Master’s Degree in Informatics Engineering
Thesis
Final Report

Flood Management in Urban Drainage

Contributions for the Control of Water Drainage Systems using Underground Barriers

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Department of Informatics Engineering

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Flood Management in Urban Drainage

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Abstract

The current document presents the *Masters Thesis* proposal in *Informatics Engineering*, taking place at *University of Coimbra*, and work accomplished of the candidate *Joaquim Leitão* during the 2015/2016 school year.

In this work an alternative and innovative usage for urban drainage systems was explored, in which barriers were installed in upstream underground conduits. This approach intends to use such components to retain water in upstream sections of a given drainage system, reducing water inflows that reach its downstream sections and, consequently, their overload degree.

As a result of this work, a rule-based control system was developed responsible for continuously monitoring the state of a given drainage system and performing the necessary control actions on a set of installed barriers.

This control system was applied in drainage systems with a given set of properties, being subjected to typical rainfall events with distinct profiles. This study, supported by more than a thousand experiments, enabled considerable flow reductions in downstream sections.

With the developed control system it is possible to prevent flood events within certain limits. When this task becomes impossible water withdrawals to external retention basins can be promoted, avoiding the need to perform any intervention.

KeyWords: Urban Floods, Urban Drainage Systems, Rule-Based Control Systems.

Resumo

O presente documento enquadra-se na proposta de *Tese de Mestrado* em *Engenharia Informática*, realizada na *Universidade de Coimbra* pelo candidato *Joaquim Leitão* no decorrer do ano lectivo 2015/2016.

Neste trabalho foi explorada uma utilização alternativa e inovadora de sistemas de drenagem, caracterizada pela instalação de barreiras em condutas subterrâneas localizadas nas suas secções a montante. Esta abordagem pretende fazer uso destes componentes para reter água nessas secções, reduzindo os fluxos de água escoados até localizações mais a jusante e, consequente, o seu grau de sobrecarga.

Do trabalho realizado resultou o desenvolvimento de um sistema de controlo baseado em regras responsável por monitorizar continuamente o estado de um sistema de drenagem e por gerir autonomamente as atuações realizadas nas suas barreiras.

Este sistema de controlo foi aplicado a sistemas de drenagem com um determinado conjunto de propriedades, sendo utilizadas chuvadas típicas com perfis distintos. Este estudo, suportado por mais de um milhar de simulações, permitiu alcançar reduções consideráveis de fluxo em secções a jusante.

Com este sistema é possível prevenir, dentro de certos limites, a ocorrência de fenómenos de inundação. Quando esta tarefa se torna impossível promove-se a retirada de água para bacias de retenção, evitando proceder a intervenções nestes sistemas.

Palavras-Chave: Inundações Urbanas, Sistemas de Drenagem Urbanos, Sistemas de Controlo Baseados em Regras.

Acknowledgments

The successful completion of this thesis would not have been possible without the support of a group of people to whom I owe proper acknowledgment.

First of all, I would like to express my sincere gratitude to my advisor Prof. Alberto Cardoso for his confidence in my abilities, tireless guidance, suggestions, and overall support throughout the entire period of this work.

My sincere thanks also goes to Prof. Alfeu Sá Marques and Prof. Nuno Simões, for their valuable guidance through the field of hydraulics and full availability to discuss any matter related with this work, either to clarify ideas and questions raised during this investigation, or to propose alternative insights to the discussions taking place.

I can not fail to mention the support provided by Alexandra Ribeiro, who was not only extremely helpful in the clarification of some specific terms and concepts in the field of hydraulics, but also for her important suggestions and contributions throughout the course of this work.

I would also like to acknowledge and thank everyone at *LIIS*, current and former members who I had the privilege of meeting, for providing a great environment during both working and leisure moments.

To all my friends and colleagues who unconditionally supported me not only throughout this journey but during my entire life.

Last, but certainly not the least, I would like to give a very special thank you to my family, without whose support this work would not have been possible, especially my parents and my sister.

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Chapter 1

Introduction

This document is framed in the *Master's Thesis* proposal in *Informatics Engineering* of the candidate *Joaquim Leitão*, taking place at the *University of Coimbra* during the 2015/2016 school year.

1.1 Motivation

Floods are one of the oldest and most common natural disasters witnessed around the world, affecting both rural and urban environments. Every year governments and other authorities spend many millions to cover damages caused by urban flood events.

In its simplest definition, a *flood* is said to occur when water covers an area that is usually dry. As harmless as this definition may sound such phenomena should not be regarded lightly. Because of water's tremendous strength, specially when moving, floods can severely affect human communities, being responsible for the damaging and destruction of many infrastructures, natural landscapes, farmlands and human lives.

The world's population growth led to an expansion of urban areas into uninhabited fields, in which a series of infrastructures necessary to human life, such as roads, homes and other buildings were built.

These observed urban expansion phenomena lead to an increase in the volume of residual waters generated as a consequence of human activity, which are routed to upstream sections of existing drainage systems, significantly increasing water volumes that reach their downstream sections. In addition, the occupation of new, uninhabited, fields strongly boosted terrain waterproofing, which also raises the amount of rainwater that is directed to existing drainage systems.

Furthermore, due to witnessed climate changes, intense rainfall events concentrated in short periods of time have increased between 5 and 10 % in number of occurrences per year. In fact, currently developed and installed drainage systems were conceived in a context where these phenomena were less frequent.

Building on these ideas, the vast majority of these systems have not been changed since their deployment. As a consequence of this, when substantially higher water volumes are continuously being drained to these systems and when they are subjected to more intense and frequent precipitation events their response capacity is significantly reduced. As a consequence of this, their tendency to overload has increased as well as flood occurrences in these urban environments.

On a large scale, urban floods damage assets common to all members of the affected communities, such as streets and infrastructures, but they can also have a more direct impact over individual members of these communities. Individuals' lives are also at risk, as well as their possessions, specially the ones stored outdoors, like personal vehicles. Private households can also be highly affected by floods. When large amounts of water accumulate in a given location its level increases, causing it to drain and seep elsewhere, severely damaging surrounding properties.

Increasing the capacity and dimension of existing drainage systems in urban areas could be seen as a possible solution for this problem. However, this could only be achieved by means of deep interventions in these structures. Considering the fact that physical access to underground drainage networks would be required, one could easily identify opposition forces to these interventions of both financial and social natures.

In fact, even though this fact may not be generally known, *water-related* infrastructures are an substantial part of a city's heritage. For example, in the city of Coimbra (located in the centre of Portugal) three underground conduits are usually installed bellow each road for water transportation purposes[1]. With this in mind, one can easily understand how deep interventions in systems of this nature may be rather expensive and, therefore, avoided as much as possible.

This motivated the search for alternative solutions, namely those seeking to make a better usage of these infrastructures, without the need to perform any major interventions.

Therefore there is a clear and real need to study this phenomena, developing and implementing methodologies that go far beyond simply dealing with their occurrence, attempting to increase our knowledge and understanding of these events, their causes and major impacts on human communities.

A considerable amount of work found in this topic focus on detecting and predicting urban floods, not only in temporal and spatial terms (when and where will floods occur) but also regarding the effects and consequences of such events in the affected environments.

In the current work a different approach is intended to be followed. Instead of attempting to develop a model able to perform flood prediction and impact assessment, this work aims to study existing water drainage systems, attempting to reduce the effects of urban pluvial floods as best as possible by improving the usage of these system's infrastructures.

1.2 Objectives

The main objective of this work is to reduce the effects of pluvial floods caused by an overload of downstream underground components of existing drainage systems, making use of available storage volumes in its upstream sections, whenever possible.

Such phenomena tends to occur in urban environments as a consequence of very intense precipitation, concentrated in a relatively short place and time window, also referred to as *flash floods*.

In the current work it is intended to explore the usage of barriers in underground conduits as a way of retaining water (or any other liquid that is being drained in these conduits) in earlier sections of the drainage system, thus lowering the degree of overload in downstream sections. With this study the precipitation profile boundaries that allow for such operations are sought to be identified, maximising water retention in upstream sections of drainage systems.

Indeed, it is possible to find scenarios in which these systems are overloaded in some of its downstream sections while its previous sections still have room to transport and store more water (again, or any other liquid that is being drained). Sometimes, as a consequence of this the liquid being drained in downstream sections suffers a *pressure flow* and eventually reaches the surface, where a flood event can occur. Thus, by implementing these retention mechanisms the amount of liquid that reaches later sections is intended to be reduced, as the effects of any potential resulting flood event.

As much as it was possible to ascertain, this type of approach and solution constitutes a novelty when handling the identified problem. Analysing the studied literature one can identify many works in urban environments whose objectives are comparable to those of this work. However, it was not possible to find any work that explored the usage of barriers in underground conduits to reduce flow and depth in downstream sections of a given drainage system.

In fact, documented solutions and approaches found to deal with this problem drain excess water in the drainage systems to existing retention reservoirs, where water is stored. This approach differs from the one already introduced in this section, as it mainly requires the development of additional infrastructures, not attempting to find better usages for these systems.

[1]Two used for water drainage and one used for water supply.

In general, urban drainage systems have been used mostly in a passive way, i.e., without resorting to the monitoring and actuation mechanisms aimed to, in any way, alter their operation. The innovative approach proposed in this work aims to counter this paradigm by precisely seeking a different usage and management of these infrastructures in order to increase their ability to hold and transport any liquid, thus hoping to increase the resistance of these infrastructures to more aggressive and adverse precipitation scenarios.

1.3 High Level Work Overview

In short, the presented work aims to develop a system capable of analysing information collected from underground conduits of an urban drainage system, determining how best to regulate a series of barriers, installed in those infrastructures in order to reduce the effects of a potential flood caused by an overload of the system.

Thus, an hydraulic and hydrodynamic model of part of a drainage system was developed, in which typical rainfall events were simulated. In the current work, aggressive precipitation scenarios, leading to an overload of the drainage system's underground conduits and causing water to reach the surface, will also be considered. In those situations once the expelled water reaches the exterior, it will accumulate in the surrounding terrains, roads, among other locations, promoting flooding events.

Furthermore, scenarios where the recorded rainfall is not enough to overwhelm drainage systems are also of interest to this work, as they help to understand and discover the *boundary conditions* of the system in question. That is to say that they allow to discover the worst conditions (in terms of precipitation intensity and duration) under which the system is able to drain the water without overloading.

To build this simulation environment, data and information regarding the drainage system was obtained and used. Such intelligence has to do with the drainage system's geometry, the height and length of its conduits, the capacity of installed barriers and their locations in the drainage systems, among other relevant data.

Adding to this knowledge, historical and typical rainfall data information will also feature in this work and will be provided as input to the developed hydraulic and hydrodynamic model. Since usage effects of barriers as water retention mechanisms are also important features in this work, their repercussion in the overall flow behaviour was considered and modelled.

Considering what has been said so far, it would not be at all inappropriate to think of the system to develop as a control system, whose behaviour is intended to be adjusted so that it reaches a given purpose or goal. In this sense, a *Rule-Based* controller was developed for a given drainage system, determining which actions should be taken in order to reach a given goal. In order to better determine the set of rules that should define this control system, expert knowledge of the operation mode of this kind of system and phenomena was considered.

In figure 1.1 a high level schema of the system operation is presented.

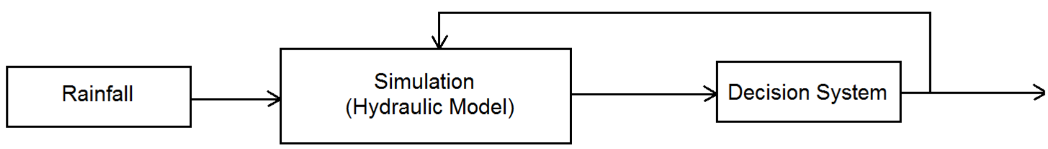

FIGURE 1.1: High level schema of the system operation.

Complementing what has been mentioned previously in this section, once the hydraulic model is built and the historical rainfall data is collected and supplied to the model it simulates water flow in the underground conduits, for a given simulation step (commonly represented by the $\triangledown$ symbol).

While performing each simulation step, the model determines new values for water levels and flow, as well as other relevant information, throughout the drainage system. These values are then provided as input data to a decision system[2], which analyses them and determines what actions should be performed on each of the installed barriers. Once this decision is made, it shall be transmitted to the model, carrying out its effects in the following simulation step.

1.4 Work Plan

In the current section a brief presentation of the planning of the work performed in the scope of this project will be made.

As presented in the thesis' intermediate defence (which took place in February, 2016) the first semester of the current school year was mainly occupied with a literature review of existing works and scientific publications related with the main topic of this thesis. Based on that literature study, a work proposal was elaborated and presented in an intermediate report, discussed during its public defence. In figure 1.2 the *Gantt Diagram* elaborated at that time is presented, with the schedule of the tasks performed during the first semester.

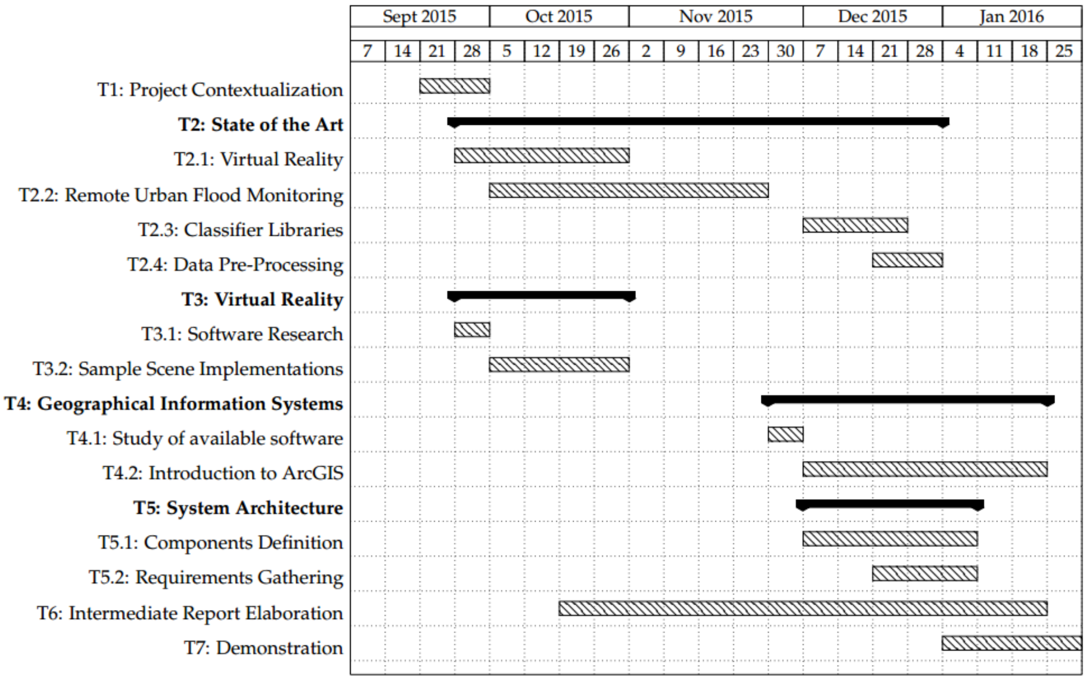

FIGURE 1.2: Gantt diagram for the first semester.

At this stage a simulator was intended to be built, in which the behaviour of both surface and subsurface components of an existing drainage system would be simulated when subjected to a given rainfall event. In this sense, there is a clear need and benefit to invest in a representation component of the simulation, which would allow an accurate visualisation of water dispersion and flow through underground conduits and at surface levels, over a given period of time. With an accurate representation of what is happening in the drainage system being simulated, a better understanding of this system and of a series of events and phenomena taking place in it can be obtained.

[2] Also referred to as the *Control System*.

