of Li et al. (1976) has less error within range values of the ratio of cumulative depth to capillary potential of 0.5 to 150. This range roughly corresponds to coarser textured error soils. Outside this range, both approximations result in small absolute error.

In the Surf-2D model, infiltration is calculated by taking into account the computed velocity at which water enters into the soil (infiltration rate) in the corresponding grid cell (area of the grid) per unit time. This is treated as a discharge point sink within the same time interval. The water infiltration is assumed to be one-dimensional, and thus there is no lateral drainage**.** To avoid an infinite infiltration rate initially (when the infiltrated volume is still equal to zero), we add a threshold to obtain the infiltration rate $f = min\ (inf\ capacity,\ i_{max})$. Because the infiltrated volume cannot exceed the water depth (h) at the surface of the cell that is available for infiltration at time t_n, the volume is updated as shown in Equation 5.11. Finally, the water depth is updated.

$$V_{inf}^{n+1} = V_{inf}^{n} + \min(h, f * \Delta t) \tag{5.10}$$

5.2.3 Rainfall-Runoff process

The performance of an outflow hydrograph generator in a 2D model domain was investigated, instead of adding the rainfall rate directly to each cell as a mass source term, as is commonly performed (Martins et al., 2018; Fan et al., 2017; Delestre et al., 2018). In this study an alternative to obtain direct surface runoff resulting from a given excess rainfall hyetograph was obtained by applying the Soil Conservation unit hydrograph known as the SCS-UH method (SCS, 2012). Figure 5.3 shows the surface runoff representation in a cell.

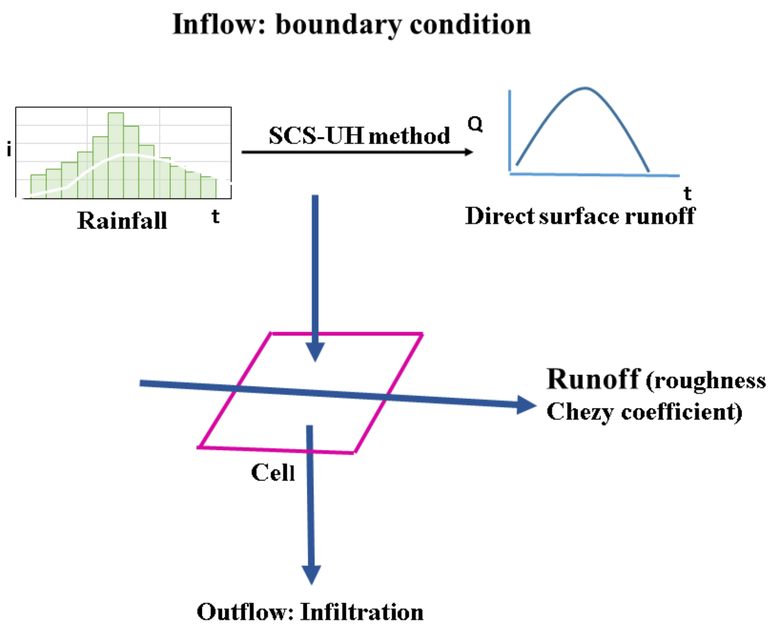


Figure 5.3 Surface runoff representation in a cell

The SCS-UH method is a synthetic unit hydrograph, in which the discharge is expressed by the ratio of discharge q to peak discharge qp and the time by the ratio of time t to the time of the unit hydrograph rising, Tp. Given the peak discharge and lag time for the duration of excess rainfall, the unit hydrograph can be estimated from the synthetic dimensionless hydrograph (see for example, Chow, 1988). In this case the direct runoff hydrograph accounts for the direct surface runoff, i.e., rainfall minus abstractions or losses, such as initial losses (interception and ponding, considered lower in urban compared to rural areas) and infiltration losses.

Initial losses such as rainfall interception from rooftops, urban trees and depression storage at the start of the design storm are defined as that part of rainfall retained by aboveground objects until it returns to the atmosphere through evaporation. These initial losses have been taken into account in the unit hydrograph computation following the observations found in the literature and recapitulated in Rammal and Berthier (2020), expressed as water depth by unit of surface during a frequent rain-event. Infiltration contributes to runoff losses during and after the rainfall event and has been considered thus, as presented in Section 5.2.2.

The corresponding hydrograph obtained as a result of this process was used as an inflow boundary condition in the Surf-2D model. This means that water is allowed to flow into the model domain at any cell. This can occur either as a source or sink in a cell, with the flow having no horizontal momentum contribution (such as rainfall or flow at surcharged manholes of a drainage network model based on, say, SWMM) or at a cell boundary, in which, in this case, the contribution of the momentum of the inflow is included.

5.2.4 1D Sewer model

SWMM is a dynamic sewer network model that solves the conservation of mass and momentum equations (1D Saint-Venant equations). The model governs the unsteady flow of water through a drainage network of channels and pipes by converting the equations into an explicit set of finite-difference equations. The network system is presented as a set of links which are connected at nodes. Links transport flow from node to node and these nodes are modelled as storage elements in the system. It is assumed that the runoff surface area of a node is equivalent to the surface area of the node itself plus the surface area that is contributed by half of each conduit connected to the node (Rossman, 2017). Continuity and momentum equations are used in the dynamic wave routine at the links, and the continuity equation is used at the nodes. This routing method can account for channel storage, backwater, entrance/exit losses, flow reversal and pressurized flow. The Saint Venant equations and their solution method as implemented in SWMM 5.1 are described in Rossman (2017).

SWMM defines a node as being surcharged condition when all the conduits connected to it are full or when the node’s water level exceeds the crown of the highest conduit connected to it (Rossman, 2017). During the surcharge and in order to prevent the surface area at the node from becoming zero (0), a limit on the full conduit width is set, equal to the width when the conduit is 96% full, the so-called minimum full conduit width parameter (Leandro and Martins, 2016a). To guarantee the mass conservation, a perturbation Equation is:

$$\Delta H = \frac{-\sum Q}{-\sum \partial Q/\partial H} \tag{5.12}$$

The gradient of flow in a conduit with respect to the head at either end node can be evaluated by differentiating the flow updating the link momentum equation (Rossman, 2017), resulting in:

$$\frac{\partial Q}{\partial H} = \frac{-g\bar{A}\Delta t/L}{1 + \Delta Q_{friction}} \tag{5.11}$$

where Q is flow rate and H is the hydraulic head of water in the conduit. $\bar{A}$ is the average flow area, Δt is the time step (sec), L is the conduit length (m), ΔH is the adjustment to the node´s head that must be made to achieve flow continuity, and $\sum Q$ is the net flow into the node (inflow – outflow) contributed by all conduits connected to the node as well as any externally imposed inflows. $\partial Q/\partial H$ has a negative sign in front of it because, when evaluating $\sum Q$, flow directed out of a node is considered negative while flow into the node is positive. Equation 5.12 is used whenever heads need to be computed in the successive approximation scheme developed for surcharge flow (Rossman, 2017).

5.2.5 Models’ linkage

In order to simulate the interaction between the sewer network and surface flow, the Surf-2D model was coupled with the SWMM 5.1 open source code through a series of calls built inside a dynamic link library – DLL (Rossman, 2017). The models’ linkage follows the work of Leandro and Martins (2016a). To avoid modifications in the original SWMM code, additional functions inside its DLL file were added to feed the interface communication with the Surf-2D model.

The coupling includes two extra functions for exchanging information between the two models. The first function extracts the node water levels and node depth during every SWMM simulation time, and this function also takes each node ID as inputs to deal with flows. The second function exchanges discharges (node inflow and outflow) between both models. The discharge values can be either positive or negative depending on whether water is being transferred from or to the Surf-2D model. As it was stated above, direct surface runoff resulting from a given excess rainfall hyetograph is added directly into the Surf-2D model thus SWMM computes the dynamic sewer network flow and its hydrological runoff module was not used.

The Surf-2D model includes two subroutines suggested in previous publications (Chen at al., 2007; Leandro and Martins, 2016a). These subroutines calculate the bidirectional discharge between the two models. Discharge between the two models is assumed to take place at the manholes. However, in reality the catchment plays the major role in this. As such, the sewer drainage efficiency depends on the instantaneous and available sewer drainage capacity and the overland flow paths overlying the location of the manholes. Manhole discharges are treated as point sinks or sources in the 2D model within the same time interval as follows.

Drainage condition

When the water level on the ground surface (h_{2D}) is higher than the hydraulic head at the manhole (h_{1D}) and the ground surface elevation (Z_{2D}), the runoff from the surface flowing into the manhole is determined by either a weir equation if the pressure head in the manhole is below ground surface elevation $h_{1D} < Z_{2D}$ (Equation 5.14), or an orifice equation if pressure head in the manhole is above the ground elevation $h_{1D} > Z_{2D}$ (Equation 5.15).

$$Q = c_w w h_{2d}\sqrt{2gh_{2D}} \tag{5.14}$$

$$Q = c_0 A_{mh}\sqrt{2g(h_{2D} + Z_{2D} - h_{1D})} \tag{5.15}$$

Where Q is the interacting discharge (m^3/s), c_w is the weir discharge coefficient. With this form of equation, c_w has a value between 0.6 and 0.7 (0.6), w is the weir crest width (m), A_{mh} is the manhole area (m^2), and c_o is the orifice discharge coefficient with values between 0.6 and 0.62 (0.62). The numbers in parentheses were used as initial values.

Surcharge condition

Surcharge is determined based on an orifice equation if $h_{2D} < h_{1D}$ (Equation (5.16).

$$Q = -c_o A_{mh}\sqrt{2g(h_{1D} - Z_{2D} - h_{2D})} \tag{5.16}$$

The timing synchronization also becomes an important issue for connecting both models appropriately. Because the sewer network model SWMM and the Surf-2D model use different time steps, the 2D model time step was restricted just before the synchronization time to the value given by applying Equation 5.17. This time-synchronisation technique can be found in detail in Chen et al. (2007).

$$\Delta t_{2Dm+1} = \min\left\{\left(T_{sync} + \Delta t_{1D} - \sum_{i=1}^{m} \Delta t_{2D}\right), \Delta t *_{2Dm+1}\right\} \tag{5.17}$$

Where Δt_{2Dm+1} is the time step size [s] used for the $m+1^{th}$ step, T_{sync} is the time of the previous synchronization [s], $\sum_{i=1}^{m} \Delta t_{2D}$ is the total duration of the time step [s] after m step of the surface water model computation following the last synchronization, Δt_{1D} is the time step used in SWMM, and $\Delta t *_{2Dm+1}$ is the time step duration determined by the Surf-2D model for the $m+1^{th}$ step.

5.3 Test cases

The formerly 2D model was previously tested in Seyoum et al. (2012) with a benchmark case where a wave propagation down a river valley was simulated. To assess the model performance of the proposed Surf-2Dmodel, two tests were selected which enable the studying of specific urban flood aspects and verification of model accuracy.

The first test is a hypothetical case to evaluate the performance of the unit hydrograph. To this purpose, the unit hydrograph (SCS-UH) computed was initially compared with the nonlinear reservoir routing method implemented in SWMM. Figure 5.4a presents this case; it is a 40m by 32m grid plane (8km^2) with a cell size of 2.5 by 2.5 m and slope of this area equal to 0.007m.

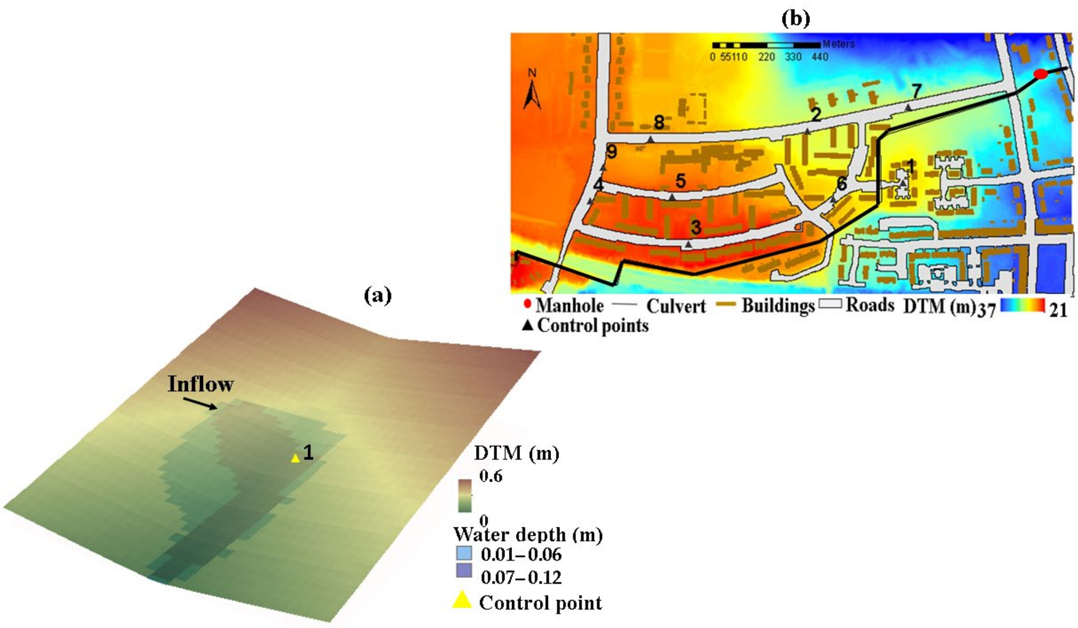


Figure 5.4. Test cases (a) hypothetical case; (b) Benchmark test "case 8B": Greenfield, Glasgow (UK)

The second test was taken from the Benchmark test "case 8B" carried out by the UK Environmental Agency (Néelz and Pender, 2010). This corresponds to a hypothetical event happening in the region of Greenfield, Glasgow, UK (see Figure 5.4b). This test has the objective of assessing the performance of the proposed SWMM/Surf-2D coupled models in terms of water depth predictions and flood volume.

A range of software packages, referred to as shallow water equations, "Full models" (i.e., InfoWorks, ISIS, TUFLOW, MIKE FLOOD and SOBEK) and a "simplified model" (some terms of the equations are neglected and simplified equations are solved) known as a UIM model are included in this benchmark test (Néelz and Pender, 2010). The infiltration process was not considered in any of these models, so that the infiltration module implemented into the proposed Surf-2D model was not used in this test.

The characteristics of overland flow and the variation in water depths are examined when the overland flow originates not only from rainfall–runoff but also from a surcharging underground pipe. An inflow boundary condition is applied at the upstream end of the pipe. A surcharge is expected to occur at a vertical manhole of 1m^2 cross-section located 467m from the top end of the culvert. The flow from the above surcharge spreads across the surface of a 2m resolution DTM created from LIDAR data.

A land cover dependent roughness value was applied with two categories: (1) roads and pavement; (2) any other land cover type. A uniform rainfall of 400 mm/h with 4 min duration and starting at minute 1 was applied with a total simulation time of 5 hours to produce direct surface runoff. Similarly, an inflow boundary condition was applied at the upstream end of the pipe (1D model), with a surcharge expected to occur at the manhole.

As was stated in this Section, the benchmark "Test 8B" model packages do not include infiltration processes. For this reason, in order to assess the Surf-2D model´s ability to simulate infiltration from a point source (direct runoff resulting from a given excess rainfall hyetograph), the validated open source Full Shallow Water equations for Overland Flow in two dimensions of space FullSWOF_2D was used for comparison purposes (Delestre at al., 2018). Several features make FullSWOF_2D particularly suitable for applications in hydrology. Small water depths and wet–dry transitions are robustly addressed, rainfall and infiltration (Green-Ampt method) are incorporated, and data from grid-based digital topographies can be used directly. In this software, the shallow water (or Saint-Venant) equations are solved using finite volumes and numerical methods, especially chosen for hydrodynamic purposes (transitions between wet and dry areas, small water depths, and steady-state preservation).