However, in order to properly model water flows at surface levels more complex hydraulic models are required, considering more than one component on the fluid's movement. Among researchers, *2D* and *3D* hydraulic models are the standard choice for such operations. More detailed information regarding the complexity of hydraulic models can be found in section 3.1.1.

At the start of the second semester further investigation regarding these more complex hydraulic models was conducted, but serious obstacles were found while attempting to obtain such models, motivating a review of the work to be performed and of its objectives.

Indeed, only immediate access to hydraulic models *1D* was granted, which led to its use over higher dimensionality models.

Moreover, as already mentioned in the intermediate defence, there was an intention and willingness to explore an alternative use to underground conduits of existing drainage systems, which consisted of making use of barriers to retain water in upstream sections of these systems, regulating its flow to downstream sections.

Furthermore, the innovative nature of this approach was confirmed during the initial weeks of the second semester. Complemented with a lack of scientific research on this idea, it was considered appropriate to focus all efforts in this study, for which the available *1D* models are sufficient, thus seeking to make important contributions in an unexplored topic in the scientific community.

Having decided to redirect the work into a relatively new and unexplored topic, some of the initially planned features of this work needed to be removed. This is the case of the representation component whose inclusion was postponed to future iterations and improvements on this work. Even though it can still make valuable contributions in this new topic, its relevance to the work comes second behind the barriers' study and because of time and scope restrictions it was decided to be removed from the current work.

Thus, the work performed during the second semester was divided in the following main points:

1. Literature Review
2. Hydraulic Model Exploration, making use of typical models[3] and those similar to the ones used in some of the main stages of the current work.
3. Control System Development
4. Final Documentation

Each of these points was further decomposed in several tasks to be performed, discussed in more detail in the paragraphs that follow. In fact, since the objective of the work was reviewed and a greater relevance and importance was given to a certain topic, it was considered appropriate to carry out a state of the art study on this topic, which is why this was included in the planning and in the activities performed during the second semester.

Regarding the performed literature review it is intended to, in an initial stage, identify the most popular and recurrent techniques applied in problems of this nature. With a deeper understanding of each of these techniques, one can easily identify the main application requirements and the advantages and disadvantages of each technique. Furthermore, potential guarantees provided regarding the optimal state of the solutions obtained are also of great interest (that is, determining whether or not a given technique is able to produce global or local optimums, or neither of them).

After this study it is crucial to determine if applying any of the studied techniques would be beneficial in the current work, and if that is the case, under what conditions. In this sense, along with this point, a study focusing on hydrology and hydraulic models was planned. Inevitably, this study will focus more theoretical aspects and details of this topic, such as concepts, mathematical formulas and documented simplifications performed in some models, in comparison to reality.

[3] That is, models commonly used in an academic environment, for example during hydrology classes.

Furthermore, available and popular software that allowed the simulation of such models was also subject of interest and attention at this stage, motivating the scheduling of some exploration tasks with the goal of obtaining a basic understanding of these software solutions, their main restrictions and other relevant informations.

Regarding the second presented point, the exploration of hydraulic models was planned with the goal of extending and complementing the theoretical study conducted in the previous point.

Having developed a basic understanding of the most popular and available solutions found in the literature, and with a basic understanding of the workings of an hydraulic model it is time to move on to a more practical approach, in which a series of simulations were conducted with distinct scenarios and configurations.

Acquiring practise in the simulation of these models (and in solving punctual problems that might arise) and merging important concepts from both theory and practical applications were not only the main goals of this exercise, but also crucial steps in the success of this work. This can only be achieved by executing an extensive number of simulations in which the morphology and internal constitution of the drainage system is varied, as well as the profile of the considered rainfall events and the actuation performed on the barriers. Throughout this work more than one thousand experiments were carried out, which greatly contributed to this goal.

Therefore, in an initial stage of this point, a series of simulations will be performed featuring hydraulic models of real drainage systems. In these experiments different configurations and behaviours of the drainage systems are intended to be studied and explored, such as determining the effect in a given drainage system of reducing in half the height of its underground conduits. Additionally, in this stage, it was also planned to investigate alternative ways to automate the execution of the simulations, supporting a manual programming of certain behaviours.

In a second stage of the mentioned point, assuming that enough experience to run simulations of this nature with the selected techniques was already acquired, an exploration of the usage of barriers in underground conduits was scheduled. According to what is documented in appendix B, these experiments were planned with the goal of determining how to properly simulate and make use of underground barriers for their idealized, and already mentioned, objective.

Finally, this work will be concluded with the development of the desired control system, if it proves feasible taking into account the results of experiments mentioned above. This development comprises the projection, implementation and validation of the control system, culminating in the elaboration of the final documentation.

In short, regarding the four points presented at the start of this section, two months were initially reserved to the execution of the first point, of which one was entirely assigned to the literature review and the other to the hydrology and hydraulic models study. Regarding the second point, featuring an exploration of hydraulic models, it was projected to last between one and two months, with the first two weeks being reserved for the initial exploration and the remainder of the time being fully committed to the barrier usage study. Finally, the remaining two months were reserved for the development of the control system and for the final documentation.

Regardless of how detailed a work plan may be, once it is set in motion it becomes subjected to delays and changes of all natures. This means that, with no major surprises, the actually performed work was somewhat skewed from its initial plans. In this sense, in figure 1.3 the *Gantt Diagram* containing the work distribution during the second semester is presented.

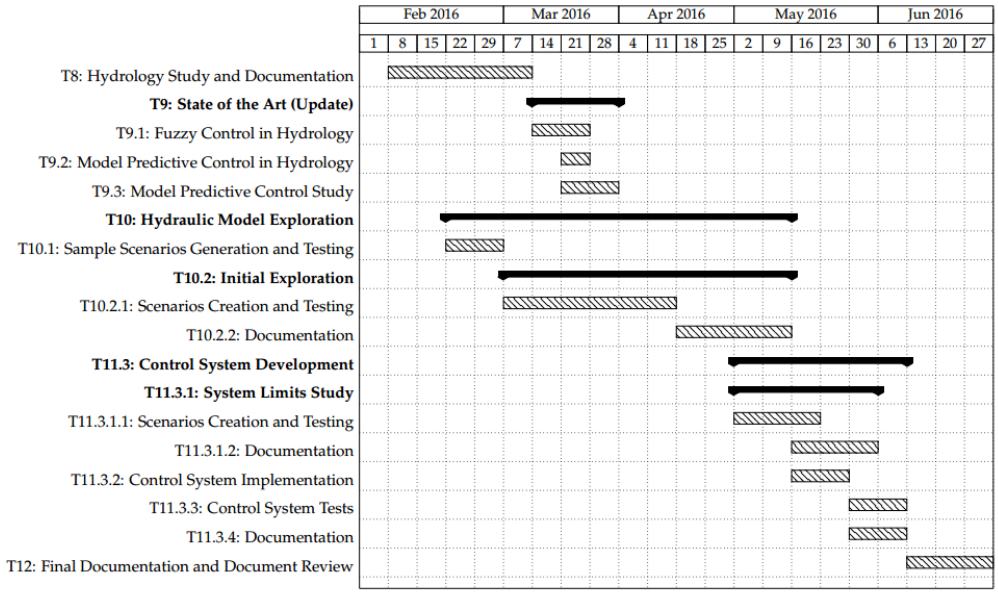

FIGURE 1.3: Gantt diagram for the second semester.

1.5 Collaborators

In the following section different entities somehow involved in this project and which made a valuable contribution to it will be presented. I would like to personally acknowledge and thank them for their important contributions to this work.

The first entity that needs to be acknowledged and thank is a research group of the *Department of Civil Engineering (DEC)* of the *University of Coimbra*, with which a collaboration was established. This research group is mostly composed of Professors Alfeu Sá Marques and Nuno Simões, specialists in urban hydrology, with many scientific contributions in this field.

This research group is currently participating in an European consortium coordinated by the University of Sheffield, UK, in the scope of the *European Union Horizon 2020 (H2020)* project *"Water Innovation: Boosting its value for Europe"*. This project aims to develop a novel low cost decentralised, autonomous RTC system for sewer networks making use of mechanically simple, robust devices to control flow in order to reduce flood risk at vulnerable sites, thus reducing local flood risk in urban areas.

From this collaboration a series of hydraulic and hydrodynamic models were provided and used in the simulations conducted throughout this work. Furthermore, the two mentioned researchers also played a key role in the initial study of hydrology and hydraulic models, often offering their help in the clarification of some concepts and questions raised throughout the conducted study. In exchange for their help, the knowledge and intelligence collected from the results of this work will be provided to this group and used to enrich their collaboration in the mentioned European project.

Alexandra Ribeiro, Civil Engineer and currently attending the doctoral program in Science and Information Technology, also had important contributions to this work. Besides offering her knowledge and experience in this area, she was also responsible for introducing Geographical Information Systems in this work which, as will be pointed out later in this document, can make significant contributions and improvements to the accomplished work.

1.6 Document Scope

This document is organised as follows:

Chapter 2 introduces a series of important concepts adopted throughout this work. Following that, in chapter 3, a state of the art review of the researched literature is presented. A small introduction to the field of hydraulic simulation and modelling is performed in chapter 4, where an introduction and clarification of some of its key concepts and tools can be found.

Chapters 5 and 6 cover the main contents of this work, presenting the strategy followed and the developed control system, as well as a series of validation experiments. Chapter 7 presents some final remarks of this work, highlighting its main contributions and presenting possible work topics to further extend and complement what has been accomplished.

Finally, annexed to the current document more detailed documentation on some of the experiments and tests performed can be found, as well as information of a different nature that provides an adequate complement and support to some ideas presented and discussed in several sections and chapters of this document.

Chapter 2

Concepts Definition

In this chapter a series of concepts adopted throughout this document and in the scope of this thesis will be defined.

2.1 Drainage System

A drainage system can be defined as a facility to transport a liquid waste over a given area. Usually drainage systems transport wastewater and rainwater, which can be disposed directly into the environment (usually in a river, lake or even in an ocean) or in a sewage treatment plant, where contaminants present in its composition will be removed so it could later be disposed into the environment.

Urban drainage systems can either be combined or separated, as mentioned by Garcia *et al.* [1] . Combined drainage systems carry both wastewater and rainwater in a single pipe, as opposite to separated drainage systems, in which wastewater and rainwater are carried in independent pipe structures.

Existing drainage systems in a given environment can also be classified into *natural* or *artificial*. In natural drainage systems excess water flows from farmers' fields to lakes or rivers, while in artificial ones pipe structures are used to carry the excess water. In many scenarios natural drainage reveals inadequate to seep all the excess water, requiring the construction of artificial solutions complete the drainage process.

Artificial drainage systems can be further classified in two distinct types: *Surface* and *Subsurface* drainage systems.

Systems of the first type are located at surface levels and their goal is to drain excess water retained in those places. This is normally accomplished by shallow ditches, also called open drains, which discharge into larger and deeper collector drains. In order to facilitate the flow of excess water towards the drains, the field is given an artificial slope by means of land grading.

Subsurface drainage systems are the most popular of the two[1]. They are built with the purpose of removing water from the rootzone, which is accomplished either by deep open drains or by buried pipe drains.

2.2 Urban Flood

The *Oxford's Advanced Learner's Dictionary of Current English* [2] defines the term *flood* as "A large amount of water covering an area that is usually dry". Based on that definition one can state that an *Urban Flood* would correspond to the same definition, applied to urban areas.

Urban Floods constitute both a social, economical and environmental hazard, as their damages to most urban infrastructures are often costly and hard to predict. Such events represent a very serious problem that continually affects our society, expected to increase in frequency as cities become increasingly populated, motivating many studies in recent years [3, 4, 5, 6, 7].

[1]In fact, the term *drainage system* is often inaccurately used when referring to subsurface systems.

FIGURE 2.1: April 15, 2015. Heavy and ongoing rainfall left streets underwater in the downtown area of Coimbra, Portugal [8].

According to [9] urban floods encompass four different scenarios: *Pluvial Flooding*, *Fluvial Flooding*, *Groundwater Flooding* and *Coastal Flooding*. *Pluvial Flooding* is related to the existence of inadequate drainage systems, unable to retain rainfall water. *Fluvial Flooding* occurs when water surpasses flood defences adjacent to rivers. *Groundwater Flooding* is an event in which sub-surface water emerges at the surface, usually as a result of persistent rainfall that overflows aquifers. *Coastal Flooding* is characterized by the flooding of a coastal area by sea water.

In the current work only *pluvial* urban floods events will be considered, that is the ones originated as a consequence of intense and prolonged rainfall events.

2.2.1 Causes of Urban Floods

The main causes for flooding events can be divided in *Natural* and *Human Causes*. Floods originated by *Natural Causes* occur naturally in a given environment, due to its climate, terrain morphology or as a consequence of other natural phenomena, without human influence. On the other hand, floods triggered by *Human Causes* occur as a result of the human presence and activity in these environments.

Among natural flood causes, the following scenarios can be found:

- *Intense and/or prolonged rainfall*, overwhelming the capacity of existing drainage systems or rivers' natural defences that cannot carry out all the water.
- *Silting*, caused by the entrainment of large amounts of sediments deposited at surface level of drain systems, reducing their capacity. In the presence of intense rainfall water accumulates in such systems and cannot be carried out, resulting in their overflow and consequent flooding.
- *Terrain morphology*. Due to its geographic characteristics a given environment may have a natural tendency to retain water. When the amount of water retained exceeds its capacity (for example, due to heavy rains) if the terrain is unable to drain it to another place a flood will probably occur.

Considering human intervention in the origin of such events one can identify a vast list of causes:

- *City growth*. Over the last few years many cities have considerably increased their effective population as a result of migration movements from rural areas, which is predicted to continue [10]. As a consequence of this, cities started demanding more resources, such as food, land, etc, leading to overgrazing and soil erosion which increases flood risk.

- *Deforestation*, which usually happens as a consequence of city growth. With the constant development and expansion of many cities, there is a need to find places where people and business can settle. To satisfy this need large forest areas are consumed, increasing soil sealing and reducing their water retention capacity.
- *Low capacity of existing drainage systems*. As cities expand and get overpopulated there is a need to increase the capacity of their drainage systems. When such changes are not properly performed these systems are unable to retain the water being carried out and overflow. Moreover, when wastewater passes through a city's drainage system it drags wastes such as sediments or garbage, drastically reducing its drainage capacity, or even blocking it. If no maintenance actions are conducted in order to properly clean these systems they can easily overflow, promoting the occurrence of floods.
- *Lack of water flow control measures*. Many fluvial and coastal cities have developed techniques to control the water level in their surroundings: Many coastal cities construct sea walls and maintain long beaches with the purpose of being protected from waves and unexpected sea level rises. Fluvial cities usually build barrages to regulate and stabilize river water elevation. However, cities that fail to implement such measures may be unable to stop an unwanted rise in the water level which can lead to a flood event.
- *Major leaks in drainage or water supply systems*, such as a burst in a water supply pipe, leading to an uncontrollable loss of water that spreads over surrounding areas.
- *Lack of awareness and implementation of preventive measures against floods*. The most common example of this situation is the construction of buildings and other infrastructures in areas with a high flood risk, or the occupation of areas created to let flood waters pass freely.

2.2.2 Impact of Urban Floods

Regarding the impact of urban floods, many classifications can be found in the literature, each one using different criteria [5, 11, 12, 13, 14, 15, 16].

One of the most simple and intuitive classifications proposed considers three categories: *Economical*, *Environmental* and *Social* [11].

The *Economical* impact of such events, as the name suggests, is related to the monetary evaluation of the damages caused by a flooding event. During floods[2] roads, bridges, houses, etc can be destroyed, forcing governments to deploy firemen, police and other emergency services to help affected people and contain the situation. All these come at a heavy cost, taking years and/or a lot of money until life in these communities comes back to normalcy.