5.4 Results and discussion

5.4.1 Performance of the outflow hydrograph generator

The performance of the outflow hydrograph generator was assessed by the hypothetical case presented in Section 5.3. The initial losses were set to 0.62 mm for both methods following the reviewed values in Rammal and Berthier (2020). This value corresponds to rainfall interception from rooftops, urban trees and depression storage at the start of the design storm. For the nonlinear reservoir, infiltration losses were computed assuming a silt soil class with the following Green-Ampt (GA) parameter values: k_e = {5 mm/h}; θ_d = {0.5}; Ψ = {190 mm}. The rainfall intensity was assumed as 70 mm/h at 1-hour duration. The result of this process is a hydrograph comparison between the two surface runoff methods (e.g., SCS-UH vs. nonlinear reservoir) as presented in Figure 5.5.

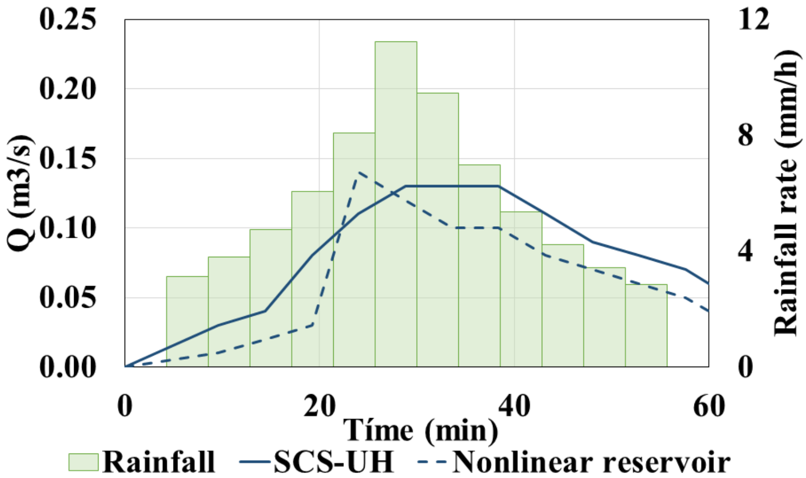


Figure 5.5. Surface runoff methods comparison for the hypothetical case

The overland flow rate obtained by applying the SCS-UH method differs slightly around the peak of the hydrograph from the nonlinear reservoir method computed in SWMM. Discharge values in the unit hydrograph method are ±7% higher in comparison to those obtained with the nonlinear reservoir. This can be associated to the different hydrological considerations taken for runoff generation such as the assumed initial losses value. It could also be due to the fact that the infiltration losses were not considered in the unit hydrograph method, as infiltration is computed with the 2D algorithm in the Surf-2D model. However, in general both methods are in good agreement. The corresponding generated hydrograph was then used as an inflow boundary condition in the Surf-2D model for the following analysis.

5.4.2 Evaluation of the effect of infiltration

This test aims to assess the Surf-2D model´s ability to simulate infiltration from a point source, direct runoff resulting from a given excess rainfall hyetograph. The hypothetical case (Figure 5.4a) was applied with an assumed 70 mm/h rainfall intensity of 1-hour duration.

As a sensitivity analysis, Figures 5.6a, 5.6b and 5.6c show the comparison of computed water depths for different GA infiltration parameters combinations (k_e, θ_d, Ψ) according to (Rawls et al., 1983; Chow, 1988). The values of each parameter correspond to sand, silt and clay soil classes, respectively. The obtained water depths were compared to those obtained with FullSWOF_2D software. No groundwater component (neither physically nor parametrized) has been included in both models.

The water depth results consist of those predicted by FullSWOF_2D for different infiltration parameters combinations (Figure 5.6). This can be inferred by the R^2 grader at 0.88, the RMSE error statistic which exhibits a small error of an average of 0.011 m, and the total volume difference between Surf-2D and FullSWOF_2D. It is in the order of 4% (see Table 5.1). The good match can be associated to the similar finite volumes and numerical methods applied in both models to solve the shallow water (or Saint-Venant) equations and to the method used to compute infiltration. These results show the importance, when evaluating the performance of the Surf-2D model, of computing infiltration.

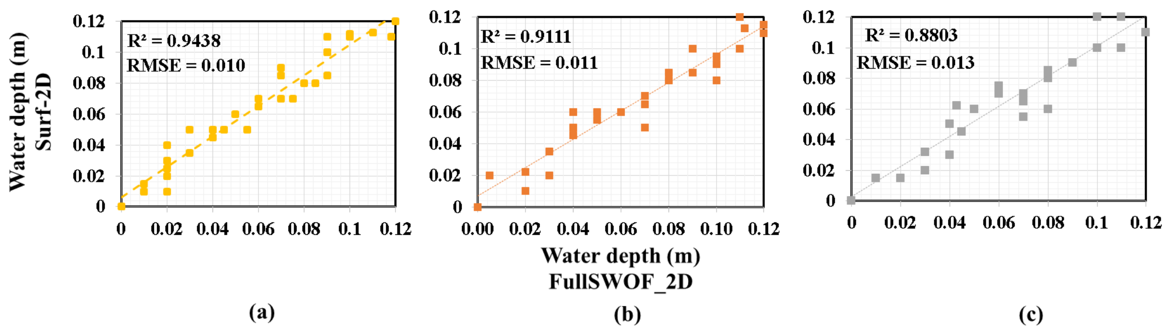


Figure 5.6. Water depths comparison between Surf-2D and FullSWOF_2D for different GA infiltration parameters ***(a)*** *k_e = {117,8mm/h}; θ_d = {4.17}; Ψ = {49.5mm};* ***(b)*** *k_e = {6.5mm/h}; θ_d = {4.86}; Ψ = {166.8mm};* ***(c)*** *k_e = {0.3mm/h},θ_d = {3.85}; Ψ = {316mm}.*

Table 5.1. Comparison of the overland surface volume differences between Surf-2D and FullSWOF_2D.

Infiltration Parameters	**Surf-2D Volume at the Surface (m^3)**	**FullSWOF_2D Volume at the Surface (m^3)**	**RMSE - Water Depths (m)**
k_e = {117,8mm/h}; θ_d = {0.417}; Ψ = {49.5mm	4570	4760	0.010
k_e = {6.5mm/h}; θ_d = {0.486}; Ψ = {166.8mm}	4619	4810	0.011
k_e = {0.3mm/h}; θ_d = {0.385}; Ψ = {316mm}	4667	4860	0.013

Figure 5.7 presents the results comparison between the Surf-2D model and FullSWOF_2D in a single grid (point 1, Figure 5.4a) according to the different soil types given in Table 5.2.

Table 5.2. Green-Ampt parameter values used (Chow, 1988; Zhan et al., 2007).

Soil Texture	k_e (mm/h)	Ψ (mm)	θ_d (mm/mm)
Sandy loam	22	90	0.5
Silt	5	190	0.5
Silt clay loam	1.8	253	0.2

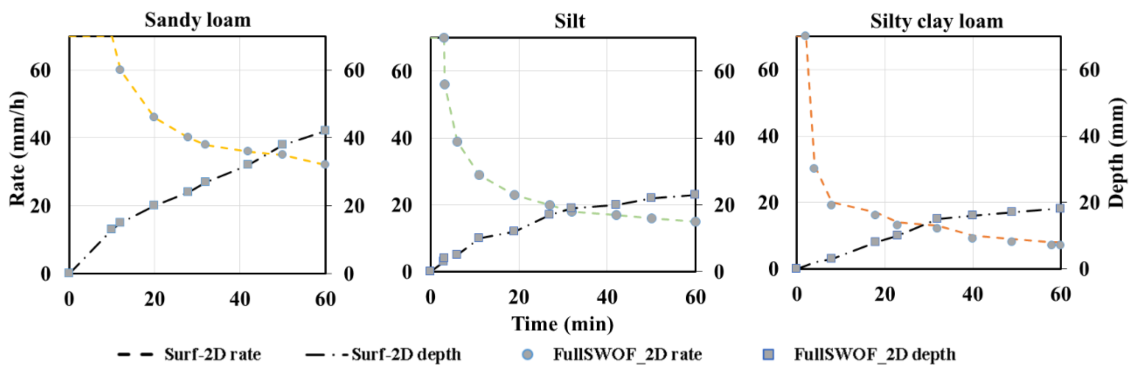


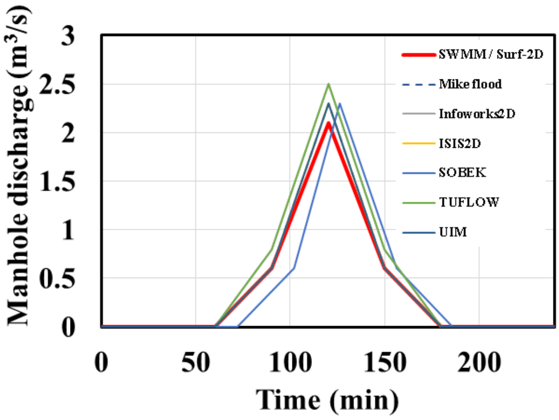
Figure 5.7. Hypothetical case: infiltration rate and depth results in a single grid (point 1)

A surface with high infiltration capacity (sandy loam) reduces the surface runoff generated to 36%, and with low infiltration capacity (clay loam) runoff has also been decreased to 27%. The results show the importance of the soil type in determining the overland flow, as this governs the infiltration capacity limits. The hydraulic conductivity is a dominant parameter as it defines the maximum infiltration capacity of the soil, as also presented in (Gülbaz et al., 2020).

The infiltration rate in a single grid (see control point 1, Figure 5.4a) was found to be reduced at a decreasing rate at a time up to 20 minutes. It shows an almost steady state after 30 minutes of continuous ponding. The infiltration depth in a single grid was also found to increase at a decreasing rate, which is consistent with previous investigations (Stone et al., 1994; Zhan et al., 2007). Figure 5.7 also shows the logical hydraulic properties of soil from the highest to the lowest: sandy loam and clay loam, respectively (see also Hossain and Lu, 2014).

5.4.3 Surface flow from a surcharge sewer

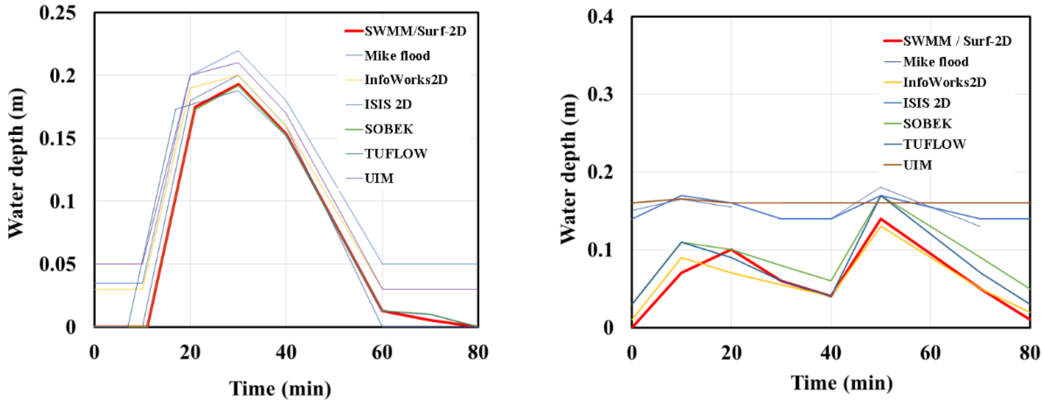
This section evaluates the capability of simulating shallow inundation, originating from a surcharging underground pipe. The benchmark test case in the region of Greenfield, Glasgow (UK) and described in Section 5.3 was applied. Figure 5.8 presents the manhole discharge results using the coupled SWMM/Surf-2D model and its comparison with the mentioned diffusive and dynamic models. Due to the fact that the infiltration process was not considered in any of the benchmark models, the infiltration module implemented in the proposed Surf-2D model was not used for this analysis. Final results have been overlapped with previously published results from the software packages (Néelz and Pender, 2010).



Model	Volume (m^3)
SWMM / Surf-2D	5700
Infoworks	5873
ISIS	5864
TUFLOW	5837
UIM	5226
MIKE FLOOD	5024
SOBEK	4987

Figure 5.8. Manhole discharge predicted by SWMM/Surf-2D superimposed with the results from the models published in the EA benchmark 8B.

The SWMM/Surf-2D model predicts similar results in terms of peak discharge at the manhole, as can be seen in Figure 5.8, although volumes differ within a 12% range (e.g., 5700 m^3 for SWMM/Surf-2D model and 5024 m^3 for Mike flood). Figure 5.9a, and 5.9b present water depths at points 7 and 9, which correspond to a green area (See Figure 5.4b).



(a) (b)

Figure 5.9. Water depths predicted by SWMM/Surf-2D superimposed with the results from the models published in the EA benchmark 8B. (a) Water depths at point 7; (b) water depths at point 9.

All models agreed in the prediction of the peak levels (red line). Maximum depths did not exceed 0.23 m at point 7. The coupled model also agrees in the prediction of water depths compared to the other models, and water depths did not exceed 0.20m at point 9. However, the UIM model results do not predict peak levels showing a quasi-constant water depth. This could be a consequence of the scale of the test used here, over smaller domains than one would typically apply a simplified model to (see, Néelz and Pender, 2010). UIM as a simplified model solves the 2D diffusion wave equation which is obtained by neglecting the acceleration terms in the 2D shallow water equations.

In terms of run times, Table 5.3 presents the efficiency of the proposed SWMM/Surf-2D coupled model compared to the model results reported in (Néelz and Pender, 2010).

Table 5.3. Summary of runtimes.

Model	Time-Stepping	Runtime (min)
SWMM/Surf-2D	1s	18.0
InfoWorks	Adaptive	6.0
ISIS	0.05s	734.30
TUFLOW	1s	9.20
UIM	Adaptive	743.30
MIKEFLOOD	1s	2.08
SOBEK	5s	18.9

Efficiency obtained herein with SWMM/Surf-2D exhibits similar run times compared to SOBEK and performs better than ISIS and UIM models. However, TUFLOW, InfoWorks and MIKEFLOOD perform better than the method herein mentioned. According to Néelz and Pender (2010), possible explanations for this run times variations include the choice of the time step partly imposed by the numerical approach, the number of iterations performed at each time step, and the efficiency of the numerical algorithm and hardware specification.

5.4.4 Evaluation of two approaches for representing urban floods

The two 1D/2D model interactions illustrated in Figure 5.1 have been evaluated on the benchmark test “case 8B”. Initial losses for the unit hydrograph were set to 0.65mm. The distributed hydraulic conductivity (k_e) in the model for roads–pavement and green areas was set to a very slow infiltration 1.0 mm/h and moderate infiltration 6.5 mm/h, respectively. Figure 5.10 presents the top view of the flood inundation extent in the region of Greenfield, Glasgow to evaluate two different flood modelling interactions. Flood is condensed across a highway going east to west. The flat slope, especially along the street, facilitates the overland flow’s inundation along the street parallel to the main highway.