The *Environmental* impact has to due with the pollution that these events can cause. Chemicals and other hazardous substances end up in the water and eventually contaminate the water bodies that floods end up in. Additionally, animals killed during these events can also attract wildlife to the affected areas, such as mosquitoes with some of them carrying *mosquito-borne communicable diseases* that can be transmitted to other animals or even humans.

The *Social* impact of floods goes beyond the death or injury of people during these events. Many key services and infrastructures necessary in our normal daily life activities can be compromised, namely electricity and water supply systems, possibly leading to their temporary shut down. In addition, as already mentioned, flood events can bring a lot of diseases and infections, as well as attract unwanted wildlife like, for example, insects and snakes. This not only has environmental consequences but also creates an additional hazard for people in such areas.

Flood impacts are also often distinguished between *Tangible* and *Intangible impacts* [16]. According to this definition, the key for this distinction is in the monetary quantification of such

[2]Namely in *Flash Floods* where water level rises rapidly.

events. Therefore, *tangible* impacts are the ones that can be quantified, such as the damage made to a given property or the losses of a given business. On the other hand, *intangible* impacts are related to unquantified losses, such as the loss of life or emotional and psychological traumas caused to those who experience such events.

Another popular distinction is between *Direct* and *Indirect damage*, which seems to generate some disagreement among researchers. When discussing this topic some researchers tend to focus on the effects these events have on humans. Bruno Merz *et al.* [13] defines *direct damages* as those which occur due to physical contact of flood water with humans, property or the environment, opposing to *indirect damages*, which are induced by direct impacts and occur outside the flooding event.

Other researchers [12, 14, 15] prefer to focus only on the economical impact of floods in urban environments, namely to existing businesses established in those areas. Frank Messner [14] shares Merz's definition of *direct damages* but defines *indirect damages* in a more economical view: In his definition, *indirect damages* are defined as the loss of flow values, including the costs of preventive actions adopted to fight flood damage, as well as emergency costs of actions taken to fight the event's effects.

Jonkman *et al.* [12] proposes a different distinction, where *direct damages* are defined as any loss occurring within the flooded area, while *indirect damages* occur outside this area.

Rose and Lim [15] share a different terminology for both *direct* and *indirect damages*, proposing a different distinction between *direct* an *indirect flow losses*. In their work, they relate the first term to the physical consequences of these events, including production and business losses, while the second term encompasses all economic impacts beyond direct flow losses.

2.2.3 Urban Flood Scenarios

As already mentioned in this section floods can have a serious impact in urban environments. Streets may become blocked, key running services and infrastructures of the area affected may become damaged and inoperable. In more extreme events the water can carry objects such as cars or trees, posing a serious threat to human lives in the affected environment.

Nevertheless, urban floods can also damage less notorious assets, such as private homes. Even though in these scenarios floods do not represent the same level of threat to human lives, they can still be responsible for extensive damages to individual property. Usually, private homes experience flood events as a consequence of a burst in a pipe, or because of water leaks in the home's water supply and drainage systems. When such events occur existing household appliances may become inoperable, and in more severe cases the affected home's structure may also be weakened.

Therefore, in the scope of this work two distinct flood scenarios were identified and are presented in the listing that follows.

- *Small-Scale Scenarios*, consisting of flood events that affect small buildings such as private homes.
- *Large-Scale Scenarios*, consisting of flood events in large public urban areas in which water level rose to abnormal and unexpected levels at the surface.

2.3 Mathematical and Statistical Modelling

In the scope of this thesis the term *Mathematical and Statistical Modelling* will be used when referring to mathematical models that describe the flow of a particular fluid in a given environment, taking into account its mechanical properties, interactions with the environment and mathematical equations and formulas that define the basic laws on the mechanics of these fluids.

According to this definition, hydraulic and hydrological models are amongst popular solutions of this kind, aiming to represent surface water and groundwater flows.

Additionally, the presented term will also be used to address models that conduct a statistical analysis of data collected from water supply systems, aiming to detect and identify potential leaks that could trigger a flood event.

Even though it is possible to find solutions in the field of machine learning that can be seen as mathematical models, in the current work a distinction between the two is made.

Thus, applications of *Mathematical and Statistical* models considered can be found in both controlled environments, such as pipe networks, and more extensive and *open* environments, such as roads or sections of a river's course.

2.4 Machine Learning Models

In the scope of this work the term *Machine Learning Models* will be used to reference models built with machine learning techniques. In fact, such techniques also make use of statistical models, which would allow them to be included in the term presented in the previous section.

Nevertheless, *Machine Learning* models also feature other approaches from the field of *Artificial Intelligence* such as reinforcement learning: These models are built from example inputs in order to make data-driven predictions or decisions, rather than following strictly static instructions.

Because of these differences, namely in their development methodology a decision was made to distinguish these two concepts, in the scope of the presented work.

Chapter 3

State of the Art

The current chapter is reserved to a state of the art review of the researched literature, covering the main topics considered in this work.

The first section of this chapter, 3.1, addresses the topic of flood detection and forecast in urban environments. In this section a detailed presentation of the major contributions to this field found in the literature can be found.

Following this, some popular and relevant control applications in the field of hydraulics, specially the ones applied to urban drainage systems, will be addressed in section 3.2.

3.1 Flood Detection and Forecast in Urban Environments

Detecting and predicting floods in large urban environments is far from being an easy and simple task, given the size, complexity and unpredictability of such areas and of its already settled communities.

Browsing the available literature in this topic it is possible to identify two major approaches in applications of this nature: Those focusing on the detection of flood events taking place in a given environment; and those attempting to forecast their occurrence in a particular environment.

Considering the two scenarios presented in 2.2.3, one can verify that applications targeting flood forecasting in urban environments are almost exclusively enforced on *large-scale* scenarios, while it is possible to identify flood detection applications in both *large-scale* and *small-scale* scenarios.

Furthermore, flood prediction applications tend to focus their analysis on the inspection of weather data, terrain information, among other relevant data, in an attempt to predict future occurrences of these events. On the other hand, flood detection applications are more interested in conducting an analysis more on an *"underground"* level, often monitoring the conditions of existing water drainage and supply systems in order to identify early signs of leaks that could lead to a flood event.

In the remainder of this section flood detection and forecast techniques found in the studied literature are intended to be detailed and discussed according to the flood scenarios already mentioned. Such techniques will be divided in the following categories, based on their nature: *Mathematical and Statistical Modelling*, *Wave-Based*, *Machine Learning* and *Manual* techniques.

3.1.1 Mathematical and Statistical Modelling

Mathematical and Statistical Modelling techniques are generally used to analyse and simulate water behaviour and flow in large environments and extensive supply systems.

Hydraulic and hydrological models are very popular solutions of this type, providing valid contributions to flood detection and prediction applications. In a simple way, flood modelling methods can be divided into three categories: They can be *1D*, *2D* or *3D* models. The main difference between each category has to do with the dimensionality considered for the fluid movement. To better understand such differences consider the following scenario:

Consider an underground water drainage system. Inside this system's pipes a given water volume is moving in a given direction (from upstream to downstream) and at a given rate. In fact, one could naively think that this is the only direction in which the water (or any other liquid in question) is moving. By doing such a major simplification is being performed in the modelling process, as horizontal and vertical movements of the liquid are not being considered.

Thus, the main differences between *1D*, *2D* and *3D* models has to due with the number of water movement components that are being considered by such models[1].

Néelz *et al.* [17] improved on this categorisation, dividing flood modelling methods used for this purpose in the following categories:

- *1D*, based on one-dimensional shallow water equations[2], or Saint-Venant equations [18, 19]. These methods are amongst the simplest solutions found in the literature, making them suitable to model water flows in controlled and less complex environments, such as river channels with floodplains or urban drainage networks.
- *1D+*, improving on *1D* solutions by making use of a storage cell approach to model floodplains, in which continuity equations are used to compute water levels in each storage cell. Like *1D* approaches, these methods do not include any momentum conservation on floodplains, which can ultimately result in significant errors in the predicted water levels.
- *2D-*, highly based on a simplified version of the two-dimensional shallow water equations [18]. In this simplifications, as with the *1D+* approach, no momentum conservation is taken into account.
- *2D*, consisting in hydrodynamic models based on the two-dimensional shallow water equations, making use of numerical methods and different numerical grids to compute their solutions [20]. Unlike the approaches presented so far, momentum conservation is applied in these methods.
- *2D+*, improving on *2D* methods by including a solution for vertical velocities.
- *3D*, based on the three-dimensional Reynolds-averaged Navier–Stokes (RANS) equations [21], a set of time-averaged equations of motion for fluid flows.

In their work one can verify that one-dimensional models are by far the faster, usually only requiring several minutes to execute. Following these methods, simpler *2D* approaches tend to require several hours, while purely *2D* methods can extend such time to days. Finally, both *2D+* and *3D* methods typically require several days to execute. The presented typical computation times refer to design scale modelling applications which can be of the order of 10s to 100s of km, depending on the catchment size.

Because many mathematical and statistical techniques model mechanical properties of water or other fluids, these solutions can be particularly useful when modelling the behaviour, level and flow of water over extensive drainage systems. Thus, with these models it is possible to predict not only which sections will be affected by these phenomena but also how will they be affected and how will it evolve over a given period of time.

Nevertheless, one can also find models that represent water consumption of a given entity or even that simulate the evolution of the conditions of a given supply system, in both *small* and *large-scale* scenarios. To create such models, mathematical and statistical analysis of water consumption, flow rate and pressure data are required. Once built, they can be used to make predictions of

[1] As expected, *1D* models only consider one component (the one from the upstream to the downstream), while *2D* and *3D* models consider, respectively, two and three components.

[2] The shallow water equations are a set of hyperbolic partial differential equations that describe the flow below a pressure surface in a fluid.

the analysed variables which may then be compared to measured values. Differences identified between these pairs of values, outside a given threshold range, may reveal an abnormal behaviour such as a leak.

Due to their mathematical nature and complexity, implementing these models to perform flood detection and forecasting often comes with a high computational cost and execution time. Furthermore, such models usually are not able to adapt themselves to changes in the environment, motivating many researchers to find alternatives, namely *Machine Learning* ones.

3.1.1.1 Small-Scale Scenarios

Among published work, not many approaches making use of mathematical modelling to identify and forecast floods in home environments can be found. In fact, considering that most mathematical models aim to model water and other fluids' behaviour in large environments, this comes as no major surprise.

Analysing the studied literature, flood detection and prediction applications found aimed to develop models capable of monitoring and predicting water consumptions, making use of such forecasts to detect potential leaks in supply systems. This is the case of Oren and Stroh [22, 23], who developed a mathematical model that performs statistical analysis of real time domestic water flow measurements and entity-specific data, defining an acceptance interval outside which water flow measurements can be reported as leaks.

Mohammed Froukh [24] also developed a generic and flexible decision-support system capable of making predictions on domestic water demands, based on historical individual consumption records.

One can also think of ways to take advantage of existing mathematical models tuned for extensive water supply systems and large urban environments, attempting to adapt them to domestic scenarios where considerably smaller supply systems are used: For example, one could study models used to monitor water flow in industrial supply systems and attempt to adapt them to simulate water distribution over a given set of apartment buildings, using its predictions to identify possible leaks in the apartment's water supply systems.

3.1.1.2 Large-Scale Scenarios

Environmental modelling of extensive urban areas with mathematical models has been heavily applied to flood prediction, providing valuable benefits to this field: Such models provide a clear physical and mathematical explanation of the environment being studied, allowing the creation of computer-based simulations that grant key data for decision-support and hazard management [25].

Without any doubt, *hydrological* and *hydraulic models* are by far the most popular applications of mathematical models in flood detection and prediction. In order to be used as a viable flood identification technique, a *quantitative precipitation estimate (QPE)* for the environment under study must be provided. Usually QPE is estimated based on rainfall data collected from weather radars, such as *Single-Polarization S-Band Radars (SPRs)* or *Dual-Polarization S-Band Radars (DPRs)* [26]. Once the QPE is obtained it can be fed as an input to the model, which will be responsible for simulating the water flow and computing its expected level.

Many researchers have been using such techniques to build and improve existing systems capable of predicting floods in urban environments:

Bedient *et al.* [27, 28] developed a flood warning system making use of radar-based rainfall estimation and next-generation radar equipment (NEXRAD) applied in Houston, Texas. Rafieeinasab *et al.* [29] analysed the sensitivity and spatio-temporal resolution of rainfall input data collected by NEXRAD equipment and fed to a distributed hydrological model used to predict flash floods in large urban areas.

Due to the mathematical complexity of many models their implementations often require too many computer resources and time to be executed. As a result of this many researchers have also focused on improving existing models, not only in terms of quality of the results provided, but also in terms of performance and resources required.

Guillén *et al.* [30] based their work on hydrological dynamical equations and finite difference techniques already developed, proposing a model that improved on existing ones, reducing the number of variables to be computed, thus making the model lighter. Qiuhua Liang [31] also proposed an extension of an existing one-dimensional flow model to a two-dimensional model, capable of supporting real-world flood simulations. Leandro *et al.* [32] compared coupled (sewer/surface) models of different complexities (1D/1D and 1D/2D) in an attempt to identify the key factors for setting an accurate one-dimensional model.

Finally, there has been extensive research in the creation and development of leak detection and management methods in large pipe networks, with many review and state of the art papers being published in this topic, covering many of the available and published procedures [33, 34, 35].

3.1.2 Wave-Based

Unlike *Mathematical* techniques, *Wave-Based* approaches do not attempt to model water flow at the surface level of a given environment. Instead, they focus on its underground level, using many sensor technologies to inspect the conditions of existing water pipe networks. Thus, they aim not only to detect existing leaks in such systems but also to forecast them, by identifying structural deficiencies that could potentially lead to leaks.

Considering the size of most urban water distribution networks, *Wireless Sensor Networks* (WSNs) are often used when these techniques are applied, as they allow for a continuous acquisition of information in different points of the distribution network and its upload to a central server in charge of processing the information gathered [36].

Using laser scans, sonars, magnets and other techniques, *Wave-Based* approaches aim to create a pipe inner profile so that it is possible to infer about its conditions, detecting and locating anomalies. Other popular techniques work with *leak-induced* vibration and sounds captured along the pipe to identify and locate leaks and estimate the flow of water that is travelling along the pipe [37, 38, 39]. The main idea behind these techniques is to continuously collect data from the pipe network, which is then compared with equivalent data collected in both *leakage* and *non-leakage* scenarios in order to identify situations where the collected data corresponds to *leakage* scenarios.

The main disadvantage of such techniques is that a physical access to the pipe system is often needed to install and configure the sensors to be used. In already deployed urban supply systems, this poses as a major obstacle, as direct physical access to said systems may not be always available and indirect access (by means of construction work) often has financial costs and a negative impact in urban life.

Even though wave-based techniques can be applied to small, domestic, environments among researched work only *large-scale* applications of this nature were found, as mentioned in the remainder of this subsection.

3.1.2.1 Small-Scale Scenarios

Similarly to what was presented in 3.1.1.1, domestic applications of flood detection and forecast techniques of this kind do not seem to be available in the studied literature, while applications for large, urban environments can be found more easily.

Nevertheless, as proposed for mathematical and statistical modelling, one can also analyse current large-scale state of the art flood detection and forecasting solutions and attempt to adapt some of them to small-scale scenarios.

3.1.2.2 Large-Scale Scenarios

Browsing the literature, one can find a variety of published work in condition assessment and monitoring of water distribution pipes in large supply networks.

Liu and Kleiner have made solid contributions to this field, with two state of the art paper reviews published. In the first review [38] the authors identify a series of methods to continuously monitor the conditions and performance of water pipes based on sensor data. Some of the referenced approaches include acoustic, electromagnetic, ultrasonic and microwave methods (among others) to identify and locate damaged sections in large water distribution networks.

In their second work [37], Liu and Keliner focus on the assessment of pipe conditions, proposing a division of current methods in two categories: *Direct* and *Indirect* methods. According to the authors, direct methods are concerned with an analysis of the pipe structure itself including automated and manual visual pipe inspection techniques. These methods include electromagnetic, acoustic, radiographic, thermography and ultrasound approaches, as well as some visual inspection techniques. On the other hand, indirect methods analyse the environment in which the pipe is inserted, assessing its impact in a water supply system's pipe conditions. Some of the mentioned solutions include soil characterization, pipe-to-soil interaction surveys and soil linear polarization resistance.

Martini *et al.* [39] have also presented work on leak detection in small diameter water pipelines by making use of vibration monitoring tools. The approach followed by the authors consists in installing accelerometers in connection pipes near domestic water meters. By taking advantage of the fact that vibration in water pipes increases whenever a leak is present, the authors were able to sense the pipe's vibration and detect leaks in the water distribution pipes.

Besides being used to identify and locate leaks in water distribution pipes it is also possible to join these techniques with machine learning and statistical approaches. In this scenario *Wave-Base* techniques would be used to extract features from the systems under study. The acquired features could them be processed using statistical methods and feed to machine learning classifiers in charge of identifying and location leaks. One example of such approach is the work of Daoudi *et al.* [40], in which the authors propose a plan to discriminate leaks in water distribution pipes according to their size using signals captured by hydrophones.

One could also build a system inspired in Jeffrey Cohen's patent proposal [41]. In his work, the author proposes a water flow monitoring system capable of identifying leaks in plumbing pipes. This system requires a controller that works with historical water usage data and processes real time information sensed through water pipes, distinguishing between a prolonged water use and a leaking pipe.

3.1.3 Machine Learning

Machine Learning techniques aim to develop algorithms that can learn and make predictions based on a provided input dataset (like data related to flood events). The main goal of such techniques is to perform well on data to which it has never been presented, based on its ability to learn relationships between the provided input training set.

Studying the available literature, *Artificial Neural Networks (ANNs)* and *Support Vector Machines (SVM)* are the most popular and extensively used to forecast and identify flood events, with a vast majority of applications for *large-scale* urban environments. However, it is still possible to apply these techniques to smaller and more controlled environments, as detailed in the next paragraphs.

3.1.3.1 Small-Scale Scenarios

Most *Machine Learning* techniques found are mainly used in *large-scale* flood detection and forecasting and there seem to be very few many machine learning applications tested in *small-scale* scenarios, such as the case found in [42] where a *K-Nearest Neighbour* was used to identify malfunction water meters and water leaks in unmetered common areas of apartment buildings.

Nevertheless, failing to identify a clear application of other classifiers does not mean that they are non-existent nor that large-scale solutions cannot be adapted to small-scale scenarios. For example, Firat *et al.* [43] and Maier and Dandy [44] studied applications of neural networks to predict water consumption in urban environments. Given the fact that even in small cities water consumption can be very unpredictable, but in home environments this tends to be more stable, it would be interesting to test some of the proposed architectures in domestic environments, assessing the resulting performance of the networks under study.

3.1.3.2 Large-Scale Scenarios

Most popular *Machine Learning* applications for flood detection and forecast in urban environments found in the studied literature make use of *Artificial Neural Networks* (ANNs) or *Support Vector Machines* (SVMs).

Even though ANNs and SVMs possess a number of similarities - since they are both classifiers, they are sensitive to *underfitting* and *overfitting*[3] and able to approximate any function to any desired degree of accuracy with sufficient training data - they also have some key differences: ANNs learn by applying a gradient descendent algorithm while SVMs attempt to solve a constrained optimization problem.

This has significant implications in the outcome and performance of both models: When being trained, an ANN changes its structure (i.e. the weight vector) to minimize the training error until a desired value is obtained, meaning that it is possible to have different network configurations able to obtain the desired performance for the same training data. On the other hand, SVMs solve an optimization problem, finding a unique optimal solution for each choice of parameters.

Analysing ANN applications for flood detection and forecasting in large urban environments one can find many published works. Usually ANNs applied in this context monitor data collected from underground water distribution and drainage pipe networks, predicting and detecting the occurrence of bursts or leaks in those networks. Besides such applications, publications focusing on rainfall prediction based on radar data can also be found.

Multi-layer Feedforward networks seem to be the most popular ANN type used for urban flood and leakage detection and prediction [4, 7, 40, 43, 45, 46, 47, 48]. *Fuzzy* neural networks approaches have also been used for leak detection in water supply pipes [49, 50], as well as *Recurrent* networks [51, 52].

Among *Feedforward* networks two distinct configurations emerge [3, 4, 47]: *Static Multi-layer* and *Time-Lagged* networks. While the first configuration simply encodes a temporal sequence on the input layer of the network, the *Time-Lagged* configuration provides a memory structure to the network, enabling it to store previous input values in order to deal with the temporal dimension of the data. This is particularly useful in the scenario being studied, as there is a clear dependency between successive input measurements: For example, the water level in a given distribution network, in a given moment, is not only conditioned by the water flow and pressure in that moment, but also by its values in previous moments, as well as past water levels.

One particular type of *Feedforward* network extensively used in the literature is the *Multi-layer Perceptron*. Belsito *et al.* [45] used this type of network to develop a leak detection and sizing system in pipe networks. Daoudi *et al.* [40] improved on existing applications of these networks and proposed a methodology to discriminate leaks in water distribution pipes according to their size. Caputo and Pelagagge [46] have also applied a *Multi-layer Perceptron* to detect spills and leaks from pipeline networks. In their work, two other network configurations showed good classification abilities: A *Radial Basis Function* and a *Probabilistic* neural network.

Regarding *Support Vector Machines* (SVMs), applications for flood detection purposes in pipe systems have also been published.

[3] As all classifiers are.

Mashford *et al.* [53] developed and trained a SVM to monitor pressure measured in different nodes along a pipe network. The main goal of this work was to predict the size and locations of leaks in the network. Nuno Simões [54] made use of a support vector machines to perform water level predictions in urban sewers, in different scenarios. Mounce *et al.* [55] have also used SVMs to detect abnormal water flow and pressure measurements based on real data from a water distribution system. In their approach, flow and pressure measurements were collected to create a time series and a SVM model was defined to perform regression estimation and novelty detection on the generated time series, identifying abnormal measurements that could represent a leak event.

In a slightly different context than what was being considered in the works referenced, Han *et al.* [6] used a SVM to forecast fluvial floods based on rainfall data and river flow measurements. In their approach, the authors considered two distinct input types to be fed to the model: One where future rainfall data was unpredicted and another, where future rainfall data could be predicted.

Besides *Artificial Neural Networks* and *Support Vector Machines*, studies using other approaches have also been conducted. One can find some applications of *Genetic Algorithms* to develop flood detection methods [56, 57], as well as a *self-organizing map* approach to detect leaks in water distribution networks [58].

Finally, during the late 1990's and early 2000's, many researchers proposed ANN configurations to replace popular hydraulic and hydrodynamic solutions as rainfall-runoff models, an area which has significant contributions to flood forecasting [59, 60]. Even though in recent years one can still find new applications of this nature making use of neural networks [61, 62], techniques based on support vector machines, such as support vector regression or least squares support vector machines, have emerged as better and more popular alternatives in applications of this nature [63, 64].

3.1.4 Manual Techniques

Finally, *Manual* techniques cover all actions performed by individuals to identify flood events in both domestic and urban environments.

These techniques are very popular due to their simplicity and intuitive nature, usually not requiring any specific knowledge nor being very time-consuming. The main disadvantage of such techniques is its dependency on an individual that can perform a manual inspection, being very limited in what they can detect and forecast.

3.1.4.1 Small-Scale Scenarios

In *Small-Scale* scenarios it is usually possible to easily access most components of water supply systems. Even for segments that cannot be physically accessed, periodic manual inspections should be able to detect most flood events, even if the inspection is not directly performed. Nevertheless, as was previously mentioned, these techniques are very limited in what they can detect and forecast.

Manual inspections often consist in simple procedures, such as checking the pressure relief valve on water tanks, monitoring the values registered by the water meter during periods when no water is being consumed, checking home taps, toilets, shower head, inspect visible water supply pipes, among others.

3.1.4.2 Large-Scale Scenarios

In *Large-Scale* scenarios manual techniques to monitor and identify flood events can be applied, however the application of these techniques to large-scale water supply systems are almost impossible, due to their size and lack of physical access.

3.1.5 Remarks

In the current section a series of flood detection and forecast techniques applied in urban environments were presented and discussed, split over four different categories.

In the studied works, the majority of applications found for these techniques focus on monitoring activities performed in large and extensive environments or, to use the terminology introduced in 2.2.3, *large-scale* scenarios. Furthermore, as it would be expected, techniques of different natures make use of distinct data collected from the environment under study (even though one can find common ground between these techniques).

Among *Mathematical and Statistical* techniques *hydraulic* and *hydrologic* models stand out as the most popular solutions. Such approaches are based on equations that express water behaviour in a given environment, making use of collected input data such as water flow and depth. Examples of these approaches are the works of Leandro *et al.*, Liang, Rafieeinasab *et al.* and Guillén *et al.* In these approaches, as dimensionality increases more complex scenarios can be modelled with the equations being considered, but their computational requirements also suffer a considerable boost: While simpler models can be executed in a matter of minutes, more complex ones may extend for days with high requirements in terms of resources needed for their computation.

Wave-Based approaches make use of techniques to analyse information about the internal state of pipe networks. Usually collected data used by such solutions features water flow, pressure and transportation speed, among others, registered over different points of the network. The main focus of these technique is on leak detection and location, with very few publications making use of such approaches to predict future occurrences of these events.

Machine Learning techniques emerged as promising alternatives to many mathematical models as, once fully developed and tuned for a given environment, they could potentially be easier to adapt to changes in such environment. Like *Wave-Based* approaches, *Machine Learning* solutions make extensive use of water flow and pressure data, to which precipitation data and estimations are also added, as well as other relevant environment-related information. Unlike other techniques (namely *Wave-Based* ones), many applications of this nature performing flood forecasting can be found, by producing either quantitative water level predictions over a given area, or by producing qualitative estimations for the threat level of these events.

Finally, regarding *Manual Techniques* not much can be said. Even though these techniques do not stand as very complex, they are not of much interest to the work presented in this document, as they rely on individuals to perform all the work.

Nevertheless, as already mentioned in this section, no urban flood detection and forecast application that shares the objectives of this work was found. This is, under no circumstance, an undesired situation as it allows this work to innovate and provide valid contributions to this field, and hopefully help to make a difference in this field. In chapter 5 the approach adopted to solve the presented problem will be detailed, as well as its main inspirations from the researched work, presented in this section.

3.2 Control of Urban Drainage Systems

As mentioned in 1.1 the world's population growth, namely in urban areas, has strongly contributed to an increase in the amount of wastewater drained to urban drainage systems, potentially leading to flood scenarios as a result of an overload of such systems.

One particular approach to solve such problems, which has already been mentioned in this document, involves the usage of underground water retention tanks. This approach focus not only in optimising the usage of such structures but also in determining strategic locations where these infrastructures can be installed. However, alternative solutions have been sought, mainly because of the high cost and negative impact on urban lifestyle related with the installation and usage of underground water retention tanks.

Several applications seeking to perform this management have been widely studied in different environments and urban settings. One common aspect between these works has to due with the fact that this is being addressed as a control problem, however different ways of solving it have been proposed and studied.

Hence, in the remainder of this section important concepts found in the literature regarding this field will be covered, followed by a more detailed overview of the main control applications found in the literature for urban drainage systems. Finally, to conclude the current section, subsection 3.2.2 contains a series of final remarks.

3.2.1 Literature Review

One of the most widely used concepts common to a series of publications found in the studied literature that needs to be mentioned is the concept of *Real-Time Control (RTC)*. This concept is often applied to a given system whose behaviour is intended to be controlled in a somewhat automated way. Even though the basic idea is the same, some authors also use the terms *Real-Time Control System* or real-time control of the components of a system when referring to this behaviour control.

In short, real-time control systems are closed-loop control systems where system's input data is collected and, taken into account its current state, an action that potentially changes the state of the system is performed, aiming to attain a given goal or objective [65]. Thus, the problem of RTC lies in a real-time collection and processing of input data and selection of the optimal action or control decision that enables the system to achieve a given goal (or possibly a series of goals).

Butler and Schütze [66] identify two distinct approaches when defining a RTC algorithm: *Online* and *Offline* control.

In the on-line approach the best decision to be taken is determined by evaluation of current input and state data collected from the system, supported by predictions of its future states and formal optimization procedures. On-line approaches require a description of the system being controlled, which needs to be both detailed enough, so the system can still achieve its objective, but simple enough, so computation times are minimized. Furthermore, it is also common among online approaches to define the system's objectives as a mathematical function, which is minimized in order to determine the optimal action to execute.

On the other hand, offline approaches work with a pre-defined control algorithm, usually described by a set of *"if-then"* rules or a decision matrix, that determines the control decisions for each input data supplied and analysed by the system. These rules and decisions require a knowledge of the system's behaviour and tend to be determined by a trial-and-error approach [67].

When applied to water drainage systems, a popular classification of RTC algorithms has to do with the types of variables being monitored and controlled, as mentioned by Borsanyi et al. [68]. In this classification three distinct types of RTC can be identified: *Volume-Based*, in which water volumes and depth is controlled throughout a given sewage system; *Pollution-Based*, where the main focus lies in pollution levels of the water discharged to bathing areas; and *Water-Quality-Based* ones, in which water quality at wastewater treatment plants is concerned.

Nevertheless, in a more general view, two major groups of approaches to RTC seem to be commonly identified by researchers: the ones where all control actions of the system are previously defined by a given set of rules. In the current discussion such approaches will be named *Rule-Base Strategies*; and the ones that formulate and solve a mathematical optimization problem in order to obtain the control actions that allow the system to achieve its intended goal. In the current discussion such approaches will be named *Optimization Strategies*.

In their literature review paper, García *et al.* [1] present a series of RTC strategies, which will also be covered in the following subsubsections, related to their respective model-based or non model-based nature.

3.2.1.1 Rule-Based Strategies

As already mentioned, in Rule-Based approaches the rules that control the system's behaviour are generally defined off-line, before the process starts. Because these rules are previously defined and do not change while the process is active, we can say that, in these scenarios, the control actions only depend on the current state of the system (at any given time). This means that any control action can be quickly computed, thus assuring the real-time component of rule-base control.

Borsanyi et al. [68] presented a benchmark methodology where two distinct Rule-Based RTC strategies were applied and evaluated in two virtual sewer systems: The first strategy focused on assuring an equal filling of existing storage tanks placed in different locations of the system; while the second strategy was more concerned with avoiding water spilling, namely just upstream of an existing treatment plant (to where the water was being carried). In their work the first strategy registered worst results when compared to a scenario where no control actions were being taken, while the second allowed for a reduction in the water spills registered near the treatment plant.

Fuzzy Logic approaches are also very popular in the field of rule-based control, being widely used in many problems. The choice of making use of fuzzy logic in control problems has proven to be an advantage in many situations as it allows the modelling of non-crisp membership relationships, which are similar to the model of human reasoning [69]. As an example application, Fuchs *et al.* [70] made use of fuzzy logic to develop a series of rules to, along with a sewer model, simulate the real-time control of a sewer system.

3.2.1.2 Optimization Strategies

For its part, *Optimization* strategies focus on continuously solving a mathematical optimization problem as a way of determining the optimal control actions that a given system must take in a given moment, in order to achieve a given goal. In these applications a criteria, expressed as a cost function $J(x)$, is usually being minimized or maximised. Multiple objectives can be included in the defined cost function, making the optimization problem either single-objective or multi-objective. Among these strategies six distinct techniques targeting multi-objective optimisation have been emphasized in [1].