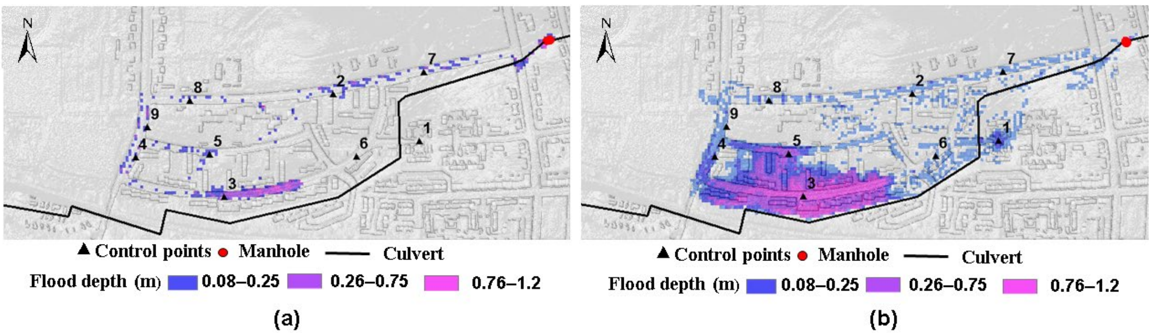


Figure 5.10. Approaches for representing urban floods: (a) rainfall-runoff is computed in 1D culvert, flood extent presented in 2D. (b) Rainfall-runoff and infiltration computation accounted in Surf-2D model and its interaction with the culvert.

The first 1D/2D model interaction is presented in Figure 5.10a. The rainfall-runoff process is performed in the 1D culvert. When the capacity of the pipe network is exceeded flow spills into the 2D model domain from manholes and is then routed by a surface 2D model. In this case, the areal extent, maximum flood depth and volume of the inundation region is about 624 m^2, 0.38 m and 187 m^3, respectively.

The second 1D/2D model interaction, based on rainfall-runoff and infiltration computation. accounted for with the Surf-2D model, is presented in Figure 5.10b. The culvert initially has no water to simulate overland flow draining back to the system. Here, the impact of rainfall–runoff and infiltration in the 2D domain is presented. An increase in the areal extent (4800 m^2), maximum flood depth (0.9 m) and volume (3600 m^3) around control point 3 is obtained (Figure 5.10b). Significant differences compared to those obtained in Figure 5.10a are shown. This approach 2 estimates the areal extent and flood depth caused by the rainfall-runoff, and the infiltration and excess flow from manholes during a flood event (flood inundation). After the event is over, water eventually drains back to the system through the downstream manhole (drainage condition). The inundation region is now about 2820 m^2 of areal extent, 0.8m of maximum flood depth and 1830 m^3 of volume.

The above results show that differences between the two types of interactions are significant. For instance, differences by around 78% in terms of flood extent, 48% in the maximum flood depths and 90% in inundation volume were found. Integrated modelling approaches are being increasingly promoted as required in order to holistically evaluate urban water systems while facilitating infiltration in urban areas. In terms of modelling, the challenge today is to move from individual considerations of urban drainage system (UDS) performance to integrated applications that include not only the interaction between the sewer network and surface flow (1D/2D coupled models) but also the inclusion of the rainfall–runoff and infiltration process for a better evaluation of the system. Similarly, the modelling of green infrastructure or Natural Based Solutions (NBS) within the evaluation of an UDS needs to be included (Martínez et al., 2018b). The above test results demonstrate an acceptable tool as an advance for further analysis of the performance of these type of infrastructures.

5.5 Conclusions

In this paper an approach to couple rainfall–runoff-infiltration and sewers is presented. To achieve this, an infiltration module algorithm based on the Green and Ampt method was coded into a model called Surf-2D. Infiltration was calculated by taking into account the computed infiltration rate. Direct surface runoff resulting from a given excess rainfall hyetograph was also computed by applying the unit hydrograph method. The corresponding hydrograph was used as an inflow boundary condition in the 2D domain for each test. The model was then coupled with SWMM 5.1 open source code through a series of calls built inside a dynamic link library-DLL. The presented modelling setup was validated with two cases: a hypothetical case and the real case of Greenfield, Glasgow (UK). A surcharging case came from an EA benchmark report, with the latest validated free software FullSWOF_2D and with two different approaches to representing urban floods.

The following conclusions are reached: the unit hydrograph (SCS-method) implemented was indeed effective for the purpose of producing direct runoff in the 2D model domain compared with the non-linear reservoir method. Despite having different hydrological considerations for runoff generation compared to the nonlinear reservoir method, both methods' results were similar. The inclusion of the Green-Ampt method in the 2D domain had a direct impact on the overland flood-depths. Although determining soil properties may sometimes be difficult for the application of the method, the presented model was capable of reproducing the influence of the infiltration capacity of the soil on the overland flow. As was observed in Figure 5.7 for a grid cell, the model follows a reasonable range of soil hydraulic properties from the highest to the lowest, sandy loam and clay loam, respectively (i.e., from high infiltration capacity with a sandy loam to low infiltration capacity with a clay loam soil type).

In the benchmark test “case 8B” in the region of Greenfield, Glasgow, the 1D sewer model contributed more reliable analyses of flooding processes due to their impact on the overland flood-depths. The presented model predicted similar results of the software packages (EA benchmark report), compared in terms of peak water depths within a range of a few centimeters. Two approaches for representing urban floods were tested in this work, leading to different flood evolution results. The direct impact of rainfall–runoff and infiltration using benchmark Test 8B allowed the provision of realistic flood volume and gradual recession after the flood peak occurs.

Finally, the presented coupled SWMM/Surf-2D model with the incorporation of rainfall–runoff and infiltration process showed a basis for addressing a better evaluation of urban floods and, in turn, holistically evaluate an urban drainage system.

6

MULTI-OBJECTIVE MODEL-BASED ASSESSMENT OF GREEN-GREY INFRASTRUCTURES

This Chapter presents the performance quantification of different green-grey infrastructures, including rainfall-runoff and infiltration processes, on the overland flow and its connection with a sewer system. The present study suggests three main components to form the structure of the proposed model-based assessment. The first two components provide the optimal number of green infrastructure (GI) practices allocated in an urban catchment and optimal grey infrastructures, such as pipe and storage tank sizing. The third component evaluates selected combined green-grey infrastructures based on rainfall-runoff and infiltration computation in a 2D model domain. This framework was applied in an urban catchment in Dhaka City (Bangladesh) where different green-grey infrastructures were evaluated in relation to flood damage and investment costs. These practices implemented separately have an impact on the reduction of damage and investment costs. However, their combination has been shown to be the best action to follow. Finally, it was proved that including rainfall-runoff and infiltration processes, along with the representation of GI within a 2D model domain, enhances the analysis of the optimal combination of infrastructures, which in turn allows the drainage system to be assessed holistically.

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6.1 Introduction

Retrofit solutions for the management of urban infrastructure have been successfully applied in cities worldwide (Environmental Agency, 2007). They have been proven to be a cost-effective solution to manage flood risk, whilst also delivering a range of other benefits (Ashley et al., 2014; Staddon et al., 2017). These solutions include constructed structures such as treatment facilities, sewer systems, storm water systems, and storage basins, which are known as grey infrastructure. A strategically planned network has also been used as an approach that projects, restores, or mimics the natural water cycle, also known as green infrastructure (GI). Previous implementation of these practices suggests that the combined green-grey measures turned out to be more effective than the grey-only option (Cohen et al., 2011; Dong et al., 2017).

Projects attempting to enhance the performance of retrofit solutions in urban catchments have discovered significant improvements, focusing on: (i) overcoming uncertainty and barriers using blue-green infrastructures for risk management (Thorne et al., 2015; Onuma and Tsuge, 2018), (ii) proposed frameworks to assess green infrastructure to mitigate urban flood hazards (Schubert et al., 2017; Joyce et al., 2017), (iii) modelling the interference of underground structures by groundwater flow and potential remedial solutions for this (De Caro et al., 2020), and (iv) integrating strategies to improve the microclimate regulation of green-blue-grey infrastructures in specific urban forms (Li et al., 2020). The results of these studies have produced, among others, a comprehensive evaluation of the integration of green-grey practices.

The use of numerical models has proved to be invaluable for dealing with urban water management issues (Ferrano et al., 2020; Costabile et al., 2020). A fast assessment framework to generate evidence for comparing strategies at low resource cost during the initial design has been carried out by Webber et al. (2018). This provides evidence to identify performance trends and consider resilience to extreme events at an early stage of planning. The impact of mitigation measures and infiltration on flash floods has been investigated by Tügel et al. (2020). A 2D robust shallow-water model including infiltration with the Green-Ampt model was used for this purpose. This model can help to define appropriate locations and dimensions of these mitigation measures.

It is possible to explore the performance of urban infrastructure with the inclusion of optimisation techniques. Previous research has implemented a multi-objective evolutionary algorithm optimization to evaluate the effectiveness of different intervention measures (Barreto et al., 2010; Giacomoni, 2015; Martínez et al., 2018a; Piscopo et al., 2018), investigate the likelihood of green infrastructure enhancement using hybrid models and machine learning techniques (Labib, 2019; Bakhshipour et al., 2019; Yoon et al., 2019), and explore multiple benefits and increase the impact of green-blue-grey infrastructures (Alves et al., 2019a; Alves et al., 2019b; Ferranti and Jaluzot, 2020).

Similarly, assessment using the 1D/2D modelling approach has also shown some significant advantages (Adeogun et al., 2015; Salvan et al., 2016; Noh et al., 2018; Martínez et al., 2018b; Yin et al., 2020). The results obtained demonstrate their potential for solving some of the biggest challenges that water/wastewater utilities are currently facing (e.g. Adeogun, 2015; Salvan et al., 2016; Noh et al., 2018; Martínez et al., 2018b; Yin et al., 2020).

In addition to the abovementioned studies, green-grey approaches for current and future urban flood mitigation have been addressed (Leng et al., 2020; Gallo et al., 2020; Leon et al., 2020; Chen et al., 2021). However, a green-grey approach assessment which includes the rainfall-runoff and infiltration process on the overland flow and its interaction with a sewer network have not been taken into consideration. Further to this, there is a lack of information on the impact of representing green infrastructure in a 2D model domain when computing flood damage and investment costs. The remaining challenge is still the performance quantification of optimal green-grey infrastructures with the mentioned considerations.

The objective of the present Chapter is to develop a multi-objective model-based assessment of green-grey infrastructure for urban flood mitigation. To achieve this, three modelling components have been developed to form the structure of the framework. The first component provides the optimal number of green infrastructures allocated in the catchment. The second component produces the optimal grey infrastructures such as pipe and storage sizing. The third component evaluates the selected optimal green-grey practices based on rainfall-runoff and infiltration computation that are included in a 2D model domain. The main contributions or novelties of the present work are that the proposed method can be significantly closer to real-world physics than traditional model-based approaches for urban flood mitigation, and as such it is likely to produce better results, and that the proposed assessment identifies flood depth maps, including rainfall-runoff and infiltration computation in a 2D model domain, more reliably than conventional approaches. The details of the proposed model-based assessment are presented below. A drainage system in a real-life case study in Dhaka City (Bangladesh) is used to demonstrate its feasibility and application procedures.

6.2 Case study

The urban catchment of Segunbagicha, Dhaka (Bangladesh) has a drainage area of 8.3 Km^2. It encloses 74 subcatchments, 88 conduits (75 circular pipes and 13 box culverts), 88 nodes (junctions), 2 pump stations and 1 outfall. The time of concentration is 20 minutes. Figure 6.1depicts the study area with the land use type (Figure 6.1a). The Digital Terrain Model (DTM) has 10 m resolution. The 1D sewer model was previously calibrated in the work described in Ahmed (2008).

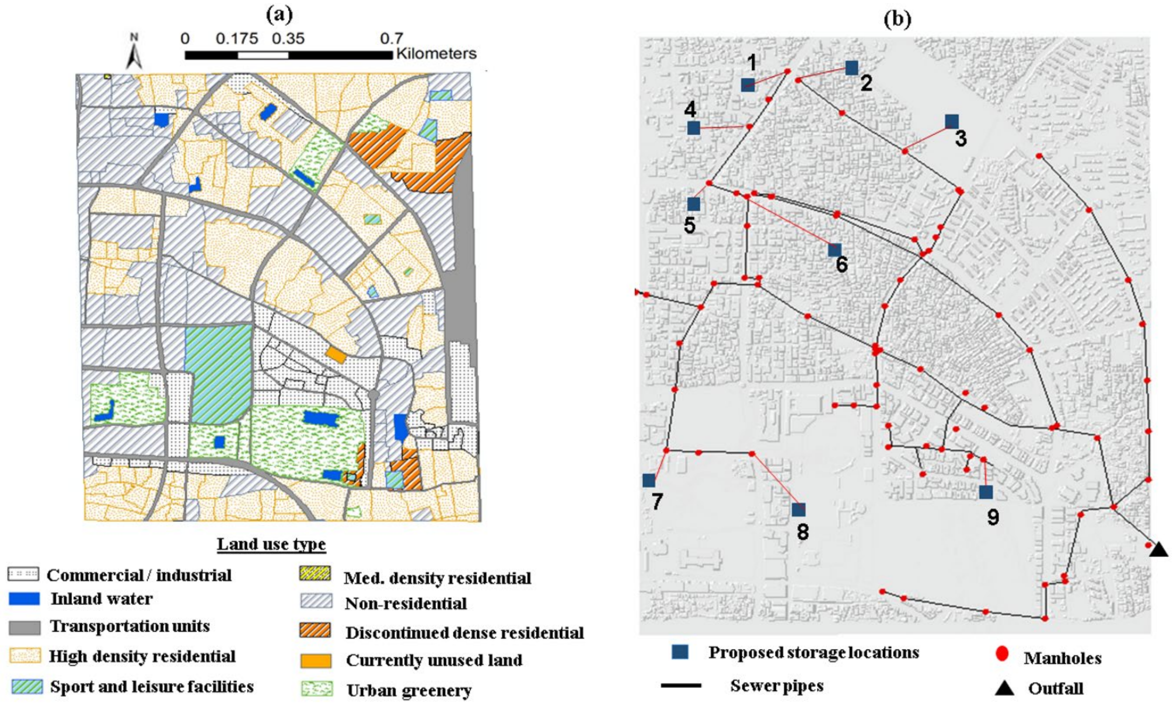

Figure 6.1. Segunbagicha urban catchment, Dhaka (a) land use type. (b) drainage layout

Nine possible sites for storage tanks were selected based on the availability of space and the performance of the system. Their location is illustrated also in Figure 6.1b. Storage tanks are defined through an elevation-storage curve with a maximum depth of 5 m. The depth is governed by a weir and a control rule.

6.3 Methodology

The present work aims to develop a multi-objective model-based evaluation of green-grey infrastructures for urban flood mitigation. To this purpose, three modelling components have been developed to form the structure of the assessment. Figure 6.2 presents the proposed framework.