One of the most popular and widely used techniques is *scarlarisation* [71], where the different criteria are combined into a single optimization function as the weighted sum of their individual cost functions.

Another popular technique for multi-objective optimization used in urban drainage control focus on the concept of a *Pareto-Optimal* solution [72]. In a multi-objective problem *Pareto-Optimal* solution is a solution that has the property that one objective can only be improved by penalizing another one.

Applications featuring *Evolutionary Algorithms*, extensively used in both single and multi-objective problems, applied to control of urban drainage systems can also be found. Among these solutions, *Genetic Algorithms* and *Fuzzy Decision Making* appear to be the most notable [73, 74, 69].

Population Dynamics-Based Control [75] has also been studied and applied in flow routing problems in subsurface water transportation systems. Using this approach, a controller to dynamically assign the outflow from a tank throughout a series of m possible paths was developed.

Linear-Quadratic Regulator (LQR) approaches, which aim to find an optimal controller that produces a linear control action in order to minimize an objective function, have also been explored in urban drainage systems. Marinaki and Papageorgiou [76] published a work in which LQR was used to minimize overflows in a sewer network in Bavaria, Germany, by using the available storage space in an optimal way. In a different context, Lemos and Pinto [77] explored this approach to improve delivery service in water delivery and irrigation channels.

Finally, another very popular optimization approach extensively used in real-time control of urban water transportation systems is *Model Predictive Control (MPC)* [78, 79]. This strategy uses a prediction of the system's response to a given input to establish an appropriate control action that meets the goals defined for the system. A detailed description on how MPC controllers are developed can be found in [1]. In urban scenarios, MPC has been applied to provide flooding and combined sewer overflow reductions, as stated by the following works: [71, 80, 81, 82, 83, 84, 85, 86].

3.2.2 Remarks

Analysing the available and mentioned literature in this field, two major applications of control strategies applied to urban drainage systems can be found: Those seeking to reduce or avoid system overload scenarios, and those seeking to control quality parameters of the water being transported by these systems.

While the first group of applications focus on valve or pump actuation in order to control the water level and flow through the system, applications of the second group are more concentrated with actuation on redirection gates that allow for a direct routing of the water to either a water treatment plant or directly into the environment, according to their level and concentration of polluting substances.

In fact, applications of the first group tend to be more popular and effective when applied to larger scenarios, such as a rivers, where both the channels and volume of water are bigger, providing a more comfortable margin to perform all water retention and discharge operations. In scenarios where the available volume is more reduced performing these control operations raises some problems, as there is few margin to retain the amount of water that is flowing through the drainage system.

As stated by Garcia *et al.* [1], MPC has shown to be the most successful optimization technique applied in the control of urban drainage systems, mostly due to its versatility to handle constraints and multiple objectives. The main drawback of this technique, as with all optimization approaches, has to due with the fact that a mathematical formulation of the problem needs to be performed, often by obtaining a model of the system, which can prove to be quite difficult, specially for large systems.

Rule-based and fuzzy logic may appear in advantage when compared to MPC, as they do not require a model of the system: only expert knowledge about its general behaviour is needed (which is usually easier to understand). These model-free techniques cannot formally consider an objective to minimize, as it is implicit in the system's rules.

However, the fact that they do not formally consider any objective to minimize or maximize makes rule-based approaches highly dependent on the expert knowledge of the system used to define its rule set.

In the absence of a precise system model, expert knowledge of the system's behaviour must be obtained in order to apply rule-based control strategies. Nonetheless, when it is possible to obtain a model of the system that is precise enough optimization strategies, specially MPC, leave no room for competition. Because of this rule-based approaches lost popularity in favour of optimization ones (specially MPC), which can be find in a wide range of published works, namely in recent years.

3.3 Final Remarks

The current chapter contains a state of the art literature review on two distinct topics related to the application field of the work covered in this document: Initially, popular techniques mentioned in the reviewed literature for urban flood detection and forecast were covered, followed by a series of notorious and cited control techniques applied to urban drainage systems.

In both topics, no work similar to the approach intended to follow and test in this dissertation could be found. From a scientific perspective, this is far from behind an upset. On the contrary, the fact that this approach has not been mentioned in the literature provides the opportunity to make a significant contribution to both the scientific community and urban environments.

With this in mind, the current work will take inspiration from key applications, concepts and ideas contained in a subset of the publications and respective techniques mentioned in previous sections.

Regarding flood detection and forecasting techniques extensive usage of *1D* hydraulic models will be made. These models will be used to represent and simulate a given drainage system when subjected to a given set of rainfall events and with different actuation patterns on its barriers. From the results of these simulations the effects of such rainfall events and retention techniques carried out on the barriers will be collected and analysed in detail.

Considering that no physical access to a real drainage system was granted, the incorporation of mentioned *Wave-Based* and *Manual* techniques was discarded, as their main requirement (physical access to the drainage system) was not satisfied.

Taking into account that *1D* hydraulic models were going to be used in the performed simulations, no significantly high execution times were expected to be faced. Because of this, the incorporation of *Machine Learning* techniques in this work seemed to provide no major benefits to its overall outcome.

In fact, if a representative behaviour of the system to be modelled was already guaranteed, duly certified by competent authorities in this matter, and with minor resource requirements (as simulations with *1D* models are performed in a couple of minutes with considerably low memory and processing needs), seeking to develop an alternative solution to perform the exact same task did not appear to be an important nor valuable investment in the scope of this work.

Furthermore, as it was understood from a deeper study of these hydraulic models, even though their computational complexity is not the most advanced found in the literature, they comprise a plurality of interactions in each section, aggravated by the adoption of a variable step simulation. All these facts further support the idea that, exploring *Machine Learning* approaches as a way of replacing existing hydraulic models used in this work is not the proper path to take, at this moment and in the scope of this work.

Regarding the control nature of this work, *Rule-Based* strategies will be used to develop and implement control system capable of performing the desired management of barriers installed in upstream sections of a given drainage system. The main responsibility of this control system is to, for each simulation step, process information collected from the hydraulic model in order to determine what actions should be taken on the mentioned barriers.

As will covered in more detail in further chapters of this document, namely in chapter 5, the choice of this strategy is related with the difficulty in obtaining, with the maximum accuracy possible, a simpler model of the system to be controlled (which, in this case, is a hydraulic model whereby a model of the model is sought to be obtained). As previously mentioned in this chapter, when no model of the system to be controlled can be properly obtained, *Optimization* strategies must be discarded. Therefore, a *Rule-Based* approach, combined with expert knowledge of the system to be controlled, must be considered in the development of the control system.

Finally, it is important to point out that, in this study, the use of hydraulic models mentioned in 3.1.1 *"as is"* will not be enough to accomplished the proposed objectives. For example, current popular implementations of hydraulic models of urban drainage systems do not seem to properly support the addition of barriers as a way of retaining water in underground conduits[4].

[4]This means that, in the course of this work, a proper parametrization of these infrastructures and of their actuations will be investigated.

In this sense, the implementation of behaviours of interest to this work, not supported by the hydraulic models used, will be explored and conducted. A brief summary of these contributions can be found in section 7.1.

Chapter 4

Hydraulic Modelling

The current chapter was included in this document to provide a small introduction and clarification of some concepts and tools related with the field of hydraulic simulation and modelling, used throughout the work.

The chapter starts with some considerations regarding flow simulations in urban drainage systems, followed by an introduction and presentation of the software tool used to perform such simulations. Finally, in the last section of this chapter an example of an hydraulic model will be presented. All the models presented in this document and used in this work were developed, calibrated and validated by a research group at DEC, with whom a partnership was established as mentioned in section 1.5.

4.1 Flow Simulation in Urban Water Drainage Systems

In general, urban drainage simulation models have two interlinked components: One in charge of transforming rainfall into surface runoff; and another which performs the actual flow simulation.

In the first component surface runoff is quantified thanks to the application of specific algorithms tuned to transform rainfall in runoff according to the features of the drainage basin. In the second component water's movement in the surface water collection and pipe networks is represented, having as input data surface runoff's quantification as a result of precipitation, produced by the first component.

In a drainage system water flow undergoes variations over time, which can be quite considerable and sudden during intense rainfall events. For example, when underground conduits become overloaded water may starting flowing towards the surface[1], potentially causing flood events. Such phenomena are properly represented by hydraulic and hydrodynamic models.

4.1.1 Water Flow Simulation

A popular and commonly used approach to describe a variable flow in a free surface makes use of the well-known *Saint-Venant* equations [19]. These equations are derived from depth-integrating the *Navier-Stokes* equations [18, 87] and allow the computation of water depth and flow speed across a cross section.

The application of these equations requires a set of assumptions to be verified, such as the assumption that the vertical component of the water's acceleration is negligible, its pressure is hydrostatic, the bottom has a small, fixed slope, turbulence and tension effects can be aggregated and, in a section, the horizontal speed is constant along the vertical axis.

In the current work, one-dimensional models can be used to represent water flow, thanks to the fact that water flow in each collector has a well-defined direction and a constant section. Thus, the water flow equations used in the approach proposed in this work are based on the conservative form of the one-dimensional *Saint-Venant* equations:

[1]In what is usually called *water pressure flows*.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{4.1}$$

$$\frac{1}{A}\frac{\partial Q}{\partial t} + \frac{1}{A}\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + g\frac{\partial h}{\partial x} = g\left(S_0 - S_f\right) \tag{4.2}$$

where:

A - Cross-sectional area of the flow (m^2)
Q - Water flow (m^3/s)
t - Time (s)
x - Direction of the flow
h - Water height (m)
g - Gravitational acceleration (m/s^2)
S_0 - Channel slope (m)
S_f - Friction function

Equation 4.1 represents the conservation of mass, while equation 4.2 the conservation of the movement.

4.1.2 System Overload Simulation

During intense rainfall events, the water level inside collectors can reach its limits, at which point the subsurface component of the drainage system is said to be overloaded. In such scenario two distinct flows can coexist: In some parts of the collector a *surface-free* flow may occur, while in others the water suffers a *pressure flow*. Such scenario needs to be properly modelled, as it constitutes a warning that the system's design limits have been reached, if not exceeded.

Equations 4.1 and 4.2 presented in the previous subsection can only be applied in *surface-free* flows. In order to apply them in *pressure flows* the concept of *Preissmann Slot* [88] needs to be introduced. This slot, represented in figure 4.1, consists in introducing an imaginary slit in the top of the collector, thus allowing water flowing through it to exceed its diameter and thus simulate the effects of a pressure flow.

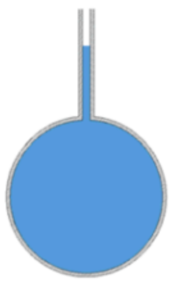

FIGURE 4.1: Preissmann Slot. Image from [89].

When the capacity of the drainage network is surpassed water flow can reach the surface. In these scenarios a simple approximation is to accept the loss of the water that reaches the surface (figure 4.2a) or an unlimited increase in water height (figure 4.2b). Another popular technique consists in storing the exceeding water volume in an artificial reservoir above surface level (figure 4.2c). Once the subsurface component of the drainage system stops being overloaded gravity brings the stored water volume back to the network.

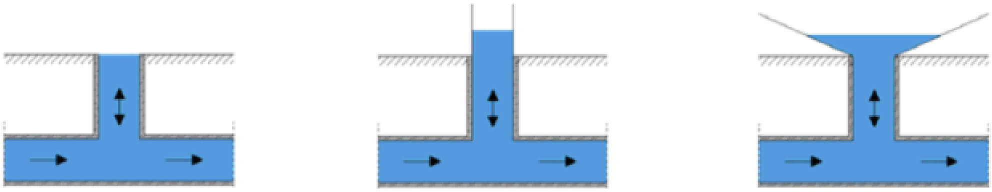

FIGURE 4.2: Techniques for dealing with an overload of the subsurface component of the drainage system: a) Drainage system overload when water that reaches surface is assumed to be lost; b) Drainage system overload when water that reaches surface is assumed to be able to increase unlimitedly in height; c) Exceeding water volume stored in an external reservoir during flooding periods. Images from [89].

4.1.3 Dual Drainage

As already mentioned in section 2.1, urban pluvial drainage systems consist of two major and distinct components [89] : A surface component, composed of streets, natural and artificial channels, depressions, among others; and an underground component (also mentioned as subsurface component), composed of a collector network.

When the capacity of existing collectors is overcome water exits them through gutters, drains and manholes, emerging and concentrating at the surface. Such surface water excess volume can be driven at low points, infiltrate (feeding existing groundwater networks), reenter the collectors system or cause runoff.

The *dual drainage* concept introduced by *Djordjević* [90], and presented in figure 4.3, aims to provide a more realistic insight of floods in urban areas. With this concept it is possible to model the interactions between the two systems, particularly between collector networks (that can partially be overloaded) and runoff in open spaces (city streets, between houses and land depressions, among others).

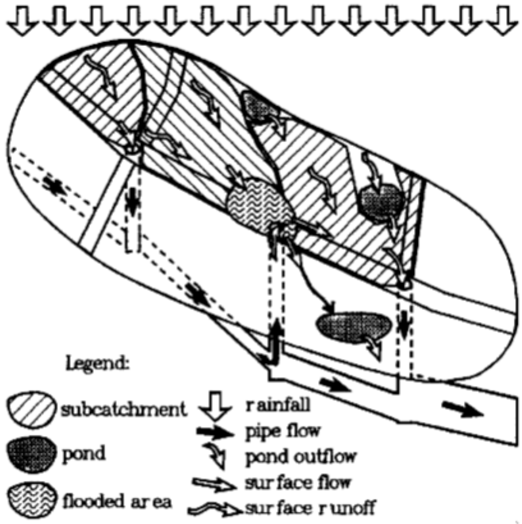

FIGURE 4.3: Dual Drainage schematic representation. Image from [90].

Currently there are two major approaches for dual drainage simulations: Both of them feature a one-dimensional model for the underground collector network, but one of them makes use of a one-dimensional model (1D/1D) for the surface while the other uses a two-dimensional model (1D/2D).

In the 1D/1D scenario urban surface is treated as a network of open channels and water retention areas, thus forming a set of channels and nodes connected to the collector network. For the

1D/2D scenario no previous identification of surface channels or water retention areas is needed. Finally, in both approaches surface and underground networks are connected and computed simultaneously.

In well-defined channels, such as in underground conduits or while the water remains inside the street profile, the 1D/1D model provides a good approximation of the reality. However, once the surface flow exceeds the street's boundaries, a two-dimensional approach becomes a better modelling solution.

4.2 Storm Water Management Model and Hydroinformatics

During the previous sections and chapters of this document a lot has been said regarding flow simulation and the use of hydraulic models to perform such simulations. Namely, in section 1.5 it was mentioned that a series of hydraulic and hydrodynamic models had been developed to be used in the simulations conducted throughout this work.

However, up until this moment, there has been almost no mention about how can one produce any sort of valid result from these models, or how are these models built. The current section was introduced exactly to clarify these aspects.

In the current work, such models were developed and executed in *Storm Water Management Model (SWMM)*[2], an open-source dynamic hydrology-hydraulic water quality simulation model developed by *The United States Environmental Protection Agency (EPA)*.

SWMM is a dynamic rainfall-runoff-routing simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. In a simulation period computed over variable time steps, it tracks flow rate, depth, quality and quantity of runoff generated within each subcatchment, pipe and channel in both surface and subsurface levels of a drainage system.

In the remainder of this section some considerations about the choice of SWMM will be made, followed by a brief explanation of how hydraulic models can be created in SWMM and how it executes them. Complete and extensive documentation on this software can be found at [91, 92] .

4.2.1 Why using SWMM?

One of the main reasons that motivated the choice of this software as the main platform for the development and execution of hydraulic models has to do with its considerable popularity and acceptance in the hydraulic and hydroinformatics research community [92] .

Besides providing a very simple and easy-to-use interface this software features the most common implementations to solve the equations governing water flow across the drainage system. Thus, SWMM users can choose between different implementations to be used in their simulations. Furthermore, internally SWMM performs all computations with a variable time step, which improves the accuracy of its computations, as it can adjust the simulation time step according to the computed disturbances and variations in the equations being solved.

In addition to this, the fact that this is an open-source software allows individual developers to perform small adjustments to the tool, which had a significant impact in the information extracted from each simulation performed in this software: Individual developers are able to implement functionalities that allow the retrieval of more information collected during and after the execution of each simulation. Furthermore, it is possible to improve on this software by adding functionalities of other natures, such as the possibility of changing the conditions of some structures as the simulation progresses.

[2] `https://www.epa.gov/water-research/storm-water-management-model-swmm`

Having said that, after an initial exploration of this tool, and after realising that it would allow us to perform the operations we desired, a decision was made to make this the primary hydraulic model development and execution tool in this work.

4.2.2 Conceptual Model

According to [91], SWMM conceptualizes a drainage system as a series of water and material flows between several major environmental compartments, including:

- An *Atmosphere* compartment, from which precipitation falls. In SWMM, precipitation inputs are represented using *rain gage* objects.
- A *Land Surface* compartment, composed of one or more *Subcatchment* objects, where precipitation from one rain gage in the atmosphere compartment is received. Precipitation in these subcatchments can be in the form of either rain or snow. Once precipitation has been received in each subcatchment object it can infiltrate into the *Groundwater* compartment or generate surface runoff and pollutant loadings to the *Transport* compartment.
- A *Groundwater* compartment where water infiltration from the *Land Surface* compartment is modelled, making use of *Aquifier* objects. Since a fraction of infiltrated groundwater can still reach the subsurface component of some drainage systems, SWMM allows a portion of the water infiltration to be transferred to the *Transport* compartment.
- A *Transport* compartment, which contains a network of elements (such as channels, pipes, pumps and regulators) in charge of water transportation to outfalls and treatment facilities. Such elements are modelled using *Node* and *Link* objects, which will be detailed further in this section. Inflows to this compartment come from surface runoff originated in the *Land Surface* compartment, groundwater interflow, sanitary dry weather flow or from user-defined hydrographs.

In the subsections that follow a brief presentation of each compartment's objects used in the models developed in this work is made, based on the work of Rossman and Huber [93]. More detailed information on the properties of the elements mentioned in the following subsections is available in their work.

4.2.3 Atmosphere Compartment

In the *Atmosphere* compartment *rain gages* appear, as expected, as the most important and relevant object to use in most SWMM applications. These objects supply precipitation data for one or more subcatchment in a given environment.

Precipitation data is supplied in a time series, which can be directly defined in the SWMM application, or provided from an external file. In both scenarios rainfall data consists, in its simplest form, in a series of *(time, value)* tuples.

Currently SWMM is supporting three distinct rain formats in which precipitation values can be measured and supplied to the application: rainfall intensity, volume or cumulative rainfall information.

Besides rainfall and rain gage properties, in this compartment it is also possible to define a variety of atmospheric simulation parameters, such as the temperature, wind speed, evaporation and snowmelt rates, among other.

4.2.4 Land Surface Compartment

As previously mentioned, the *Land Surface* compartment is composed of a set of *subcatchments*. These elements are hydrologic units directing surface runoff to a single discharge outlet point, which can be either nodes of the surface component of the drainage system or other subcatchments.

A subcatchment can be divided into pervious and impervious subareas. The first ones are permeable to water, allowing it to infiltrate into the *Groundwater* compartment. On the other hand, impervious areas are impermeable to water, and can be further divided into two distinct areas: The ones featuring depression storages and the ones without them[3].

4.2.5 Groundwater Compartment

In order to model water infiltration and flow at an underground level SWMM makes use of an *Aquifer* object, which is also the main component of this compartment.

Aquifers are subsurface groundwater areas where the vertical movement of infiltrating water is modelled. In SWMM, aquifiers also allow for groundwater to infiltrate into the drainage system, or to exit such systems, thus feeding the aquifer.

The behaviour of an aquifer is characterized by a series of parameters such as soil porosity, hydraulic conductivity, evapotranspiration depth, bottom elevation, and loss rate to deep groundwater.

Aquifers are only required in models that need to explicitly account for the exchange of groundwater with the drainage system or to establish baseflow and recession curves in natural channels and non-urban systems.

4.2.6 Transport Compartment

The *Transport* compartment features a series of objects representing structures of real drainage systems, used to transport water across the system up to its discharge point.

Among all elements of these nature represented in SWMM, in the current work *junctions*, *conduits*, *orifices*, *outlets* and *outfall* nodes were used in the developed models, and will be briefly detailed in the remainder of this subsection.

4.2.6.1 Junction Nodes

The main function of junctions nodes is to join links of the drainage system together. Physically these structures can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connection fittings, allowing external inflows to enter the system. An image of a real junction node installed in the subsurface component of a drainage system is presented in figure 4.4.

When conduits connected through a junction node become surcharged excess water in junction nodes can start to be partially pressurized and either be lost in the system or allowed to pond atop the junction and subsequently drain back into it.

[3] A depression storage is a natural depression in a given environment that much be filled prior to a runoff occurrence

FIGURE 4.4: Junction Node structure example[4].

4.2.6.2 Conduits

Conduits are pipes or channels of both surface and subsurface components of the drainage system that move water from one node to another in the conveyance system. Figure 4.5 presents the interior of an underground conduit.

The cross-sectional shapes of these elements can be selected from a variety of standard open and closed geometries, including circular, parabolic, trapezoidal, rectangular, among others. Most open channels can be represented with a rectangular, trapezoidal, or user-defined irregular cross-section shape. Underground pipes are commonly defined using circular or elliptical shapes.

SWMM uses the *Manning equation* to express the relationship between flow rate (Q), cross-sectional area (A), hydraulic radius (R), and slope (S) in all conduits. For standard U.S. units, this equation makes use of the Manning roughness coefficient (n) and is defined as:

$$Q = \frac{1.49}{n} \times A \times \sqrt[3]{R^2} \times \sqrt{S}$$

SWMM also supports the use of alternative equations to define the flow rate in a given conduit, including the *Hazen-Williams* or the *Darcy-Weisbach* equations. The choice of which equation to use is a user-supplied option.

For U.S. units the *Hazen-Williams* formula is defined as:

$$Q = 1.318 \times C \times A \times R^{0.63} \times S^{0.54}$$

where C is the *Hazen-Williams* C-factor which varies inversely with surface roughness and is supplied as one of the cross-section's parameters.

For U.S. units the *Darcy-Weisbach* formula is:

$$Q = \sqrt{\frac{8 \times g}{f}} \times A \times \sqrt{R \times S}$$

where g is the acceleration of gravity and f is the *Darcy-Weisbach* friction factor.

[4]`http://northernbeachesplumbing.net.au/portfolio/`

FIGURE 4.5: Interior of an underground conduit.

4.2.6.3 Orifices

Orifices are used to model outlet and division structures in drainage systems, typically openings in the wall of a manhole, storage facilities or control gates. In figures 4.6 and 4.7 two examples of these structures are presented, used in small conduits, or in larger, industrial-size applications.

Internally, SWMM represents an orifice as a link connecting two nodes, which can have a circular or rectangular shape located either at the bottom or along side the upstream node. The opening of this shape can be changed over time, controlling the flow that passes through the orifice. These elements can also have a flap gate to prevent backflow.

The term *orifice setting* is also very used among several authors and in a wide variety of applications of this nature when referring to the orifice's opening state: Usually the setting of an orifice is mentioned as a decimal number between 0 and 1, with 0 being used when the orifice is fully closed, and 1 when fully open.

As the orifice's setting is being reduced so is the area through which water flows. However, this does not happen in a direct proportion to the variation of the setting, nor in an uniform way across its entire aperture: The closure of the orifice's opening occurs in a vertical movement, from top to bottom[5].

In the current work orifices will be used to model the structure and behaviour of the barriers installed in the drainage system's conduits, to regulate water flow inside them.

When an orifice's opening is fully submerged the flow passing through it is computed as:

$$Q = C \times A \times \sqrt{2 \times g \times h}$$

where h is the head difference across the orifice, measured in meters.

When an orifice's opening is only partially submerged the flow passing through it can be computed as:

$$Q = C \times A \times \sqrt{2 \times g \times D \times f^3}$$

where D is the height of full orifice opening and f is the opening fraction that is submerged.

[5] A video illustrating the behaviour of these infrastructures can be found at `http://steinhardtgmbh.com/videos/hydroslide/`.

FIGURE 4.6: Small-scale example of an orifice[6].

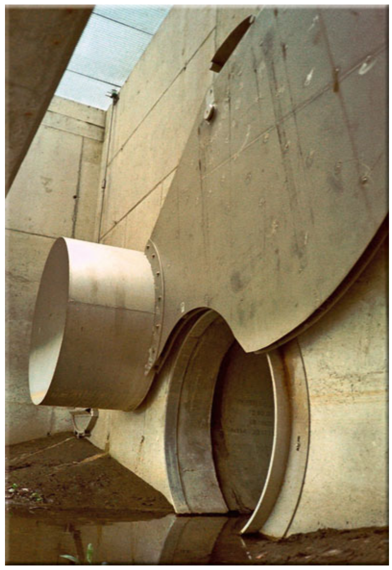

FIGURE 4.7: Orifice used in industrial-size applications[7].

4.2.6.4 Outlets

Outlets are flow control devices typically used to control outflows from storage units, modelling special head-discharge relationships that cannot be accomplished with pumps, orifices, or weirs.

Similarly to orifices, outlets are internally represented in SWMM as a link connecting two nodes and can also have a flap gate to prevent backflow. A user-defined rating curve determines the outlet's discharge flow, as a function of the head difference across it.

Just like with precipitation values, SWMM offers two alternatives when defined such function: The user can supply a set of points that compose the functions [8]. Alternatively, a functional curve function in the form of $Q = A \times y^B$ can also be used, where the user can define the values of the parameters A and B. In this equation the value of y stands for the water depth.

4.2.6.5 Outfall Nodes

Outfalls are terminal nodes of the drainage system used to define final downstream boundaries in the model created, when *dynamic wave flow routing* is being applied. For other flow routing types outfall nodes can also behave as junction nodes. In SWMM, an outfall node can only be connected to a single link.

[6] `http://steinhardt.de/images/flyer/hr_t.png`
[7] `http://steinhardt.de/wp-content/uploads/steinhardt-image09.jpg`
[8] That is, pairs of tuples *(x,y)*.

FIGURE 4.8: Outfall[9].

4.3 Application

In the developed work a dual hydrodynamic 1D/1D model was used to simulate water flow in an urban drainage system. Thus, 1D models were used to simulate water flow in both surface and subsurface components of the drainage system.

According to the members of the research group at DEC (with which a partnership was established for this work, as referred in section 1.5) the adoption of 1D dual drainage models is a perfectly acceptable decision, which allows their execution in a short time window and with results that are not far from what happens in reality.

In fact, with regard to underground water flow, considering the simple geometry and boundaries of the conduits, the components neglected by 1D models have a very small contribution to the overall outcome of the simulation. As for surface flow, 1D models assume considerable simplifications when compared to reality. However, the higher computational costs associated with more complex models (such as 2D or 3D) make the simulation process extremely time and resource consuming.

As supported by the research group at DEC, these higher computational costs are not rewarded for their better adaptation to reality, at least taking into account the objectives of this work. In this sense, the choice of a dual 1D/1D model is justified.

To facilitate the simulation process, either in depth or on the surface, the drainage system under consideration was divided into several sections individually processed.

At a surface level each section is defined as an open rectangular channel, with a width of 8 meters, representative of the width of most roads. Surface flow in this environment occurs by gravity, since the road is also characterised by a given slope, defined by the height difference between adjacent channels.

At an underground level, each section is defined by a circular channel, representative of an underground conduit, which has a previously defined height. Just like in surface flows, at the underground level water flows occur by gravity, with a slope between adjacent channels also being defined.

In this model communication between the surface and subsurface components is accomplished by means of circular channels similar to underground conduits. Through them water can enter into the underground system through grills or collectors that are on the surface. Conversely, when

[9] `https://i.ytimg.com/vi/UwYk9x8ldw8/maxresdefault.jpg`

the subsurface component of the drainage system is overloaded a pressure flow may occur, in which water levels rise, forcing it to leave the system into the surface. In the *SWMM* software the existing structures to represent this ascending or descending flow of water only allow it to be done one way, so it is necessary to duplicate these structures in order to reflect the two scenarios.

In underground conduits barriers on which the control system operate can be found. In the used and developed models these barriers are defined by orifices with a given opening, used to determine the water flow passing through them.

In figure 4.9 an example of a 1D/1D dual model, developed in the *SWMM* software is presented.

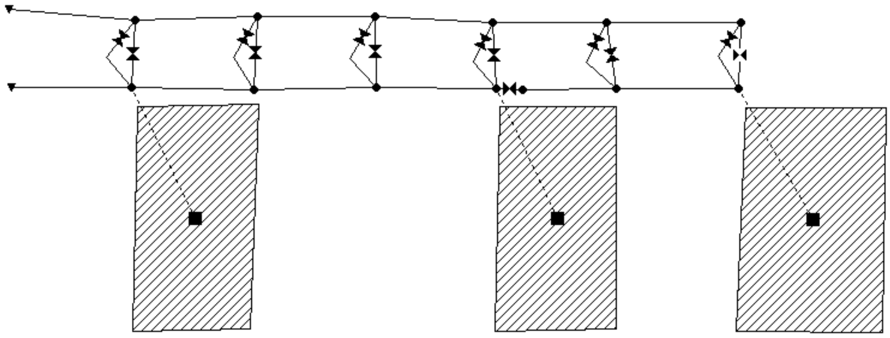

FIGURE 4.9: Example of an hydraulic model developed with the *SWMM* software.

Observing the model one can identify the existence of two horizontal channels, composed of multiple links and junction points. In fact, each of these channels represents one of the two components of a drainage system: The first, above, is the surface component of the drainage system, while the second one represents the subsurface component of the system.

The dots observed in these components correspond to the junction nodes presented in section 4.2.6.1 while the horizontal lines are the conduits, presented in section 4.2.6.2. As already mentioned the two components of the channel are connected with vertical channels, represented by *outlets*, which only allow for water flows in one direction, meaning that they appear in duplicate, thus allowing for water flow in both ascending and descending directions.

In the subsurface component of the system, at the end of the second section an orifice has been added (the object with the shape of a sand clock, rotated by 90 degrees). Regarding this object it is important to note that, even though this would not apply in a real scenario, in the *SWMM* software all orifice object must be placed between two junctions. Therefore, as supported by figure 4.9, the presented orifice is placed between two junctions, although the right-most one is not a part of the drainage system (its addition to the model is merely a software imposition).

Finally, in the presented figure rectangular shaded structures have also been included. These structures represent the subcatchments mentioned in section 4.2.2, where precipitation is received. These structures are connected to a junction where collected precipitation precipitation water is drained, thus entering the system.

In this system water flows from the right to the left. In the last section two triangular-shaped structures have been placed. These structures represent outfall nodes, presented in section 4.2.6.5, and are used as an exit point for the water in model. In reality this water would drain to later sections of the drainage system, which are not represented in this model.

Chapter 5

Approach

The current chapter is, possibly, the most important of the entire thesis, in which the main object of study of this work is presented and discussed.

This chapter contains the main results and contributions of the study conducted during this work of which resulted a control system, developed with the intention of applying the proposed new approach to a particular set of drainage systems, which share amongst them a given group of properties.