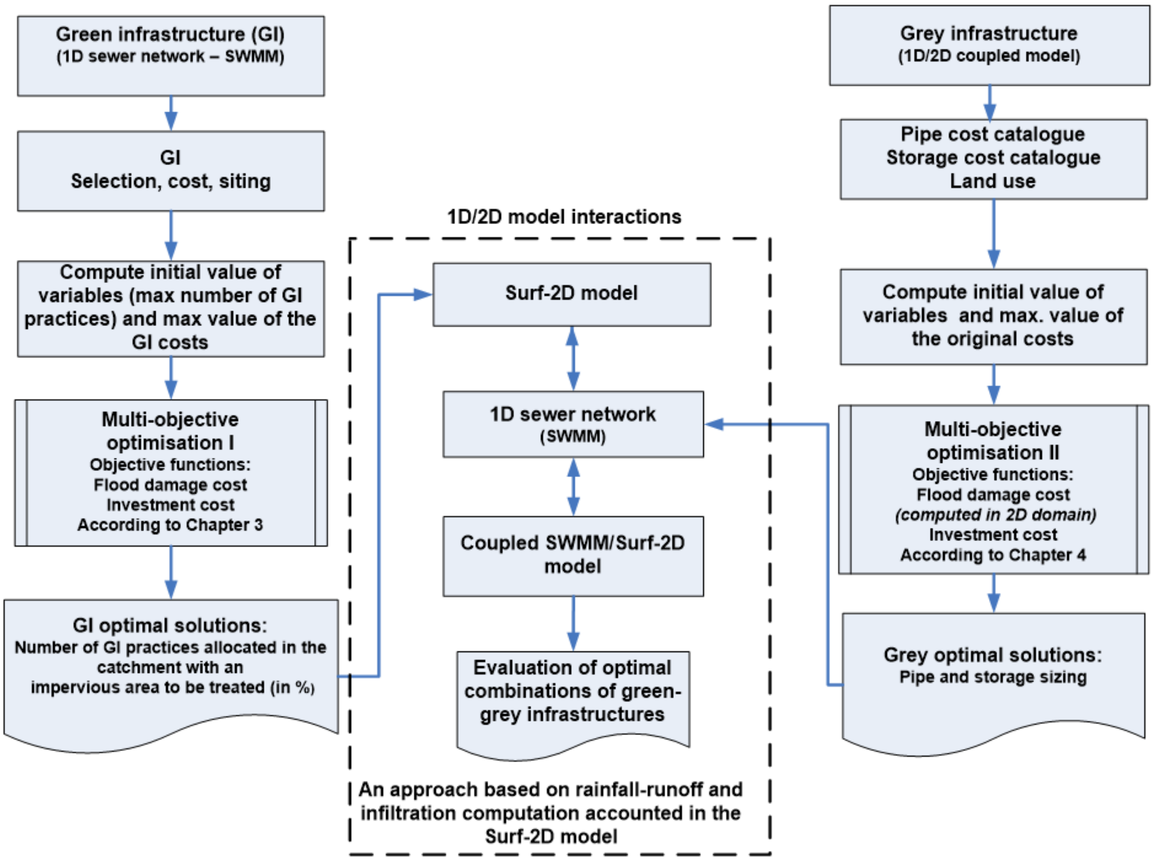

Figure 6.2. Structure of the proposed multi-objective model-based assessment

The first component is a 1D sewer model with implemented green infrastructure (GI). This takes advantage of using the LID control module of the Storm Water Management Model SWMM 5.1 (Rossman, 2017). The sewer model was then coupled with an optimisation algorithm (NSGA-II) with the purpose of obtaining the optimal number of GI practices allocated in the catchment using the percentage of impervious area. Different GI practices were evaluated for their minimisation of flood damage and investment costs.

The second component simulates the hydrological rainfall-runoff process and routing of flows in drainage pipes using a 1D sewer network built in SWMM 5.0. The 1D/2D model determines the interacting discharge between the manholes and the overland flow. Grey infrastructure such as the sizing of pipes and storage was implemented within the sewer model. This second component was set as a multi-objective optimisation problem by coupling the 1D/2D model with the optimization algorithm NSGA-II, and also has the aim of minimising flood damage and investment costs.

The third component evaluates the optimal combination of green-grey infrastructures obtained from components 1 and 2. It simulates the rainfall-runoff and infiltration process on the overland flow along with the interaction with the sewer system. The purpose of this component is to reproduce the real-world physics (i.e. the rainfall-runoff and infiltration computation accounted in the 2D model) The details of this component are presented in Section 6.3.3. A detailed description of each of these three components is given below.

6.3.1 Green infrastructure

The first component follows the work presented in Martínez et al. (2018b). Its objective is to allocate the optimal number of environmental practices (green infrastructure - GI) to each subcatchment, taking into account the percentage of impervious area. The optimal number of GI practices was addressed as a multi-objective optimization problem by minimizing the cost of its placement (i.e. the number of GI practices to be installed by their cost) and flood damage cost (i.e. flooding of each node).

This component is a 1D sewer network built in SWMM 5.1 software (Rossman, 2017) and coupled with the optimization algorithm NSGA-II (see Chapter 3 for details). The subcatchment parameters of imperviousness percentage, width, and slope were modified according to the original calibrated 1D sewer network of Dhaka (Ahmed, 2008). The GI were implemented in its LID control module. The result of this process is a Pareto front with non-dominated solutions (i.e., the optimal number of GI practices that minimize flood damage and investment costs).

Appropriate sites for GI placement were acquired from the best management practices tool–siting tool (Shoemaker et al., 2013). This tool identifies suitable locations for different GI. It finds potential fitting areas considering the urban land use, classification of soil, streams, impervious regions, and land ownership to allocate the proposed GI. A free space was then computed by deducing the land use area coverage from the total area of each subcatchment. The GI placement was accomplished by identifying the available space for GI implementation in each subcatchment. The ArcGIS tool was used to define the available area. Subcatchment parameters such as width and imperviousness were modified by using Equations 6.1 and 6.2 (Rossman, 2017):

$$\mathrm{Imp_{new}} = \frac{\mathrm{Imp\ \% * subcatchment\ area\ after\ GI}}{\mathrm{Total\ subcatchment\ area}} \tag{6.1}$$

$$\mathrm{W_{new}} = \frac{\mathrm{subcatchment\ area\ after\ GI}}{\mathrm{Total\ subcatchment\ area}} * \mathrm{W} \tag{6.2}$$

Where *Impnew* is the new impervious percentage and *Wnew* is the new width of the subcatchments after GI placement. The subcatchment area after GI was defined by the difference of the total area of each subcatchment, minus the total area taken by the GI

(Rossman, 2017). As a result, 130 GI types were located along the catchment area and used as decision variables within the optimization method. Table 6.1 presents a summary table with the detailed information of the selected GI for the case study.

Table 6.1. Summary table of the implemented GI for Segunbagicha urban catchment - Dhaka (Bangladesh)

Sub Catchment	GI type	Area (Ha)	% Imper	Drainage area (Ha)	Flow (m^3/hr)	Volume (m^3)	Size depth (m)	Width (m)	Unit area (m^2)	GI # units	% Imper area treated	GI unit cost	GI total cost
0		8.12	0.80										
	BR02			2	181	4339	1.6		2712	2	34	$70,861	$141,722
	IT02			2	181	4339	1.4	7.5	3099	2	39	$67,971	$135,943
	VS01			0.5	45	1084	1.6	10	678	3	13	$52,869	$158,607
	PP01			0.054				6	200	1	1.3	$270,923	$270,923
1		8.32	0.80										
	BR02			2	181	4339	1.6		3187	2	33	$70,861	$141,722
	IT02			2	181	4339	1.4	7.5	3642	2	38	$67,791	$135,943
	VS01			0.5	45	1085	1.6	10	797	3	12	$52,869	$158,607
2		9.57	0.80										
	BR02			2	181	4323	1.6		2702	3	43	$70,861	$141,722
	IT02			2	181	4323	1.4	7.5	3088	2	33	$67,971	$135,943
	VS01			0.5	45	1081	1.6	10	675	2	7	$52,869	$158,607
3		31.58	0.80										
	BR02			2	181	4339	1.6		2712	3	13	$70,861	$212,583
	IT01			2	181	4339	1.4	7.5	3099	2	10	$67,971	$135,943
	VS01			0.5	45	1084	1.6	10	1736	2	6	$52,869	$105,738
	PP01			0.054				6	1450	1	2	$270,923	$270,923
4		9.57	0.70										
	BR02			2	157	3773	1.6		2358	2	16	$70,861	$141,722
	IT02			2	157	3773	1.4	7.5	2695	1	9	$67,971	$67,971
	VS01			0.5	39	943	1.6	10	590	1	2	$52,869	$52,869
	PP01			0.054				6	4542	1	16	$270,923	$270,923
5		13.6	0.2										
	BR02			2	45	1078	1.6		674	5	3	$70,861	$354,306
	IT02			2	45	1078	1.4	7.5	770	5	4	$67,971	$339,856
	VS01			0.5	11	2695	1.6	10	169	3	0	$52,869	$158,607
6		7.35	0.5										
	BR02			2	112	2695	1.6		1684	2	9	$70,861	$141,772
	IT02			2	112	2695	1.4	7.5	1925	2	10	$67,971	$135,943
	VS01			0.5	28	674	1.6	10	421	1	1	$52,869	$52,869

Table 6.1. Cont.

Sub Catchment	GI type	Area (Ha)	% Imper	Drainage area (Ha)	Flow (m^3/s)	Volume (m^3)	Size depth (m)	Width (m)	Unit area (m^2)	GI # units	% Imper area treated	GI unit cost	GI total cost
7		9.02	0.9										
	BR02			2	202	4851	1.6		3032	1	34	$70,861	$70,861
	IT02			2	202	4851	1.4	7.5	3465	1	38	$67,971	$67,971
	VS01			0.5	51	1212	1.6	10	758	1	8	$52,869	$52,869
8		23.59	0.70										
	BR02			2	157	3773	1.6		2358	2	7	$70,861	$141,772
	IT02			2	157	3773	1.4	7.5	2695	2	8	$67,971	$135,943
	VS01			0.5	39	943	1.6	10	589	2	2	$52,869	$105,738
	PP01							6	11353	1	16	$270,923	$270,923
9		8.68	0.95										
	BR01			2	213	5121	1.6		3200	1	74	$70,861	$70,861
10		28.72	0.90										
	BR02			2	202	4851	1.6		3032	3	32	$70,861	$212,583
	IT02			2	202	4851	1.4	7.5	3465	3	36	$67,971	$203,914
	VS01			0.5	50	1212	1.6	10	758	2	5	$52,869	$105,738
	PP01							6	2174	1	8	$270,923	$270,923
11		34.37	0.80										
	BR02			2	180	4312	1.6		2695	2	8	$70,861	$141,722
	IT02			2	180	4312	1.4	7.5	3080	2	9	$67,971	$135,943
	VS01			0.5	45	1078	1.6	10	674	2	2	$52,869	$105,738
	PP01							6	2174	1	8	$270,923	$270,923
12		12.79	0.81										
	BR02			2	181	4339	1.6		2712	2	22	$70,861	$141,722
	IT02			2	181	4339	1.4	7.5	3099	2	25	$67,971	$135,943
	VS01			0.5	45	1085	1.6	10	678	2	5	$52,869	$105,738
	PP01							6	2174	1	26	$270,923	$270,923
13		8.22	0.81										
	BR02			2	181	4339	1.6		2712	2	34	$70,861	$141,722
	IT02			2	181	4339	1.4	7.5	3099	2	39	$67,971	$135,943
	VS01			0.5	45	1085	1.6	10	678	1	4	$52,869	$52,869
14		9.96	0.70										
	BR02			2	157	3773	1.6		2358	2	16	$70,861	$141,722
	IT02			2	157	3773	1.4	7.5	2695	2	18	$67,971	$135,943
	VS01			0.5	39	943	1.6	10	589	1	2	$52,869	$52,869

Table 6.1. Cont.

Sub Catchment	GI Type	Area (Ha)	% Imper	Drainage area (Ha)	Flow (m^3/hr)	Volume (m^3)	Size depth (m)	Width (m)	Unit area (m^2)	GI# units	% Imper area treated	GI unit cost	GI total cost
15		4.56	0.70										
	BR02			2	157	3773	1.6		2358	2	34	$70,861	$141,722
	IT02			2	157	3773	1.4	7.5	2695	2	39	$67,971	$135,943
	VS01			0.5	39	943	1.6	10	589	1	4	$52,869	$52,869
16		36.41	0.95										
	BR02			2	213	5120	1.6		3200	2	35	$70,861	$141,722
	IT02			2	213	5120	1.4	7.5	3657	2	40	$67,971	$135,943
	VS01			0.5	53	1280	1.6	10	800	1	4	$52,869	$52,869
17		32.89	0.95										
	BR02			2	213	5120	1.6		3200	2	39	$70,861	$141,722
	IT02			2	213	5120	1.4	7.5	3657	2	44	$67,971	$135,943
	VS01			0.5	53	1280	1.6	10	800	1	5	$52,869	$52,869
18		21.41	0.95										
	BR02			2	213	5120	1.6		3200	2	60	$70,861	$141,722
	IT02			2	213	5120	1.4	7.5	3657	1	34	$67,971	$ 67,971
19		10.21	0.90										
	BR02			2	202	4851	1.6		3032	1	30	$70,861	$70,861
20		6.02	0.90										
	BR02			2	202	4851	1.6		3032	1	50	$70,861	$70,861
	VS01			0.5	51	1212	1.6	10	758	1	13	$52,869	$52,869
21		7.67	0.95										
	BR02			2	213	5120	1.6		3200	1	83	$70,861	$70,861
22		14.59	0.9										
	BR02			2	202	4851	1.6		3032	2	42	$70,861	$141,722
	IT02			2	202	4851	1.4	7.5	3465	1	24	$67,971	$ 67,971
	VS01			0.5	50	1212	1.6	10	758	1	5	$52,869	$52,869
23		26.56	0.9										
	BR02			2	202	4851	1.6		3032	2	23	$70,861	$141,722
	IT02			2	202	4851	1.4	7.5	3465	2	26	$67,971	$135,943
	VS01			0.5	50	1212	1.6	10	758	1	3	$52,869	$52,869
24		15.44	0.95										
	BR02			2	213	5121	1.6		3200	1	41	$70,861	$70,861
	IT02			2	213	5121	1.4	7.5	3657	1	47	$67,971	$ 67,971

Total: $. 9,918,150

Flood damage cost function for this component has been computed based on the flooding of each particular node, as follows:

$$FD_{(v)} = (\sum_{i=1}^{n} \beta * Exp(\frac{Floodvolume}{1000}) - 1) = (\sum_{i=1}^{n} \beta * Exp(\frac{S_i}{1000}) - 1) \quad (6.3)$$

Where $FD_{(v)}$ is the flood damage with the function of volume, S_i is the flood volume at each node (m^3), N is the number of nodes analysed in the network, β is a penalty factor (100000 in this case), this value will depend on the value of property.

The investment cost of each GI is connected to the total number of GI units that were implemented in each subcatchment multiplied by their implementation cost. The implementation cost is calculated from a catalogue that contains unit costs for different GI. The number of GI results from the following Equation 6.6:

$$f_2(x_i) = \frac{\sum_{j=1}^{n}(GI.cost_j \cdot GI.number_j)}{cost_{max}} \quad (6.4)$$

Where $f2\ (x_i)$ is the fitness function 2 of solution i, $GI.cost_j$ is the cost (US \$/m^2) of GI type j. $GI.number_j$ is the number of GI type j and $cost_{max}$ is the maximum implementation cost.

6.3.2 Grey infrastructure

The second component follows the work presented in Martínez et al. (2018a). The grey infrastructure evaluated in this research combines the sizing of both pipes and storage tanks as a multi-objective problem. This minimizes investment and flood damage costs. Damage is computed based on the maximum flood depth at the overland surface.

This component uses a 1D sewer network built in SWMM software and then coupled with a non-inertia 2D model (see Seyoum et al.,2012). In this coupled model the entire catchment hydrology is computed in the 1D model; when the stormwater volume of the network is surpassed and the manholes are surcharged, flow runs out into the 2D model domain from manholes and is then routed. Non-dominated solutions (sizing of pipes and storage tanks) visualized through a Pareto front are obtained from this step (flood damage vs. investment costs) as the 1D/2D model has also been coupled with the optimization algorithm NSGA-II (see Chapter 4 for details). Equation 6.5 presents the investment cost function for pipes sizing as a function of pipe length:

$$RCost = \sum_{i=1}^{n} (C(P)_i) * L_i \quad (6.5)$$

Where *RCost* is the pipe rehabilitation cost (US \$), n is the number of pipes to be upgraded, i is the index of pipes i^{th}, $C(P)_i$ is the cost of the pipe ith (US \$/m) based on the catalogue of commercially available sizes and L_i is the length of the pipe i^{th} (m). For storage tanks, the costs depend on the cost/area of storage from the catalogue. For the combination of pipes and storage sizing, the investment cost was computed including the construction cost of the storage plus a summation of each pipe length multiplied by the cost of that particular pipe based on its diameter.