Thus, in section 5.2 a presentation of the experimental setup used in this work, where the study environment for which the control system was developed can be found. In section 5.3 a brief overview of the different steps carried out during the course of this investigation is presented. Following this, section 5.4 contains a series of experiments, performed on the environment detailed in 5.2. The control system produced as part of this work will be presented in section 5.5. Finally, section 5.6 contains some final remarks regarding what was presented at this point.

5.1 Introduction

As already mentioned in this document, this work intends to explore alternative usages for existing water drainage systems that aim to retain excess water in its upstream sections, thus reducing water levels in downstream sections.

If confirmed to work, this approach can prove quite helpful in scenarios where upstream sections are not operating at full capacity, but downstream sections are already overloaded and unable to drain all its inflows, originating ascending water pressure flows towards the surface in these sections.

By making use of barriers installed in upstream sections of a given drainage system excess water arriving in such places can be stored, thus reducing water volumes drained to downstream sections and, by extension, causing levels in such locations to drop.

Thus, in the current work, a control system capable of managing the actuation on these barriers, for drainage systems that share a given set of properties, is intended to be built. This controller should analyse data collected from a given drainage system and determine how to actuate on these structures in order to store the maximum volume of water possible in earlier sections of the system, without them becoming overloaded. In other words, this controller should monitor a given drainage system, "reacting" to a particular sequence of events, aiming to achieve a previously defined objective.

Considering that no access to a real system to be controlled has been granted, this entire work was performed based on hydraulic models of drainage systems. Therefore, the controller to be developed will monitor the evolution of the conditions of a representative drainage system, designed and simulated using an hydraulic model. To accomplish this, as already mentioned in section 4.2, the *SWMM* software will be used. The hydraulic models used in this work were developed and validated by a research group at DEC, as mentioned in section 1.5.

Studying the available literature in the subject of control, two different ways to design and define the controller's operations can be identified: according to *Rule-Based Control* strategies, all actions that can be performed are previously defined, usually by a set of *IF-THEN* rules; alternatively, an optimisation problem may be formulated and iteratively solved, from which a set of actions that should be adopted in order to attain the desired goal is derived. In the discussion taking place in this document approaches of this nature were named *Optimization Control Strategies*.

Optimization strategies rely on the definition of a model of the system being controlled, describing its behaviour with a certain degree of accuracy. These models are described by a set of differential equation representing the existing interactions between the systems' inputs, outputs, perturbations and states.

Given the complexity of the hydraulic models used in this work, its translation using this type of models does not seem a feasible option. García *et al.* [1] suggest that, when no appropriate model of the system can be obtained, the only feasible way of attempting to control the given system relies on the definition of a set of rules that govern the actions performed on the barriers. Furthermore, García *et al.* recommends that the definition of these control rules relies on expert knowledge of the behaviour and characteristics of the system being modelled, performed in a *trial-and-error* process.

In a very simplistic view, the controller to be developed in this work could be seen as a series of *IF-THEN* rules that govern its behaviour, determining what actions should be performed in the barriers in the presence of a given set of (previously defined) scenarios. Such rules are obtained as a result of an extensive process of definition, test and correction, according to what was mentioned in previous paragraphs.

In reality, the control system to be developed is not going to be that simple. Given the complexity wrapped around the definition of possible sequences of events that can occur and the respective reactions taken in the barriers, it will be implemented with a state machine.

Transitions between states will be carried out only if a given set of predetermined conditions are satisfied. Each state of this system has a certain set of actions to be taken on the barriers, which may be dependent on the current state of the system[1].

Even though the software to be used in the development and simulation of the hydraulic models allows the definition of some rules that govern actuation on barriers, these rules must all be defined in a single block of *IF-THEN-ELSE* instructions. As stated in the previous paragraphs, a solution that allows for a higher degree of freedom in the definition of the controller's behaviour is required.

In this sense the *MatSWMM* library[2], developed by Gerardo Andrés Riaño Briceño, was introduced and used in this work. This library provides modules in several programming languages (namely *Python* and *MATLAB*) that can interact and simulate models developed in the *SWMM* software. Therefore, by making use of a high-level programming language in the implementation of the control system more complex behaviours are easier to define. In the course of this work the *Python* version of this library was extensively used in all the performed experiments[3].

Before deepening into particular aspects of the definition and development of the control system, it is important to provide a general overview of the path followed which allowed its definition, presented in further sections of this chapter.

Therefore, the remainder of this chapter will be structured as follows: in section 5.2 the hydraulic model and the scenarios used throughout the work will be presented and shortly discussed. Following that, in section 5.3 the strategy adopted in the study of these systems, and in the development of the control system, will be briefly presented. In section 5.4 a series of intermediate experiments will also be presented and discussed. Finally, the developed control system will be presented in section 5.5.

[1]That is, water levels and flows in underground conduits, junctions and outlets of the system, among other relevant information.

[2]`https://github.com/water-systems/MatSWMM`

[3]As a side note, at the moment of writing of this document the *MatSWMM* library has no support for *Python3*. For this reason, in this work, the *Python2* version was used.

5.2 Experimental Setup

Studying the applications and publications mentioned in 3.2 it was not possible to find approaches and goals similar to the one proposed in this work. In fact, as much as it was possible to ascertain, this type of usage for barriers in underground conduits and drainage systems has never been attempted (or, at least, no published works describing such approaches were found in the conducted literature study). If, on one hand, this underlines the novelty proposed in this work, which from a scientific point of view has a substantial value, it also requires a greater effort and investment: for example, understanding how to properly model the behaviour of these structures in current and popular hydraulic model software is crucial for the success of this approach.

In this sense, an intensive exploration of the *SWMM* software and of some hydraulic models was initially conducted. Because no previous work experience with *SWMM* had been gathered before the start of this work, this study aimed to, in an initial stage, acquire experience and sensitivity in the execution and parametrization of some hydraulic models.

Additionally, this study also hoped to understand how to properly model the usage of barriers in underground conduits in the *SWMM* software. Detailed coverage of this exploration, including the models used, the scenarios sought to be obtained and the results of control operations carried out can be found in appendix B.

With the knowledge and expertise acquired with this extensive study it was possible to move forward in the work, towards the definition of the control system and its operating rules.

Considering that different architectures, extensions and internal constitutions can be part of distinct drainage systems, in the current work one drainage system with a given constitution and structure, presented in figure 5.1, was used.

Indeed, performing a deeper study aiming to examine the effect of variations in the dimensions and architecture of the structures that constitute the drainage system would be an important complement to this work. However, given the time restrictions imposed on this work (which needs to be completed in the 2015/2016 school year), and considering the investment performed in the study of concepts and hydraulic models, a decision was made to maintain the same drainage system. In fact, extending this study to systems with different architectures and characteristics is also a very demanding and extensive task, which can be resumed in future iterations of this work.

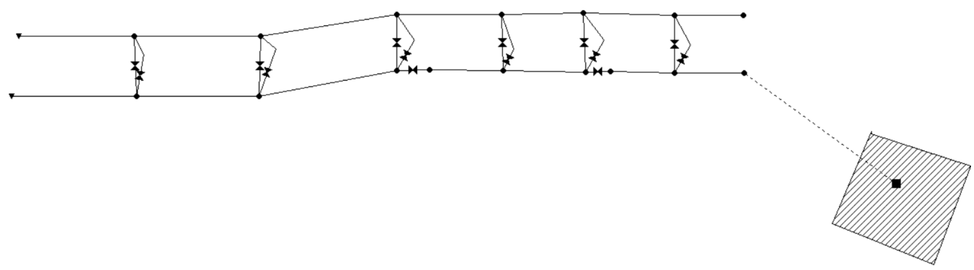

FIGURE 5.1: Scheme of the hydraulic model used as a baseline in the performed experiments.

As presented in the figure above, a model of a dual drainage system was developed, consisting of seven sections. The first four sections and the last two have a slope of about 0.3%, while fifth section has a more significant slope, rounding 12%.

As mentioned in 4.3, at a surface level each section of the system is represented as an open rectangular channel. In the current scenario all surface sections of the drainage system were given a length of $40\ m$ and a bottom width of $5\ m$.

All underground upstream conduits were given a height of $1\ m$, while downstream conduits (including the one in the fifth section) have a height of $0.8\ m$. Similarly to upstream sections, downstream conduits have a fixed length of $40m$. The height difference between both components is 3 m in the initial four sections and 2 m in the last two sections.

Similarly to other models presented in this document, each section of the drainage system contains two outlets assuring communication between the surface and subsurface components of the system (one in which all ascending water flows occur, and another in which all descending flows occur).

In the subsurface component of the system, two orifices have been placed at the end of second and fourth sections. As mentioned in section 4.2.6.3, orifices are used in the *SWMM* software to model the structure and behaviour of underground barriers. Their goal is to retain excess water, preventing it from draining through the remainder of the system. In an initial stage, experiments with only one orifice were conducted. However, this proved to be insufficient to retain and properly accommodate considered precipitation events, motivating the decision to add a second orifice to increase water retention capacity.

The choice of the location where the second orifice was installed had to take into account the location of the other orifice. In fact, when installed in an underground conduit, an orifice causes an impact in conduits upstream of its location up until the last conduit placed at a depth equal to the difference between the orifice's depth and height. Considering that the first orifice was placed in the section immediately before the steepest slope, the second orifice had to be placed in a conduit at a depth at least 1 meter higher (because 1 meter is the height defined for the orifices), thus it was placed in the second section.

Regarding the adopted naming conventions, as in the experiments presented in appendix B, the conduits and junctions were named, respectively, *C-<conduit_number>* and *J-<junction_number>* sequentially from the rightmost conduit of the subsurface component of the system to the leftmost conduit of its surface component. The outlets in the model followed a similar naming convention: *L-<outlet_number>* from the rightmost outlet to the leftmost outlet. Finally, an opposite naming convention was adopted for the orifices: the upstream orifice (that is, the rightmost one) was named *O-2*, while the downstream orifice (the leftmost one) was named *O-1*.

Precipitation data sources used at this point were also subject of attention and care in this study. Rainfall events with a profile that gathers characteristics of several typical precipitation events responsible for urban flood scenarios were generated and used at this stage. Similarly to what was performed with the generation of the hydraulic models, these typical rainfall events were obtained and validated as a result of the partnership established with the research group at *DEC*, as mentioned in 1.5.

In figure 5.2 one of the used rainfall events is presented, whose profile matches what has been stated in the previous paragraph. The respective impact on water levels in downstream conduits of the system, when no actuation is being performed on the barriers are also presented in figure 5.3.

In fact, the profile of these artificial rainfall events differs substantially from that of the events used in the experiments conducted in appendix B. At this stage, less aggressive rainfall events with smoother precipitation intensity variations were sought. Indeed, from those tests it became clear that if the measured precipitation intensity keeps increasing for considerable periods of time, then actuation on the barriers needs to be very smooth.

The reason for this has to due with the fact that when one actuates upon the installed barriers water levels in upstream conduits tend to rise. If, adding to that, the amount of water that reaches such conduits is continuously increasing then the drainage system will become overloaded in a very short period of time. One could try to fight this by relaxing actuation in the barriers but, by doing so, the amount of water that is being stored in upstream conduits starts decreasing, possibly becoming negligible.

Because of this the need to consider less aggressive rainfalls became apparent, leading to the generation of these precipitation events, with which smaller water levels in upstream conduits were achieved, allowing a better control and water retention in those structures, easing the control system's development process.

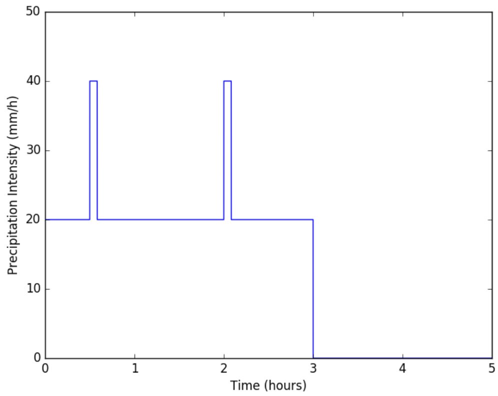

FIGURE 5.2: Example of an artificially generated rainfall event featuring precipitation intensity peaks of a short duration.

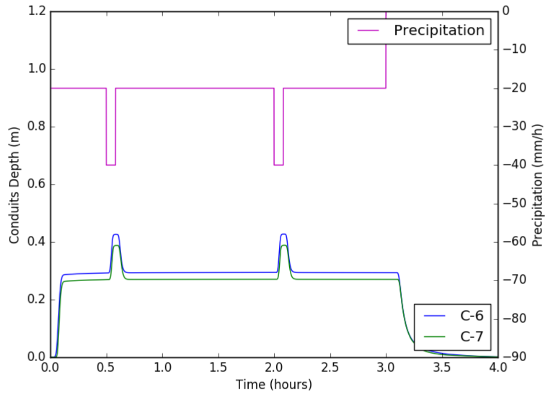

FIGURE 5.3: Water levels in downstream conduits (*C-6* and *C-7*) of the system when the previous artificially generated rainfall event is used.

In figure 5.2 two distinct precipitation peaks can be identified, sharing the same properties. The reason for their existence has to due with the intention to, in the same rainfall event (and therefore in the same simulation), register and compare the system's reaction when no control actions are being taken and when such measures are applied. This also justifies the fact that the two peaks are distant in time. Thus, in the performed experiments and tests all control actions are only triggered when the second precipitation peak is detected (at exactly 2 hours into the simulation).

In all the tests and experiments performed, different values for the intensity and duration of the peaks were considered. Initially, peaks lasting for 5, 10 and 15 minutes were considered during which the registered precipitation intensity remained unchanged (in the example presented in figure 5.2 a precipitation peak of 5 minutes can be observed, in which a $40\ mm/h$ precipitation intensity was measured).

Subsequently, an alternative scenario for the precipitation was also considered. In this scenario, during the registered peaks, measured intensity values is allowed to change. However, such variation will be far more controlled than what would be found in a real situation.

Instead of allowing for fast variations, where the intensity would remain constant for only a few seconds, precipitation intensity will have to remain constant for a minimum of 5 minutes. The reason for this is to allow the system to react to the registered variations in the precipitation intensity and for those actions to take effect.

In fact, from a point of view regarding the definition of rules for the controller, allowing the existence of very quick changes to the precipitation intensity values may not be beneficial, given that the actions taken by the controller do not have an immediate effect, which may hinder and prolong the process of defining and tuning its control rules. Faced with a real situation, it is necessary to process the recorded precipitation values in some way (for example, consider two values very close to one another as constant, but always taking into account the observed trend for precipitation - that is, if its intensity is increasing or decreasing, and for how long it has remained).

In both profiles considered for rainfall events a control system that enables a maximum water retention in upstream sections of the drainage system, without causing it to overload, will be sought.

In section 5.4 these experiments will be presented and discussed in more detail. Following this, in section 5.5 the control system developed for the drainage system shown in figure 5.1 will be detailed, along with its rules, obtained with the experience and knowledge acquired in all the performed experiments.

5.3 Work Overview

The current section is reserved to a brief overview of the different steps covered in the development of the controller. A variety of rainfall events, similar to the one presented in figure 5.2, were used in simulations of the hydraulic model presented in figure 5.1.

Prior to the development of the controller three major groups of experiments were conducted, which will be covered in the next section of this document. In these experiments a rapid rise in water levels in downstream conduits of the drainage system was sought. Different combinations of values and actuation periods on the installed barriers were then tested in order to attenuate these level rises as much as possible without forcing an overload situation in upstream sections of the system.

In all the experiments performed, the same approach was used when determining the actuation values for the barriers: they would be closed as far as possible, resulting in a reduction of their settings, in an attempt to retain maximum water volumes. Whenever, for a given barrier setting, upstream conduits became overloaded and an ascending flow of water was observed, the actuation value in the barrier immediately downstream of that outlet would be relaxed by $5\ \%$. That is, if for a setting of 0.2 an overload scenario was observed, then the same simulation would be executed with a setting of 0.25.