In this component, the flood damage cost estimation was carried out based on the maximum flood depth at the overland surface. Nine depth-damage curves were built from five water depth ranges (0.3m, 0.61m, 0.91m, 1.22m, 1.52m) based on the average damage/loss dataset developed for Dhaka city by Islam (2005). Land use categories (e.g., residential, commercial, governmental, and educational sectors) were also built by fitting a linear equation (Dutta et al., 2001). The nine land-use damage curves and five water-depth ranges led to 45 damage-cost functions to estimate tangible direct damages. Damage costs in each grid cell of the 2D model were computed using Equation 6.6 given by:

$$DamageCost[i,j] = (\propto + \beta) * MaxWdpth[i,j] \tag{6.6}$$

Where $MaxWdpth$ $[i, j]$ is the maximum water depth at the cells [i, j], α is the slope and β is the intercept of each linear regression.

6.3.3 Multi-objective model-based assessment

As presented in Figure 6.2, the third component of the model-based assessment includes the computation of rainfall-runoff and infiltration losses on the overland flow. In this study, the overland flow simulation builds on the work started in Seyoum et al. (2012). The 2D model solves the 2D Saint-Venant shallow water equations. The continuity equation is given as follows:

$$\frac{\partial h}{\partial t} + \frac{\partial (uh)}{\partial x} + \frac{\partial (vh)}{\partial y} = 0 \tag{6.7}$$

Where h is the water depth, u and v are the velocities in the directions of the x and y directions. The momentum equations without considering eddy losses, Coriolis force, variations in atmospheric pressure, the wind shear effect, or lateral inflow are given in Equations 6.8 and 6.9:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\left[\frac{\partial h}{\partial x} + f\frac{u\sqrt{u^2+v^2}}{4gh}\right] = 0 \tag{6.8}$$

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}+g\left[\frac{\partial h}{\partial y}+f\frac{v\sqrt{u^2+v^2}}{4gh}\right]=0 \tag{6.9}$$

Where *g* is the acceleration due to gravity and the coefficient *f* is represented in the friction terms of Chézy roughness (taking an average value of 45). The conservation of mass and momentum equations (the Saint Venant equations) given in discretized form were written as a computational engine in C++ language applying the alternating direction implicit scheme (ADI algorithm).

The main features of the Surf-2D model include a two-point forward spatial and temporal difference scheme adopted on the basis of a uniform time step $\Delta t = tn+1 - tn$, in which *n* is the time step counter. For the wetting and drying procedure, the water depth of a grid cell is calculated as the average depth over the whole cell (Yu and Lane 2006). When the cell first receives water, the wetting front edge usually lies within the cell. In most cases, only part of the cell will be wetted at that time step. When the flow volume leaving a cell is more than that entering the cell, the cell dries and there is the possibility that the water depth may be reduced to zero or a negative value (Seyoum et al., 2012).

In order to avoid negative depth values, the wetting process is controlled by a wetting parameter. When the cell is wetting, the water should not be allowed to flow out of the cell until the wetting front has crossed the cell by a property called percentage wet, as given in Equation 6.10:

$$percentage\ wet = min\left(1, \frac{\sum(v\Delta t)}{\Delta x}\right) \tag{6.10}$$

Where *v* is the velocity computed from the discharge crossing the cell boundary divided by the cell width and the cell flow depth; Δx is the cell width and Δt is the current time step. Water is not allowed to flow out of the cell until the wetting parameter reaches unity. The wetting parameter is updated in each time step to describe the water travelling across a cell. The whole surface of the cell is used as an active infiltration surface, even if rainfall intensity is zero and the cell is only partially wet. In terms of the numerical scheme, the model has the ability to halve or double the time step; halving to meet the convergence criterion, and doubling after a certain number of time steps without halving.

Previous 1D/2D coupled models in which the hydrological and hydrodynamic flood processes are modelled entirely in a 2D model domain have been proposed, see for instance (Jang et al., 2018; Yin et al., 2020; Li et al., 2020). In this study, in order to obtain surface runoff coming from a rainfall hyetograph, the unit hydrograph method proposed by the US Soil Conservation Service (SCS, 2002) was implemented as a surrogate model. The unit hydrograph is convoluted with the effective rainfall hyetograph to acquire the composite flood hydrograph. It is estimated from a synthetic dimensionless hydrograph by considering the ratios of *q/qp* (flow/peak flow) on the ordinate axis and t/t_p (time/time to peak) on the abscissa. The shape of the unit hydrograph, as identified with its peak and time lag, is the quantification of the amount of runoff diffusion predominant in the catchment.

The corresponding hydrograph was used as an inflow upstream boundary condition in the 2D model domain. In this way, water flows into the model area as a sink or source in a grid with the flow having no horizontal momentum addition, or at a cell boundary so that the contribution of the momentum of the inflow is involved. The basic geometry for the model domain consists of a square grid of point values of the ground level across the urban area being modelled.

Initial losses (rainfall interception from roofs, urban trees, and depression storage) were included in the unit hydrograph at the start of the design storm. The initial loss value follows the observations presented for urban areas in Rammal and Berthier (2020). Infiltration contributes to runoff losses during and after the rainfall event, and has thus been described as follows.

An infiltration module algorithm based on the Green and Ampt method was coded into the 2D model and called hereafter Surf-2D. The Green-Ampt equations are given as:

$$f(t) = k_e\left[1 + \frac{\Psi\theta_d}{F(t)}\right] \tag{6.11}$$

$$F(t) = k_e t + \Psi\theta_d ln\left[1 + \frac{F(t)}{\Psi\theta_d}\right] \tag{6.12}$$

Where $f(t)$ is the infiltration rate (mm/h), *F(t)* is the cumulative infiltration depth (mm), *ke* is the effective saturated conductivity (mm/h), θ_d is the moisture deficit (mm/mm), *t* = time and Ψ is the suction head at the wetting front (mm). The ponded water depth (*ho*) calculated at the surface of the grid cell as described previously is considered insignificant in comparison to Ψ as it becomes surface runoff. Nevertheless, in cases when the ponded depth is not insignificant, the value of Ψ - *ho* is substituted for Ψ for infiltration computation at time t_n in Equations 6.11 and 6.12 (see Chow, 1988; Delestre et al., 2018). In this study, Equations 6.11 and 6.12 have been solved for infiltrated depth within the Surf-2D model using a Taylor-series expansion, a method proposed and validated in Stone et al. (1994).

Infiltration was calculated by taking into account the computed velocity at which water enters into the soil (infiltration rate) in the corresponding grid cell (area of the grid) per unit of time (Green and Ampt, 1911; Mein and Larson, 1973). It is treated as a discharge point sink within the same time interval. The water infiltration was assumed to be one-dimensional, and thus there is no lateral drainage**.** To avoid an infinite infiltration rate initially (when the infiltrated volume is still equal to zero), a threshold was added to obtain the infiltration rate $f = min(inf\ capacity,\ i_{max})$. Because the infiltrated volume cannot exceed the water depth (h) at the surface of the cell that is available for infiltration at time t_n, the volume was updated as shown in Equation 6.13. Finally, the water depth was updated.

$$V_{inf}^{n+1} = V_{inf}^{n} + \min(h, f * \Delta t) \tag{6.13}$$

The Surf-2D model was then coupled with SWMM 5.1 software. Originally, SWMM code is separated into functions inside a dynamic link library file which enables an easier handling and linkage to other models. The linking methodology includes three extra functions that were written into the code for exchanging information (i.e., Node ID, water levels, discharges) between the two models during every simulation time. Full details of the linking methodology can be found in Leandro and Martins (2016). As stated above, direct surface runoff resulting from a given excess rainfall hyetograph was added directly into the Surf-2D model and thus for this component SWMM computes the dynamic sewer network flow and its hydrological runoff module was not used.

Selected optimal combinations of green-grey infrastructures obtained from corresponding components 1 and 2 have been evaluated. Optimal green infrastructure was represented in the Surf-2D model. For this purpose, the optimal percentage of impervious area to be treated obtained from the model component 1 (see Section 6.3.1) was used to assign a specific hydraulic conductivity *(ke)* of the soil (i.e., *ke* value assigned for each grid cell). The *ke* values were selected from the corresponding land use type (Figure 1a) and fieldwork infiltration data from the study presented in Uddin (2014). Optimal grey infrastructure was simulated in the 1D sewer model, taking into account the optimal sizing of pipes and storage tanks obtained from the model component 2 (see Section 6.3.2). The 1D model does not initially have water to simulate the overland flow draining back to the system.

6.4 Results and Discussion

6.4.1 Green infrastructure

In order to test the initial performance of the drainage system, the 1D sewer model built in SWMM was run for a 2-year rainfall event with the purpose of analyzing the green-grey infrastructures for a small, frequent rainfall event. The simulation results without implementing any infrastructure indicate a total flood volume of 6,040 m^3. The estimated damage cost computed based on the flooding of each node were found to be \$5.2 million. The overall project investment cost using this maximum number of green infrastructures

(GI practices) was found to be $10 million dollars. This value was obtained according to the maximum number of practices of each GI and its equivalent cost.

Promising relevant location/areas for different GI practices were identified from the analysis of urban land use, soil classification, land ownership, and impervious layers. The distribution of GI was carried out by calculating the available space in each sub-catchment. The minimum percentage of available area (ha) was found to be 5% and the maximum 80%. Figure 6.3 shows the maximum number of GI practices obtained for each subcatchment.

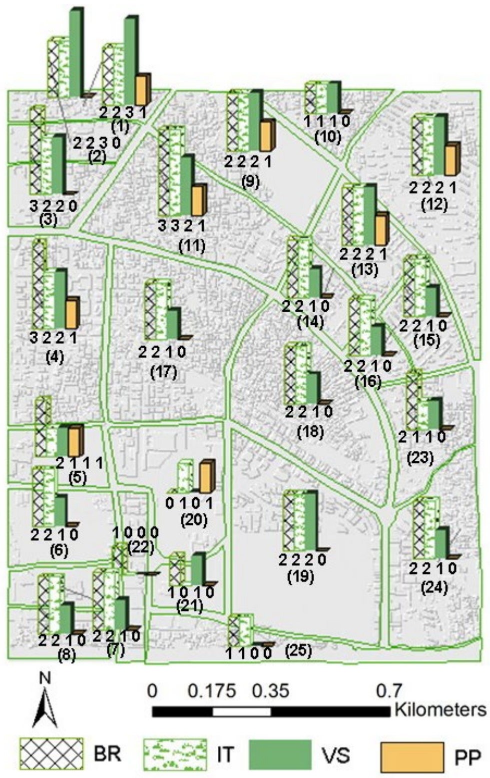

Figure 6.3. Green infrastructure for the Segunbagicha catchment in Dhaka showing the maximum possible number of practices in each subcatchment: bio-retention cells (BR). Infiltration trench (IT), vegetative swale (VS), porous pavement (PP) (subcatchment numbers in brackets)

With the criteria presented in Chapter 3, the maximum number of GI units found was 130: 47 bio-retention cells, 42 infiltration trenches, 33 vegetative swales, and 8 porous pavements. As stated in section 6.3.1 and presented in Figure 6.2, the first component of the proposed assessment searches for the optimal number of GI practices that minimizes both flood damage and investment costs. Figure 6.4a shows the non-dominated solutions which reduce flood damage and investment cost by implementing green infrastructure for a 2-year rainfall event. Figure 6.4b presents the optimal number of GI from a selected solution s1 with the aim of identifying the GI practice that best reduces flood damage. Figure 4c presents the optimal number of green infrastructure deployed in the catchment.

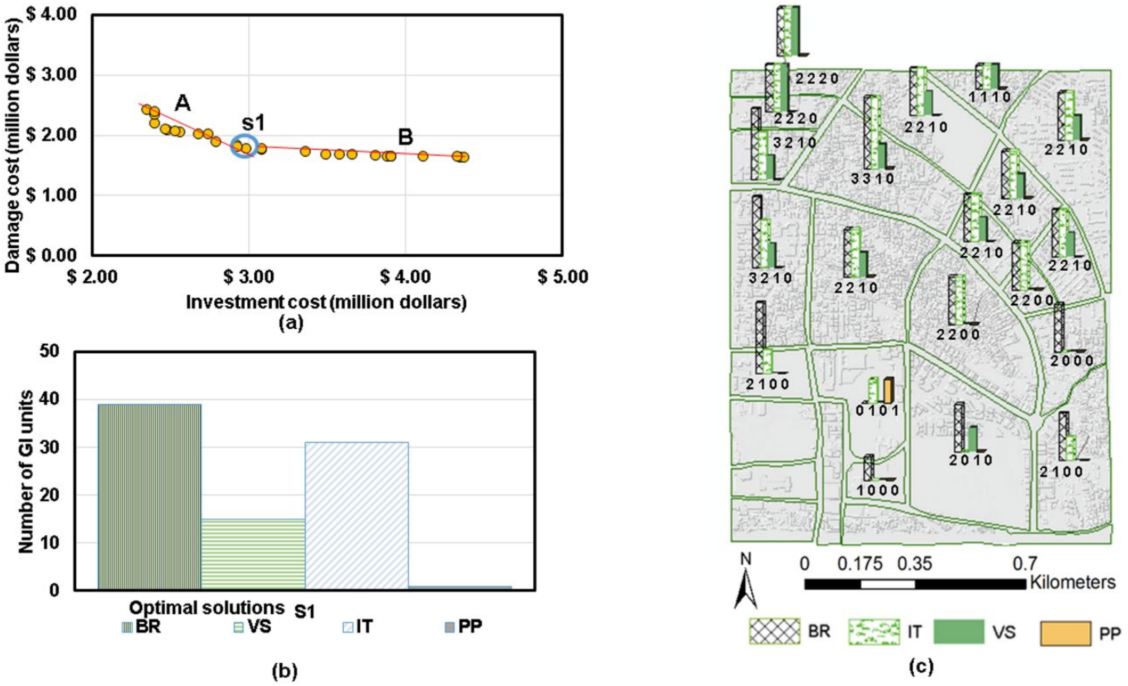

Figure 6.4. (a) The non-dominated optimal solutions implementing green infrastructure. (b) Optimal number of GI practices associated with solution s1. (c) Optimal number of green infrastructures allocated in the catchment (solution s1).

6.4.2 Grey infrastructure

A coupled 1D/2D model was run and the initial damage cost without implementing any infrastructure was computed in each grid cell of the 2D domain using the maximum water depth. The drainage system was optimized with the hydrological rainfall-runoff process and routing of flows in pipes performed in the 1D sewer network for a 2-year rainfall event. In order to obtain the optimal grey infrastructure (i.e., sizing of pipes and storage tanks), the second component of the proposed assessment (see Figure 6.2) searches for a non-dominated solution (Pareto front) that minimizes flood damage and investment costs.

The maximum value of the grey infrastructure (sizing of pipes and storage tanks) was found to be in the order of \$22.3 million. The investment cost was computed taking into account pipe length and cost based on its diameter plus the storage structure cost.

An initial damage cost of \$3.7 million was obtained. For this case, there is a 29% difference compared to the damage calculated (i.e., in component 1) from flooding of each particular node ($5.2 million). Figure 6.5a presents the non-dominated solutions by implementing grey infrastructure for a 2-year rainfall event, while Figure 6.5b depicts the optimal solution s2 of grey infrastructure.

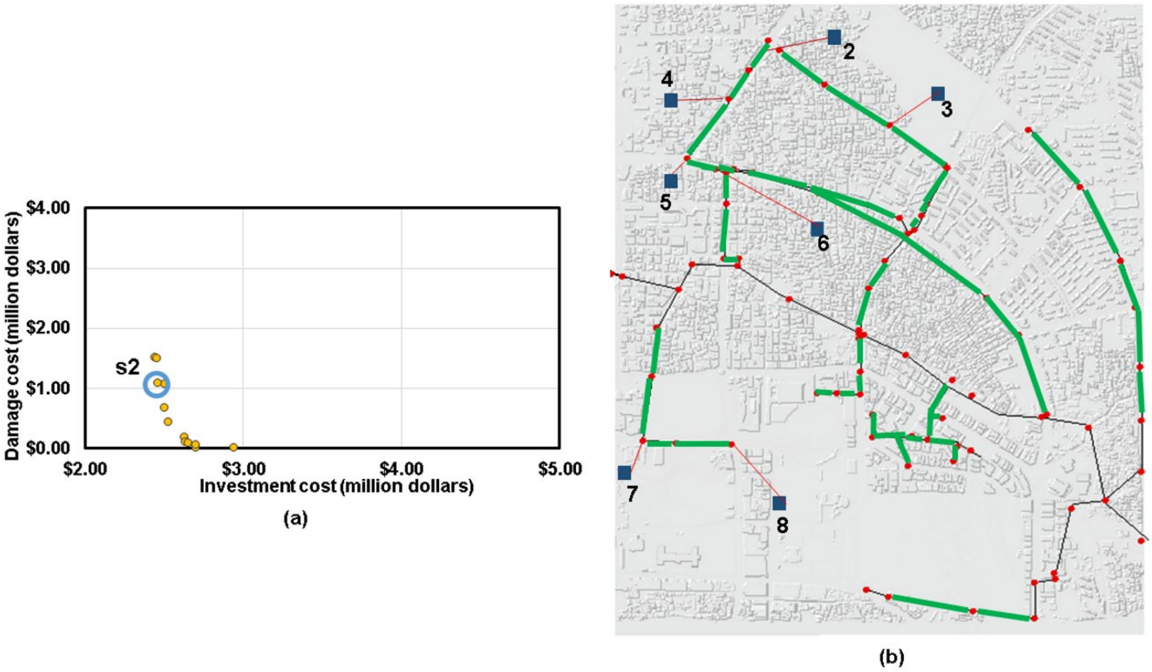

Figure 6.5. (a) The non-dominated optimal solutions implementing grey infrastructure. (b) Optimal solution (s2) of grey infrastructure, pipes to be replaced (green lines) and number and location of storage tanks (dark blue squares).

6.4.3 Combined Green-Grey Infrastructures

Using the third component presented in Section 6.3.3, which is based on the rainfall-runoff and infiltration computation in the Surf-2D, optimal solutions s1 and s2 were combined and further evaluated. The results of the surrogate model which produces surface runoff as a boundary condition into the Surf-2D model using the unit hydrograph method, were compared to the validated nonlinear reservoir routing method coded in SWMM software. The rainfall intensity for this case study is 70 mm/h of 1-hour duration corresponding to a return period of 2 years.

According to Rammal and Berthier (2020), a value of 0.65 mm was assumed as initial losses (rainfall interception and depression storage) for both methods. Infiltration losses were computed with the nonlinear reservoir method assigning clay and sandy soil types (present in the area) in the corresponding subcatchments. Average Green-Ampt parameter values were assigned as trial and error, and listed as follows: suction head (Ψ = 50 mm), effective saturated conductivity (k_e = 0.65 mm/h), and the moisture deficit (θ_d = 0.38) for a clay soil; and Ψ = 49.5 mm, k_e = 64.3 mm/h, θ_d = 0.41 for a sandy soil. Figure 6.6

shows the comparison between the unit hydrograph and the nonlinear reservoir methods for the case study. The hydrograph obtained was used as an inflow boundary condition in the Surf-2D model.

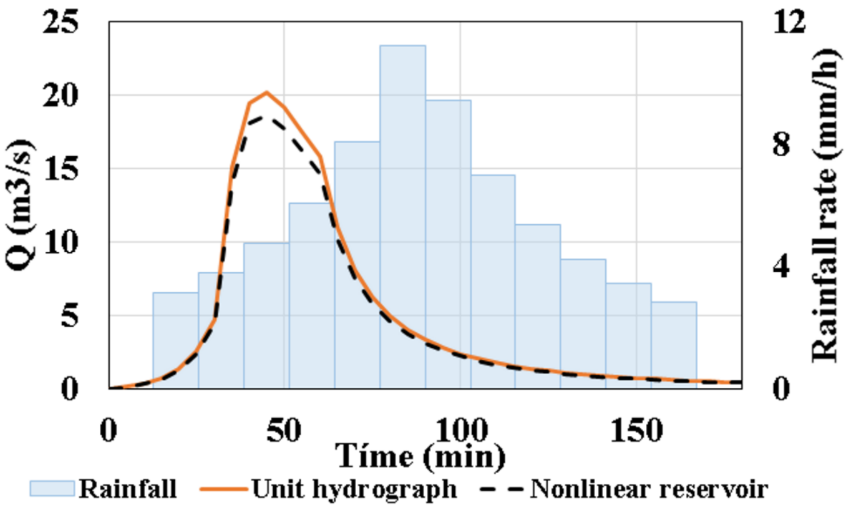

Figure 6.6. Comparison between the unit hydrograph and the nonlinear reservoir methods for the case study

In order to assess the performance of the Green-Ampt algorithm 2D, a sensitivity analysis was carried out to evaluate its outcome as the effective hydraulic conductivity parameter *(ke)* in the equation, which is necessary to obtain good estimates of infiltration rates and water depth. To this purpose, the infiltration rates measured in the field of the case study presented in Uddin (2014) were used for comparison purposes. According to this work, two types of soils (clay and sandy) cover the majority of the soil types in the area. The Green-Ampt parameter values presented in Table 6.2 were applied following the recommended values listed in Rawls et al. (1983) and Chow (1988).

Table 6.2. Green-Ampt parameters values used

Soil type	k_e (mm/h)	Ψ (mm)	θ_d(mm/mm)
Clay	0.3 – 1.0	50	0.38
Sandy	10.9 – 117.8	49.5	0.41

Figure 6.7a presents a scatter plot with the simulated infiltration rate in mm/h for a clay soil compared to the field records presented in Uddin (2014). Similarly, Figure 6.7b shows the simulated infiltration rate in mm/h for a sandy soil compared to the measured

infiltration rates. Figure 6.7c shows a flood depth map of the case study without implementing any infrastructure. Damage and investment costs were also computed within this approach and compared with those obtained in Sections 6.4.1 and 6.4.2. In the absence of field records, the coupled SWMM/Surf-2D model results in terms of flood depth were found to be similar to those obtained in previous studies in Seyoum (2013) and Vojinovic et al., (2014).

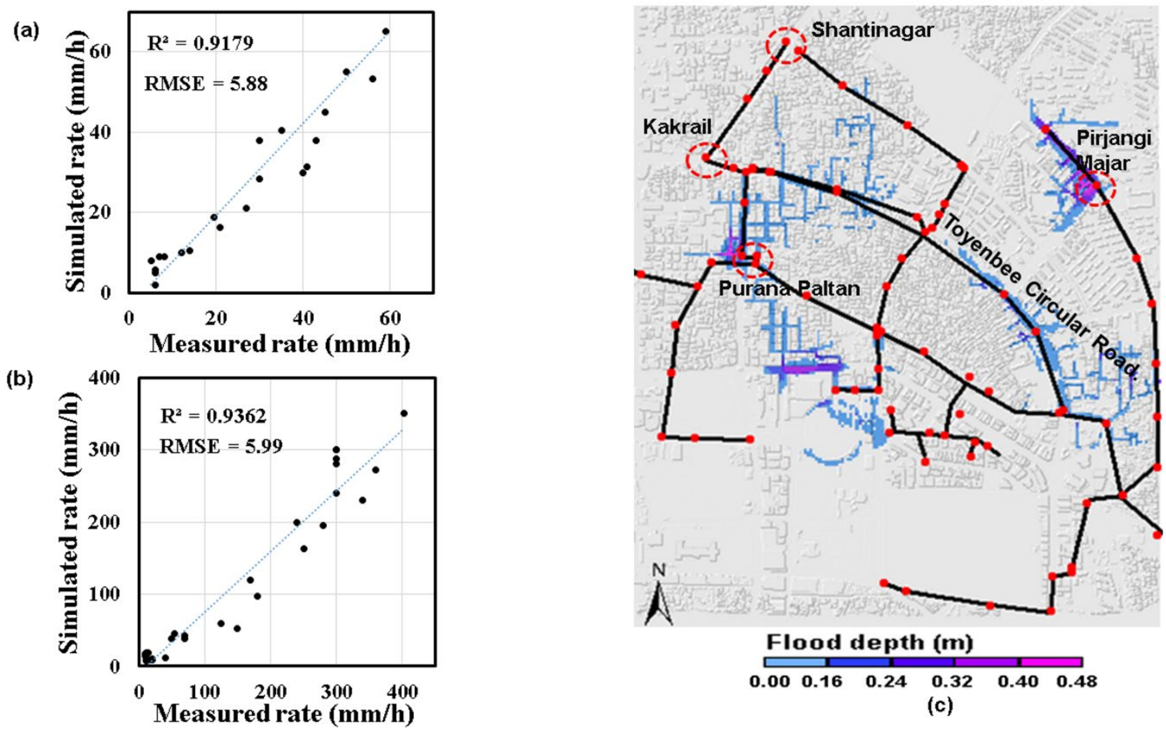

Figure 6.7. (a) Simulated infiltration rates vs. measured infiltration rates for a clay soil; (b) simulated infiltration rates vs. measured infiltration rates for a sandy soil; and (c) flood depth map without implementing infrastructure.

Table 6.3 presents a summary of the initial damage cost without implementing solutions and the related green-grey infrastructure performance.

Table 6.3. Summary of the green-grey infrastructures performance.

Modelling approach	**Initial damage cost (max.) without infrastructure ($ million dollars)**	**Infrastructure type**	**Maximum investment cost of infrastructure ($ million dollars)**	**Selected optimal solution**	**Damage cost ($ million dollars)**	**Investment cost ($ million dollars)**
1D	5.2	Green	10	s1	1.8	3.0
1D/2D (1)	3.7	Grey	22.3	s2	1.1	2.5
1D/2D (2)	4.0	Green	10	s1 (3)	1.4	3.0
1D/2D (2)	4.0	Green-grey	32.3	s1(3) + s2	0.97	4.4

(1) Computed with the entire catchment hydrology simulated within the 1D sewer network.
(2) Computed with rainfall-runoff and infiltration process on the overland flow and its connection with a sewer system.
(3) Green infrastructure represented in the Surf-2D model using the percentage of impervious areas to be treated.

Figure 6.8a shows a flood depth map with the model simulation of the optimal green infrastructure represented in the Surf-2D model and its connection with the sewer network. To be able to represent this, the optimal percentage of impervious areas to be treated obtained from optimal solution s1 were used to assign a specific hydraulic conductivity *(k_e)* of the soil (i.e., k_e value assigned for each grid cell). Figure 6.8b presents a flood depth map of the optimal combination of green-grey infrastructures.

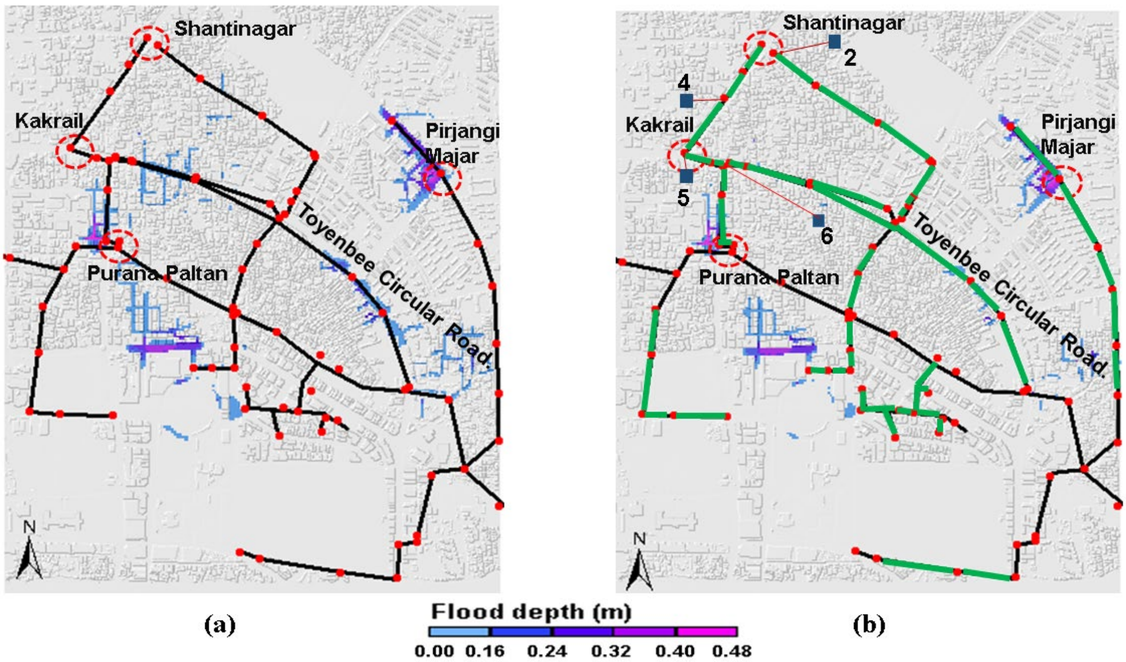

Figure 6.8. Flood depth maps with (a) the optimal green infrastructure represented in the Surf-2D model, hydraulic conductivity (ke) assigned for each grid cell, and (b) the optimal combination of green-grey infrastructures, pipes to be replaced (green line) and location of storage tanks (dark blue squares).

The results show a maximum damage cost of $2.35 million generated by implementing green infrastructure (see Figure 6.4a). With green infrastructure it is possible to further minimize damage cost to 65 % (line A) with solution s1 ($1.8 million) by investing $3 million. However, flood damage is not reduced to zero and, despite more than $3 million of investment, the damage cost reduction is not noticeable (line B). The optimal number of pieces of green infrastructures was found to be 86, consisting of 39 bio-retention cells, 31 infiltration trenches, 15 vegetative swales, and 1 porous pavement (Figure 6.4b). Green infrastructure types such as infiltration trenches and bio-retention cells present the largest number of practices. A larger number of these practices would have an effect on reducing flood damage when compared to vegetative swales and porous pavements (Figure 6.4c).

Grey infrastructure can reduce damage cost to zero for different levels of investment (Figure 6.5a). A maximum damage cost of $1.5 million is achieved compared to that obtained with GI ($ 2.35 million). It can be observed that for total protection against flood damage, an investment of between $2.7 and $3.0 million needs to be made. For comparison purposes, similarly to the GI, a non-zero damage solution was selected. Solution s2 reduces damage to 70% with an investment of $2.7 million. The analysis from the non-dominated solutions (Figure 6.5b) shows that optimal solution s2 suggests the implementation of seven storage tanks, numbers 2, 3, 4, 5, 6, 7, and 8, and the replacement of 48 pipe diameters. This would reduce flood damage to $1.1 million.

The surface runoff achieved with the surrogate model (unit hydrograph method) is in good agreement with the nonlinear reservoir method that uses SWMM (Figure 6.6). However, there is a slight difference in the peak discharge because the hydrological analysis for runoff formation performed by each method is different. Runoff values obtained with the surrogate model are approximately 6% higher compared with the nonlinear method, and NB infiltration losses in the surrogate model are not as infiltration is computed with the 2D algorithm in the Surf-2D model. The nonlinear method computes runoff with both initial and infiltration losses.

In general, the simulated infiltration rates are consistent with those measured as the coefficient of determination (R^2) found was 0.91 and a RMSE of 5.88 mm h^{-1} for a clay soil (Figure 6.7a) and R^2 = 0.93 and a RMSE = 5.99 mm h^{-1} for a sandy soil (Figure 6.7b). Simulation results show the considerable hydraulic surcharge in the system that leads to flooding (Figure 6.7c). The impact of rainfall runoff and infiltration in the 2D domain is also presented. For instance, average flood depths from 0.1 m to 0.23 m were found in Purana Paltan and from 0.17 m to 0.28 m in Pirjangi Majar. Similarly, average flood depths were observed from 0.1 m to 0.3 m in Toyenbee Circular Road.

The estimated maximum damage costs (without infrastructure) including rainfall-runoff and infiltration computation calculated in the Surf-2D were found to be $4.0 million (see Table 6.2). Results for this study also indicate that there could be an overestimation of approximately 23% ($5.2 million) if computing damage cost taking into account flooding in each particular node of the sewer network (the 1D approach). Similarly, there could be an underestimation of approximately 7.5% ($3.7 million) if damage cost is calculated with the entire catchment hydrology simulated within the 1D sewer network (the 1D/2D approach).

A reduction in flood depths and flood extent was observed (Figure 6.8a). The inclusion of the infiltration process in the Surf-2D model reduced the flood depth values. The impact of representing green infrastructure in the 2D domain was noticed in the Kakrail, Pirjangi Majar, and Purana Paltan regions as well as in Toyenbee Circular Road. However, a flood depth reduction to zero was not obtained. This is due to the optimal percentage of the area treated available in the entire catchment for each GI practice being less than 8% (i.e., GI practice represented in 2D domain as percentage of area treated). This means that for approximately 400 hectares (ha), the catchment has only 28 ha that can be treated with green infrastructure.

With the optimal green infrastructure represented in the 2D domain (solution s1), the calculated damage cost including rainfall-runoff, the infiltration computation included in the Surf-2D, and its connection with the sewer network, were found to be $1.4 million (see the summary in Table 6.2). This value is 22% less than the damage cost value originally obtained from the selected solution s1 ($1.8 million) shown in Figure 6.4. The investment cost computed for this solution was $3 million (see the summary in Table 6.2).

Green measures from solution s1 were represented in the 2D domain as previously explained while grey measures (solution s2) were implemented in the 1D sewer model, as explained in Section 6.3.2. In general, a reduction in flood depths and flood extent is observed (Figure 6.8b). The impact of the combined green-grey infrastructures is also shown. In the Kakrail, Pirjangi Majar, and Purana Paltan regions, a flood-depth reduction to zero was achieved in some places. This impact can be due to the assigned hydraulic conductivity *(k_e)* of the soil for each grid cell in these regions according to the values obtained for clay and sandy soils in the eastern part of Dhaka City (Uddin, 2014).

With the implementation of green infrastructure in the catchment, and it being represented in a 2D domain, it is no longer necessary to site storage tanks numbers 3, 7, and 8 (Figure 6.8b), despite the initial suggestion of optimal solution s2 (Figure 6.5b). This is because, for this component, the 1D sewer model does not initially have water to simulate the overland flow draining back to the system so these tanks are not filled with stormwater throughout the simulation.

With the optimal combinations of green-grey infrastructures (solutions s1 + s2), the calculated damage cost was found to be $0.97 million for a level of investment equal to $4.4 million (shown also in Table 6.2). These results show a 75% reduction in damage cost compared to the reduction obtained from implementing only green infrastructure (65%). However, although $4.4 million would need to be invested to address the damage of $4.0 million, the considerable multiple additional benefits that green infrastructures offer, such as water quality enhancement (due to runoff filtration and groundwater recharge), recreation, enhanced liveability, and direct traffic, among others, should also be taken into account so that this combined solution is, nevertheless, selected.

The abovementioned results show that for some cases the differences between flood extent, flood depths, and damage cost estimation from different approaches can be significant. Hence, this model-based assessment based on rainfall-runoff and infiltration computation calculated in the proposed Surf-2D model and its connection with the sewer network is closer to real-world physics, and as such it is likely to produce more accurate results.

6.5 Conclusions

This Chapter describes a novel model-based framework to evaluate optimal combinations of green-grey infrastructures for urban flood reduction. The assessment includes the performance of these solutions when dealing with a minimization of investment costs and direct flood damage. Three modelling components have been developed to form the structure of the framework. The first component provides the optimal number of pieces of green infrastructure (GI) allocated in the catchment. The second component provides optimal grey infrastructure such as pipe and storage tank sizing. The third component evaluates the selected optimal combinations of green-grey infrastructures based on rainfall-runoff and infiltration computation included in the proposed Surf-2D model. The potential of this model-based assessment has been demonstrated in the real-life case study of Dhaka City (Bangladesh), where different green and grey infrastructures were evaluated in relation to investment and flood damage costs.

The results obtained demonstrate in quantitative terms how the performance analysis of green-grey infrastructure for flood mitigation can be improved substantially through this proposed model-based assessment. When including rainfall-runoff and infiltration processes within a 2D model domain, along with its connection with a sewer system, the damage cost results differ from the other approaches presented. In the case analysed here, there could be an overestimation of this cost (approximately 23%) if green infrastructure is fully represented in a 1D modelling approach. Similarly, there could be an underestimation even if the overland flow is taken into consideration but the catchment hydrology is entirely computed in a 1D domain (approximately 8%). Thus, the direct impact of rainfall-runoff and infiltration enable real-world physics to be reproduced when identifying the best green-grey solutions to improve urban flood risk management.

A combination of green-grey solutions has been shown to be the best course to follow. This combination shows a better damage cost reduction compared to the value obtained from only implementing green infrastructure (GI). GI practices were represented in this case using a specific hydraulic conductivity *(k_e)* of the soil and assigned to each grid cell depending on its land use type. Even though it is beyond the scope of this study, multiple benefits of green and grey solutions should be assessed, especially where there are space limitations, as is the case here. Similarly, uncertainty associated with the optimal solution results should also be addressed.

7

REFLECTIONS AND OUTLOOK

7.1 Introduction

This chapter concludes the findings of the previous chapters and discusses them in relation to the research questions presented in Chapter 1. The outlook and reflections of the research are presented in the next sections, following the same order in which the contributions were developed and applied.

To provide answers to the research questions presented in Chapter 1, the outcomes have been grouped into the following sections.

7.2 Outcomes

RQ1: ¿How can hydraulic models and optimisation techniques be coupled for the purpose of assessing the effectiveness of green and grey infrastructures for flooding and pollution?

7.2.1 Performance of green infrastructure to reduce runoff and pollution

In order to answer this question and according to the literature, available methodologies are more focused on the optimal coverage area of Green Infrastructure (GI) instead of a GI-type preference. In Chapter 3, an optimisation procedure was proposed and tested with the aim of obtaining the optimal number of GI units distributed within the catchment for runoff reduction (*The optimal configuration of GI*). This first optimisation procedure consisted of adjusting the number of GI units, taking into account their location within the catchment area. For this purpose, an optimiser algorithm was used with the goal of finding a representative set of optimal Pareto solutions and to quantify the trade-offs between pollution load, peak runoff, flood volume, and investment cost. Two interfacing routines to join an optimiser with a 1D sewer model (*Quantity and Quality*) were developed.

The GI were implemented in the LID control module of SWMM software. The input file of the 1D sewer model specified how a particular GI was deployed. The data entry fields included the GI type and its number of units (*i.e. decision variables*) for each subcatchment. The first routine was coded to run the sewer model, set the initial value of variables and compute the normalized values of the objective functions. The maximum value of GI investment cost was computed from a catalogue file that contains the GI unit cost. The optimiser generated a file with the lower and upper range values of the decision variables.

The second routine was coded to run the 1D model and update its input file. This routine uses the generated file with the decision variables range to modify the number of GI units by overwriting its value in the input file for each iteration. With this procedure, a new objective function value was obtained. This framework has proved that coupled models that search for optimal GI units can achieve reduced runoff, better runoff quality and less investment cost. The application of multi-objective optimisation process for GI configuration may become a good choice in terms of reducing investment cost without compromising the efficiency of the drainage system.

This research demonstrates that there is an advantage in having an optimal number of GIs as the GI types mainly reflect the impact on the reduction of environmental (*pollution, peak runoff and flood volume*) and economic (*investment cost*) objectives. This suggests that if the type of GI measure and its number of units are taken into account within the coupled concepts (*numerical and multi-objective based models)*, it is possible to achieve optimal GI solutions to reduce the proposed reduction objectives with lower investment cost.

The potential of this framework applied to the Meléndez catchment in the city of Cali, Colombia, demonstrated how different GI solutions can be used to address different objectives and can also be suitable for the study area. GI such as bio-retention cells (*BR*), infiltration trenches (*IT*), vegetative swales (*VS*) and porous pavements (*PP*) were evaluated considering pollution load, peak runoff, and flood volume objectives at the lowest possible investment cost. Specifically, it was found that by investing an amount of $7.7 million approx. with a higher number of *BR* units (up to 83 units) within a specific configuration, a pollution load reduction for larger events can be obtained with solution s-10. The solutions also showed that an increase in the number of *VS* units (up to 76 units) with the same investment can yield a reduction in peak runoff for both smaller and larger events (s-16 and s-18). Similarly, with the same level of investment and with a larger number of *PP* units (up to 22 units), solution s-30 would help to reduce flood volume for shorter and larger events. There are currently actions that are aiming to reduce the pollution load in the Meléndez river, but its water quality is still continuing to decline. Similarly, in spite of substantial investment in flood control structures in the catchment, there is still the risk of flooding as the investments are not being executed according to their priority and their true impact in the catchment.

7.2.2 Interactions between different grey infrastructures to assess a drainage system capacity

Chapter 4 explores the interactions between different grey infrastructures to assess a drainage system´s capacity. The work combined computational tools such as a 1D/2D flood inundation model and optimisation engine in the loop to compute in a 2D domain the potential damage for different rainfall events. The approach of expected annual damage cost (EADC) was also introduced into the evaluation as the probabilistic cost caused by floods for a number of probable flood events.

The main conclusion obtained about the 1D/2D coupled model is that the linking model allows the value of potential flood damage to be predicted and contributes with suitable information to the purpose of studying the interactions between different grey infrastructures. Different grey solutions such as upgrading pipes (UP), distributed storage tanks (DS) and the combination of both (UP+DS) were evaluated in relation to flood damage and investment costs. The estimation of the flood damage cost was done based on the maximum flood depth at the overland surface. A second proposed optimisation procedure consisted of having an initial simulation of the hydraulic 1D/2D coupled models. It computes the maximum value of damage costs for different return periods of rainfall and the defined objective functions. It updates the pipe diameters or size of storage and also updates the 1D/2D model input file.

Similar to the first optimisation procedure tested in Chapter 3, in this optimisation method two routines were coded. The first routine runs the 1D sewer model to compute the initial value of the variables, objective functions and maximum value of the original costs. The second routine runs the coupled 1D/2D model, selects from the range of decision variables, changes the pipe diameters of the selected elements to be rehabilitated, resizes the selected storage, and computes the objective functions. As initially thought, the coupled model simulations were computationally expensive so parallel computing was performed. Two clusters of four laptops were set up and run using the master-slave approach and executed into a Parallel Virtual Machine environment (built on the work presented in Barreto, 2012).

The results indicate a promising potential of the proposed approach to achieve optimal solutions *(less damage and lower rehabilitation cost)* for different rainfall events. Although the proposed approach does not specifically include aspects such as social and environmental concerns, it can be concluded that this approach demonstrates its usefulness for planning of measures once they have passed evaluation of these concerns.

The concept of expected annual damage cost (EADC) was presented as the probabilistic cost caused by floods for a number of rainfall events. This tool calculates in the 1D/2D approach the accumulation of damages during a timeframe so that the coupled model was simulated for 2, 10, 20 and 50-year rainfall events simultaneously. The optimisation procedure was updated in order to compute, in addition to the damage costs, the EADC for each return period of rainfall. To assess the damage, depth-damage curves needed to be developed taking into account different water depth ranges and land use categories. The combining land use included residential, commercial, governmental, and educational sectors for an urban catchment in Dhaka (represented as a district scale with 10m resolution DTM).

In this research, the EADC tool demonstrates for the case study that there is no return period that exceeds the capacity of the drainage system for an optimal investment. This confirmed that the *UP* measure was capable of substantially minimizing flood damage, and it can ensure protection against up to 50-year design events. With the same optimal investment obtained with the *UP* measure, by implementing a *DS* measure seven out of nine storage tanks can reduce flood damage and contribute to a total protection of up to a 10-year event. It can also be concluded that the EADC Pareto sets confirm the significance of including the benefits (reduced rehabilitation cost) over the design life.

The potential of this framework applied for the Dhaka, Bangladesh case is that the results specifically showed that for larger design events (up to 50-year return period events), *UP* as a measure would be a good option as it requires an investment of $4.7 million to minimise the damage cost to $1 million (solution 8). With the same investment, if smaller events (up to 10 years) are selected for design purposes, a *DS* measure can reduce damage down to $0.7 million, indicating the importance and efficiency of the *UP* and the practicality and economy of constructing *DS* along the urban catchment area. However, with the locations available to implement storage tanks, it was not possible to reduce flooding to zero for 20 and 50-year rainfall events. This suggests that for the present case study area, the *DS* measure can become less effective beyond certain design events and the additional damage reduction would depend on their location instead of their volume.

The combined *UP+DS* measures show that solutions are able to reduce damage cost to zero for different levels of investment with less investment costs when compared to *UP* and *DS* measures implemented separately. In order to have a total protection for a 50-year event, between $3.5 and $4.0 million have to be invested for total protection against flood damage.

RQ2: ¿How can the infiltration process, overland flow and sewer system interactions be implemented within a coupled 1D/2D model?

7.2.3 Assessing the optimal combination of green-grey infrastructures

A proposed modelling setup which includes the rainfall-runoff and infiltration process on the overland flow and its interaction with a sewer network was presented. An infiltration module algorithm based on the Green and Ampt method was coded into a model referred to in this work as Surf-2D and then coupled with a 1D sewer model based on dynamic link libraries. Four test cases were implemented to validate the model.

The first case was a hypothetical catchment to assess the ability to simulate infiltration from a point source (direct runoff). The unit hydrograph (SCS method) implemented was indeed effective in the purpose of producing direct runoff in the 2D model domain. Despite having different hydrological considerations for runoff generation compared to the nonlinear reservoir method, both method results were found to be similar. The inclusion of the Green-Ampt method in the 2D domain had a direct impact on the overland flood depths. Although determining the soil properties may sometimes be difficult for the application of the method, the presented model was capable of reproducing the influence of infiltration capacity of the soil texture in the overland flow.

In order to verify the performance of the Green-Ampt method integrated into the 2D domain, free software FullSWOF_2D which was already validated was used for this purpose. The hypothetical case was used to compare the infiltration results obtained in the Surf-2D model with the results obtained from the FullSWOF_2D software. Here, water depth results fit closely with those computed in the mentioned software. The statistics of the model errors were found small across the time simulation.

The second case was a surcharging benchmark with previously published results to examine water depth predictions and flood extents. In this test, water depths were computed while the overland flow was originated from rainfall-runoff and surcharging underground pipes. The 1D sewer model results contribute to a better hydraulic performance due to their impact on the overland flood depths. In general, the presented model predicted similar results to the software packages (the EA benchmark report) compared in terms of peak water depths within a range of a few centimetres.

Two 1D/2D model interactions for representing urban floods were evaluated; these were conducted to different flood evolution results. With approach 1, where the entire catchment hydrology is computed in a 1D model, it was observed that depth and flood extent were still controlled by the excess flow from manholes. Approach 2 was based on rainfall-runoff and infiltration computation accounted with the Surf-2D model. For this approach 2, a culvert structure initially did not have water to simulate overland flow draining back to the system. The results showed that differences between the two types of interactions are significant. Differences of around 78% in terms of flood extent, 48% in the maximum flood depths and 90% in inundation volume were found.

Integrated modelling approaches are being increasingly promoted as required in order to holistically evaluate urban water systems while facilitating infiltration in urban areas.

RQ3: ¿How can the 1D/2D coupled model addressed in Q2 be used to assess the optimal combination of different infrastructure measures?

7.2.4 A multi-objective model-based framework to assess green-grey infrastructures

A multi-objective model-based framework to assess green-grey infrastructure holistically for urban flood reduction was proposed. This framework suggested three main components to form the structure of a proposed modelling framework. Combined optimal green-grey solutions were evaluated. The first component brought an optimal number of GI units distributed within the catchment. The second component evaluated the optimal grey infrastructure such as pipe diameters and area of storage tanks. The third component evaluated selected optimal green-grey solutions based on rainfall-runoff and infiltration computation accounted in a 2D model domain. The potential of this model-based framework was demonstrated in the real-life case study of Dhaka City (Bangladesh) where different green-grey infrastructure were evaluated in relation to investment and flood damage costs.

The results obtained demonstrate in quantitative terms how the performance analysis of green-grey infrastructure for flood mitigation can be improved substantially through this proposed multi-objective model-based assessment. When including rainfall-runoff and infiltration processes within a 2D model domain along with its interaction with a sewer system, damage cost results differ from the other presented approaches. In the case analysed here, there could be an overestimation of this cost (around 23%) if green infrastructure is fully represented in a 1D modelling approach. Similarly, there could be an underestimation even if the overland flow is taken into consideration but the catchment hydrology is entirely computed in a 1D domain (around 8%). Thus, the direct impact of rainfall-runoff and infiltration allowed real-world physics to be reproduced when identifying the best green-grey solutions to improve urban flood risk management.

A combination of green-grey solutions has shown a better damage cost reduction compared to the value obtained from implementing only green infrastructure (GI). GI solutions were represented in this case using a specific hydraulic conductivity *(k)* of the soil and assigned to each grid cell depending on its land use type.

7.2.5 Research limitations

This research still has certain limitations regarding the influence that data has in the application of the methods specifically in terms of its type, quality and availability. More case studies are required to be developed for further comparative analysis. The proposed methods do not specifically include topics such as social-institutional aspects and data from climate change scenarios, which should be included to expand the capabilities of the analysis. The developed algorithms and codes can be further refined and tested again. A combination of green-grey solutions has shown to be the best action to follow. Although in this proposed framework benefits such as reduced investment cost over the design life and water quality improvement were addressed, multiple benefits of green-grey solutions should be included and assessed specially where there are space limitations. Similarly, uncertainty associated with the optimal solution results should also be addressed.

7.3 Reflections

7.3.1 Modelling green-grey infrastructures

Although the developed framework for modelling green-grey infrastructures for runoff control in urban areas was tested in two real UDS with different hydrologic, hydraulic, socioeconomic and political conditions, it is important to test the methods in other case studies with different topography, climate and socio-economic development. The feasibility of the methods can also be tested with the use of current data available on the World Wide Web particularly for urban areas with limited information in terms of quantity and quality.

With respect to the GI design, the accuracy of the output data provided by the siting tool depends on the level of detail of the input data. The results of the multi-objective optimisation can be improved by using more accurate spatial data for the urban land use, land imperviousness, and land ownership layers. Also, the configuration of the soil layer on SWMM software can be improved by using more detailed information related to the soil taxonomy. Within the current framework, the configuration parameters of each GI unit cannot be optimised (i.e. surface berm, soil thickness, storage depth or underdrain flow rate). These parameters could also be taken into account during the optimisation of the initial GI system configuration.

Further analysis need to be extended by assessing the cost efficiency of each GI unit by comparing the hydraulic capacity of each GI unit with their investment cost. This assessment can help to reduce noise during the optimisation. The performance of the GI system for the Cali case study can be improved by taking into account the other urban catchments (Lili and Cañaveralejo rivers) that are contributing to the stormwater inflow of the network.

Rain barrels and rain gardens have an important role in GI configuration since these types of units are low-cost solutions that are not directly dependent on land use. Further, the implementation of this type of unit as a low-cost technology could be applied in a developing country context. A deeper exploration of the social and technical aspects related to the implementation of rain barrels and rain gardens could also help to reach new paradigms for stormwater management for the case study.

For improving stormwater treating capacity of the GI when using the LID module of SWMM software, it is valuable to go further into detail when designing the layers of the GI which has been seen to affect the treatment process. Also, to assign treatment at the conveyance of the UDS as the LID controls in SWMM model does not treat the TSS at the conveyance. The GI are effective in treating the TSS for a short period of time so it should be designed based on a return period of a small number of years of time events.

In previous investigations, the addition of green infrastructure in hydrodynamic models are mainly based on the capabilities of the LID module of SWMM software. In this dissertation, GI solutions were represented in a 2D model domain using a specific hydraulic conductivity *(k)* of the soil and assigned to each grid cell depending on its land use type. This is according to the percentage of area to be treated obtained from an optimisation procedure. The inclusion of the quantification of its uncertainty would enhance this representation as the biased categorization of the soil, lack of an updated data set and their infiltration parameters which are challenging to estimate are known. In addition, ground surface cover and soil type indices to determine surface roughness, interception and infiltration parameters could be also implemented within the proposed Surf-2D model.

In this research only flood damage was considered to calculate damage cost, so it is suggested to include other parts of flood damage cost such as indirect economic flood damage. Current methods also need to be extended by taking into consideration a preference-based multi-objective model that can reflect different stakeholder preferences for different green-grey solutions. Additional components could be added to the framework such as a detailed selection and location of the measures along with the optimal number of measures distributed within the area using other optimisation methods.

7.3.2 Visualizing the Pareto Frontier

Presenting results through Pareto fronts is not always the best way to exchange results; just by only using clear plots to assist the analysis between scientist and decision makers would be enough. However, visualization of feasible Pareto optimal solutions in the objective space is a valuable tool for communicating results. Around thirty-five years of real-life applications with real experts and decision makers have confirmed that visualization of a Pareto frontier has been of practical importance in water management (Buber *et al.*, 2019). Since the 2010s, due to the development of meta-heuristic algorithms, the approximation and visualization of the Pareto frontier has gradually become a verified tool of decision support in non-linear water management issues. New techniques such as using decision maps for visualization, applying hybrids of metaheuristic algorithms and classic gradient-based methods for approximation would help to further improve the results presented in this proposed framework.

7.3.3 The Surf-2D model

The unit hydrograph (SCS-method) used in the Surf-2D model was indeed effective in the purpose of producing direct runoff in the 2D model domain. Despite having different hydrological considerations for runoff generation compared to the nonlinear reservoir method, both methods results were found to be similar. However, it should be further validated with the possibility of having spatially distributed rainfall data mapped onto the model grid within the model.

The inclusion of an infiltration algorithm based on the Green-Ampt method makes Surf-2D particularly suitable for urban flood simulations. Moreover, Surf-2D has demostrated its ability to simulate the direct impact of rainfall-runoff and infiltration using both hypothetical and benchmark tests, allowing real-world physics to be reproduced where the flood volume and area recede gradually after the flood peak occurs. As is already well known, a coupled 1D /2D model is a time-consuming process. In this research, two clusters of four laptops were set up and run using the master-slave approach and executed into a Parallel Virtual Machine environment (from the work of Barreto, 2012). However, previous studies using cellular automata (CA) inundation models (see, for example, Guidolin *et al.*, 2016) have overcome this issue. This CA-based approach should be explored as an alternative method within this proposed framework.

As identified during the model development, apart from the usual bug correction work, several features to the current code might be included as follows:

- Pre-processing basic input data.
- Computation on a square grid at any resolution while retaining for resolutions greater than 1 at least some details of the ground topography of the basic grid, in terms of the local storage, and the surface texture and form roughness and conveyance over the ground surface.
- Identification of ponds or lakes which can be treated as single, large irregular cells.
- Treatment of coarse cells as sloping planes rather than flat cells in order to maintain accurate calculations down long slopes in the terrain. This is useful for simulations down a long road or street.
- Flow results at the basic fine grid level.
- The possibility to model channels and rivers within the grid by appropriate definition of the boundary base levels between cells.
- Inclusion of the nested refined grids within the model domain.

7.4 Future direction

Flood modelling is a crucial tool in the implementation of best practices and in developing policies for flood risk management. Flood risk models generally involve developing algorithms which are valuable to represent flooding in terms of flood depths and flood extents. It is important to understand the potential factors that make the implementation of flood modelling techniques difficult. Some general issues such as the need for quality dataset are now being undertaken by a number of geospatial data development packages. In terms of reduction in computational time, stability and model complexity, flood models which include the kinematic wave, diffusive wave, inertial wave equations, GIS-based and cellular automata (CA) have proven to be reasonable choices instead of using shallow water equations (SWEs). Similarly, as an alternative to the use of SWEs-based models, studies are now also exploring the development of what are referred to as “bespoke flood models”; these are capable of simulating flood inundation hazards without much reliance on distributed topographic and friction datasets (Nkwunonwo *et al*, 2020). However, the above mentioned hybrid models still need further enhancement to establish their potentials and usefulness. Integration of numerical schemes into reduced complexity flood models has also been proposed but not yet validated as the problem of uncertainty and model sensitivity to external datasets and test locations still dominate.

An assessment of the performance of green-grey solutions requires a more holistic judgment, a better model conceptualization and an effectively dissemination of the results. Improved software implementation would benefit from a team of hydroinformaticians and actors involved by participating in every step of the project and in this way it would be possible to minimise the intuitive bias of a single modeller. The research community is continuing its effort proposing new frameworks which include novelties in terms of uncertainty, flexibility, robustness and sensitivity analyses as a challenge to evaluate the performance of green-grey infrastructures for flood risk mitigation.

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LIST OF ACRONYMS

1D	One Dimensional
2D	Two Dimensional
BR	Bio-Retention cells
BMP	Best Management Practices
CVC	Corporación Autónoma Regional del Valle del Cauca
DAGMA	Departamento Administrativo de Gestión del Medio Ambiente
DC	Direct Costs
DEM	Digital Elevation Model,
DO	Dissolved Oxygen
DS	Distributed Storage
DWASA	Dhaka water supply and sewerage authority
DWF	Dry Weather Flow
EADC	Expected Annual Damage Cost
EMCALI	Empresas municipales de Cali
FullSWOF	Full Shallow-Water equations for Overland Flow
GI	Green Infrastructure
GVC	Governance of Valle del Cauca
IFRC	International Federation of Red Cross and Red Crescent Societies
IT	Infiltration Trenches
LID	Low Impact Development
NSGA	Non-dominated Sorting Genetic Algorithm
PP	Porous Pavement
SCS	Soil Conservation Service

SUDS	Sustainable Urban Drainage systems
SWMM	Storm Water Management Model
TEC	Total Expected Cost
TSS	Total Suspended Solids
UP	Upgrading of Pipes
USEPA	Unites States Environmental Protection Agency
UDS	Urban drainage systems
VS	Vegetative swales

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About the Author

Carlos obtained his BSc, in Civil Engineering in February 2000 from Universidad de la Salle, in Bogotá, Colombia. At the same time, from 1995 to 2000 he took music lessons and played clarinet in the Colombian youth symphony orchestra also in Bogotá. He later obtained a post-graduate certificate as specialist in Hydraulic Resources, from the Universidad Nacional de Colombia, sede Bogotá, in the year of 2003. His post-graduate study focused on morphology and fluvial dynamics of the Magdalena river, San Rafael de Chucurí and Carmelitas sector. During 2003 and 2005 he worked in public and private engineering companies as a construction engineer and consultant in water related projects.

In January 2006 he moved to London, United Kingdom to improve his english skills and meanwhile he worked at Hard Rock Café as a busser. In October 2007, he moved to The Netherlands to study his Master of Science in Hydroinformatics at UNESCO-IHE, Institute for Water Education in Delft thanks to the support of Nuffic-NFP scholarship. He obtained his MSc degree in September 2009. His MSc study focused on using a hybrid approach to improve rainfall prediction for water management.

In January 2010 he moved to Cali, Colombia to work as a researcher at Instituto Cinara, Universidad del Valle. There, he participated among other projects in the Sixth Framework Programme "SWITCH Managing water for the City of the future" and in the cooperation agreement project "Operational Flood Forecasting Warning and Response for Multi-Scale Flood risks in Developing Cities (FORESEE)" under the Univalle and IHE Delft alliance.

In June 2013, Carlos got married and started his PhD research at IHE Delft, Institute for Water Education and Delft University of Technology on the evaluation of green-grey infrastructures for runoff and pollutant reduction. Parallel to this, in July 2015 his Son Carlos Daniel was born and in December 2015 he moved to Santa Marta, Colombia for continuing his academic career at Universidad del Magdalena. There, he also plays clarinet with a lecturers-students music band.

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K O N I N K L I J K E   N E D E R L A N D S E
A K A D E M I E   V A N   W E T E N S C H A P P E N

The SENSE Research School declares that **Carlos Arturo Martinez Cano** has successfully fulfilled all requirements of the educational PhD programme of SENSE with a work load of 36.2 EC, including the following activities:

SENSE PhD Courses

- Environmental research in context (2017)
- Research in context activity: 'Organisation of Workshop on flood control for Melendez river Cali Colombia (2016)

Other PhD and Advanced MSc Courses

- Urban Flood Management and Disaster Risk Mitigation, IHE-Delft Institute for Water Education (2014)
- Academic writing for PhD fellows, IHE Delft (2018)
- Creative Tools for Scientific Writing, TU Delft (2018)

Management and Didactic Skills Training

- Co-organising SIMPOSIUM AGUA 2013 Manejo del riesgo en el ciclo del agua (2013)
- Supervising three MSc students with thesis (2014-2017)
- Teaching in the MSc course 'Mathematical modelling in the planning and management of water resources' (2014)
- Teaching in the MSc course 'Modelling tools in the integrated management of water resources' (2014)
- Teaching assistant for MSc course Urban Drainage and Sewerage (2017)

Oral Presentations

- *Flood resilience assessment in urban drainage systems through multi-objective optimisation*. 11th International Conference on Hydroinformatics HIC, 17-21 August 2014, New York, United States of America
- *Selecting optimal sustainable drainage design for urban runoff reduction*. 36th IAHR World Congress, 28 June-3 July 2015, The Hague, The Netherlands
- *The Urban water management: From Integrated to Interactive*. XIII Feria de la Ciencia, La Tecnología y el Emprendimiento. 25 – 27 May 2016, Neiva, Colombia
- *A quantitative and qualitative assessment of flood risks for the southern drainage system of the city of Cali*. XXI Seminario Nacional de Hidráulica e Hidrología Sociedad Colombiana de Ingenieros, 25-27 Septiembre 2014, Villa de Leyva, Colombia

SENSE coordinator PhD education

Dr. ir. Peter J. Vermeulen

Nowadays, economic development, urbanisation and heavy rainfall events are present in urban areas. A major change in approaches to the management of flooding is also ongoing in many countries worldwide. There are some decision support system tools available to evaluate green and grey infrastructures across a wide range of conditions as well as to compare alternative options. However, the performance of urban drainage systems that combines different green-grey solutions is still unclear. The present book introduces a framework for evaluation of the performance of green and grey infrastructures for runoff and pollutant reduction. To this end, it presents an evaluation of how different combinations of green infrastructure (GI) measures perform within a drainage system to reduce runoff and pollution and how the interactions between different grey infrastructures can influence the drainage system capacity. The modelling approach introduced here also combines the infiltration process, overland flow and sewer system interactions to assess the optimal combination of green-grey infrastructures for urban flood reduction. The results of this research demonstrate that including rainfall-runoff and infiltration processes, along with the representation of GI within a 2D model domain, enhances the analysis of the optimal combination of infrastructures, which in turn allows the drainage system to be assessed holistically.

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