Regarding measured precipitation intensity different *baseline* values were considered, each with a corresponding water level in downstream conduits of the system. In the observed precipitation peaks intensity increases from $10\ mm/h$ up to $80\ mm/h$ were allowed. To avoid subjecting the system to excessive rainfall intensities an upper bound rounding $80\ mm/h$ was defined for all recorded precipitation intensity values.

In the performed experiments, water levels in the underground conduit *C-6* (immediately after the section with a steeper slope) were selected as the reference. Therefore, precipitation time series were generated whose base intensity produced a reference levels ranging from about *0.1 m* to about *0.4 m*, with a step of about *0.05 m*.

Regarding the actuation on the barriers, a closing speed of *1 m/min*, that is *0.01667 m/s*, was defined. Its opening speed was set to *0.0006 m/min*, that is, *0.00001 m/s*. The main motivation for this slow opening speed was due to the fact that, when the orifice is opened the volume of water that flows through it raises significantly, meaning that more water would reach downstream conduits which can promote significant level rises.

Earlier in this section three groups of experiments that supported the development of the controller were mentioned. In the remainder of this section these experiments are intended to be presented and briefly discussed.

In an initial phase, precipitation peaks with a constant intensity were defined. During this phase peaks with a duration of either 5, 10 or 15 minutes were considered.

According to what will be presented further in this chapter, in some cases it was found that actuation values capable of retaining water in upstream conduits during 5-minute peaks were unable to do so for longer peaks. In those situations the actuation on the barriers was relaxed until a proper retention (that is, with no overload of upstream sections of the system) was accomplished.

These situations where this adjustment was needed were the main feature in the second group of experiments conducted. At this stage it was known that, for example, when a rainfall event with a given baseline intensity and with precipitation peaks lasting for 5 minutes at a given intensity, a certain actuation in each barrier was sufficient to retain water in upstream sections. However, if these peaks lasted for 10 minutes, then a more relaxed (and known) actuation would have to be considered. The same was also verified between peaks lasting for 10 and 15 minutes.

In this second phase it was intended to determine if, for these longer peaks, performing a more relaxed actuation during all its duration produced better results than simply adjusting the actuation after the initial 5 or 10 minutes.

Therefore, at this stage, rainfall events which satisfy the mentioned conditions were used. In these experiments, after the initial 5 or 10 minutes the actuation on the barriers were adjusted, recording the evolution and maximum water levels in downstream conduits. Subsequently, this information was compared with simulations where the actuation on the barriers remained unchanged (and more relaxed) during the entire peak.

For the final group of experiments, rainfall events featuring precipitation intensity variations during registered peaks were explored. In this last stage only rainfall events with peaks lasting for 10 minutes were considered. To avoid excessively abrupt variations, which hinder the rules definition process, only one intensity variation per peak was allowed. This alteration, which can either be an increase or decrease of the measured precipitation intensity, took place 5 minutes after the peak's start.

Similarly to what has been adopted in the previous stages, precipitation intensity variations were kept at relatively low values, with increases and decreases not exceeding a limit of $10\,mm/h$.

Summarising what has been presented so far, in each of these groups of experiments rainfall events similar to the one presented in figure 5.2 were used. These events are characterised by featuring a constant *baseline* precipitation intensity for most of its duration. During a very short time window a peak in the measured precipitation intensity was observed, with a duration of either 5, 10 or 15 minutes. During this precipitation peak measured intensity values remain constant, except for some 10 minute-peak events, in which intensity values may increase or decrease after the initial 5 minutes. Finally, measured intensity during precipitation peaks will be a direct increase of the *baseline* intensity ranging from $10\ mm/h$ increases up to $80\ mm/h$, with a step of $10\ mm/h$.

Tables 5.1, 5.2 and 5.3 summarise and present the experiments covered in this chapter. It is worth pointing out that the number of experiments presented in the *TNSE* field only corresponds to tests in which the defined actuation values in the barriers did not lead to an infeasible scenario (that is, where upstream conduits became overloaded and an ascending water flow to the surface is registered). The total number of successful and unsuccessful tests performed, including the ones presented in appendix B, is far greater, exceeding a thousand experiments.

TABLE 5.1: Summary of the simulations executed in the first group of experiments.

Stage	Initial Base Level (m)	Final Base Level (m)	Level Step (m)	Peak Duration	Initial Peak Increase (mm/h)	Final Peak Increase (mm/h)	Intensity Step (mm/h)	TNSE[4]	ASD[5]
1	0.1	0.4	0.05	5 min	10	80	10	46	3 min
1	0.1	0.4	0.05	10 min	10	80	10	46	3 min
1	0.1	0.4	0.05	15 min	10	80	10	46	3 min

[4]Total Number of Successful Experiments.
[5]Average Simulation Duration.

TABLE 5.2: Summary of the simulations executed in the second group of experiments.

Stage	Initial Base Level (m)	Final Base Level (m)	Level Step (m)	Peak Intensity Transition	TNSE[4]	ASD[5]
2	0.1	0.4	0.05	5 min - 10 min	33	3 min
2	0.1	0.4	0.05	10 min - 15 min	19	3 min

TABLE 5.3: Summary of the simulations executed in the third group of experiments.

Stage	Initial Base Level (m)	Final Base Level (m)	Level Step (m)	Peak Intensity Durations	Intensity Variation During Peak	TNSE[4]	ASD[5]
3	0.1	0.4	0.05	5 min + 5 min	Increasing	23	3 min
3	0.1	0.4	0.05	5 min + 5 min	Decreasing	23	3 min

5.4 Experiments Prior to the Development of the Controller

In the current section the three groups of experiments preceding the actual development of the controller, but from which important ideas and information was extracted to be used in the controller, will be covered.

5.4.1 Constant Intensity With Constant Actuation During Peak

As mentioned in section 5.3, in an initial stage the performed simulations featured rainfall events with a constant baseline precipitation intensity, countered with the occurrence of two well-defined peaks in measured precipitation intensity.

At this point different pairs of actuation values for both barriers were tested, aiming to retain as much waster as possible in upstream sections of the drainage system. Initially, the defined actuations for the barriers were kept for some time after the precipitation peak had disappeared. This choice arose in an attempt further control the flow passing through the barriers after a more aggressive precipitation period. A series of tables summarising key results of these tests were included in appendix C.

Analysing the results of the performed experiments, it can be found that upon starting the actuation in the barriers a considerable level reduction in downstream conduits can, usually, be obtained. Furthermore, it is possible to attenuate water levels in downstream conduits for considerable extensive time periods. This is more evident when the system is subjected to less aggressive conditions, in which the intensity of the precipitation peaks and the reference levels in the conduits are lower, as presented in figure 5.4.

The reason for this has to due with the fact that, when more intense rainfall events are being used the actuation on the barriers needs to be relaxed, reducing their retention capacity, thus causing water levels in downstream conduits to be, on average, higher. However, even when more aggressive rainfall events are being considered, a peak in downstream conduits' water levels can still be identified, only being flatter and featuring a smaller maximum value than when no control actions are being taken. These ideas are supported by figure 5.5.

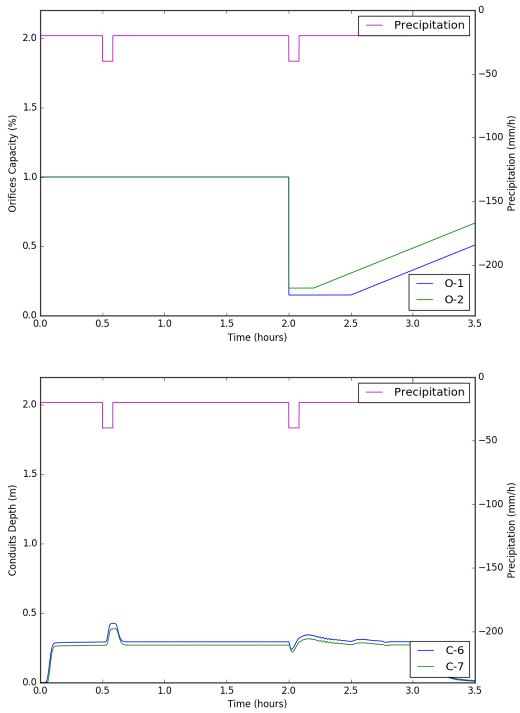

FIGURE 5.4: Actuation history in the barriers and resulting effects in downstream conduits (*C-6* and *C-7*) when the system is subjected to less aggressive conditions.

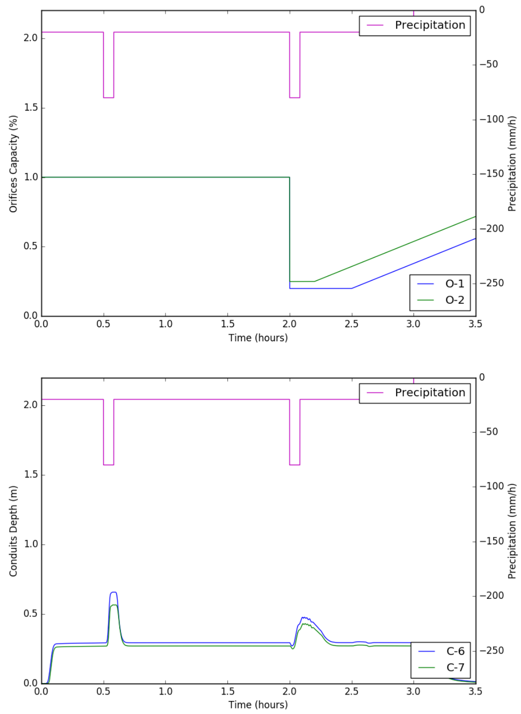

FIGURE 5.5: Actuation history in the barriers and resulting effects in downstream conduits (*C-6* and *C-7*) when the system is subjected to more aggressive conditions.

In fact, control strategies following a different approach and motivation can be considered and tested, namely those that aim to make use of the barriers in a more complementary way: for example, while a more rigid actuation on the upstream barrier is being carried out[6], a more relaxed control can be stimulated in the downstream one. However, when the upstream barrier cannot retain more water, it needs to be opened forcing the downstream barrier to be closed, to reduce the flow of water that passes through it, for as long as it is capable of avoiding an overload of its immediately upstream conduits.

Indeed, these alternative control strategies were somewhat tested in the presented system, but their results suggested the need for a high coordination between control actions performed in both barriers. At an early stage of the exploration of this scenario the initially presented approach showed better results, leading us to believe that it would still be valid choice.

As supported by the contents of appendix C, quite a considerable number of tests were performed at this stage. From these experiments a set of actuation values for the barriers were extracted, for each reference level considered, which enabled water retention in upstream sections of the system. Such values are presented in tables 5.4, 5.5 and 5.6. In these tables the notation a/b has been adopted to present the actuation values in the downstream and upstream barriers, respectively. Therefore, when presenting the values $0.1/0.15$ this corresponds to an actuation of 0.1 in orifice *O-1* (the downstream one) and an actuation of 0.15 in orifice *O-2* (the upstream one).

[6]Meaning that the setting of the barrier in question would be smaller, forcing it to retain more water.

TABLE 5.4: Orifices' actuation values for precipitation peaks lasting for 5 minutes, in all the considered reference levels.

Peak Increase	Level 0.1 m	Level 0.15 m	Level 0.2 m	Level 0.25 m	Level 0.3 m	Level 0.35 m	Level 0.4 m
10 mm/h	0.1/0.1	0.1/0.1	0.1/0.1	0.1/0.1	0.15/0.15	0.15/0.2	0.2/0.2
20 mm/h	0.1/0.1	0.1/0.1	0.1/0.1	0.1/0.1	0.15/0.15	0.2/0.2	0.2/0.2
30 mm/h	0.1/0.1	0.1/0.1	0.1/0.15	0.15/0.15	0.15/0.2	0.2/0.2	0.2/0.2
40 mm/h	0.1/0.15	0.1/0.15	0.15/0.2	0.15/0.2	0.15/0.2	0.2/0.25	0.2/0.25
50 mm/h	0.15/0.2	0.15/0.2	0.15/0.2	0.2/0.25	0.2/0.25	0.25/0.3	0.25/0.3
60 mm/h	0.15/0.2	0.15/0.2	0.2/0.25	0.2/0.25	0.2/0.25	-	-
70 mm/h	0.2/0.25	0.2/0.25	0.2/0.25	0.25/0.3	-	-	-
80 mm/h	0.2/0.25	0.2/0.25	-	-	-	-	-

TABLE 5.5: Orifices' actuation values for precipitation peaks lasting for 10 minutes, in all the considered reference levels.

Peak Increase	Level 0.1 m	Level 0.15 m	Level 0.2 m	Level 0.25 m	Level 0.3 m	Level 0.35 m	Level 0.4 m
10 mm/h	0.1/0.1	0.1/0.1	0.1/0.1	0.1/0.1	0.15/0.15	0.2/0.2	0.2/0.2
20 mm/h	0.1/0.1	0.1/0.1	0.1/0.15	0.15/0.15	0.15/0.2	0.2/0.2	0.2/0.2
30 mm/h	0.1/0.15	0.15/0.15	0.15/0.2	0.15/0.2	0.15/0.2	0.2/0.25	0.25/0.25
40 mm/h	0.15/0.2	0.15/0.2	0.2/0.2	0.2/0.2	0.2/0.25	0.25/0.25	0.25/0.3
50 mm/h	0.15/0.2	0.2/0.2	0.2/0.25	0.25/0.25	0.25/0.25	0.25/0.3	0.3/0.3
60 mm/h	0.2/0.25	0.2/0.25	0.25/0.25	0.25/0.3	0.25/0.3	-	-
70 mm/h	0.25/0.3	0.25/0.3	0.25/0.3	0.3/0.3	-	-	-
80 mm/h	0.25/0.3	0.3/0.3	-	-	-	-	-

TABLE 5.6: Orifices' actuation values for precipitation peaks lasting for 15 minutes, in all the considered reference levels.

Peak Increase	Level 0.1 m	Level 0.15 m	Level 0.2 m	Level 0.25 m	Level 0.3 m	Level 0.35 m	Level 0.4 m
10 mm/h	0.1/0.1	0.1/0.1	0.1/0.1	0.15/0.15	0.15/0.15	0.2/0.2	0.2/0.2
20 mm/h	0.1/0.1	0.1/0.15	0.15/0.15	0.15/0.15	0.15/0.2	0.2/0.2	0.2/0.25
30 mm/h	0.15/0.15	0.15/0.15	0.15/0.2	0.2/0.2	0.2/0.2	0.2/0.25	0.25/0.25
40 mm/h	0.2/0.2	0.15/0.2	0.2/0.2	0.2/0.25	0.2/0.25	0.25/0.3	0.25/0.3
50 mm/h	0.2/0.2	0.2/0.2	0.2/0.25	0.25/0.25	0.25/0.25	0.3/0.3	0.3/0.35
60 mm/h	0.2/0.25	0.25/0.25	0.25/0.25	0.25/0.3	0.3/0.3	-	-
70 mm/h	0.25/0.3	0.25/0.3	0.3/0.3	0.3/0.35	-	-	-
80 mm/h	0.3/0.3	0.3/0.35	-	-	-	-	-

In tables 5.4, 5.5 and 5.6 it can be seen that, in some scenarios, actuation values for each barrier are relaxed when the duration of the precipitation peaks increases. For example, this is case for a precipitation whose baseline intensity produces a level of $0.2\ m$ and with a peak increase of $20\ mm/h$: when the peak only lasts 5 minutes both orifices have an actuation of 0.1. However, when this peaks has twice the duration (lasting for 10 minutes) the actuation on orifice *O-2* needs to be relaxed to 0.15, while actuation in *O-1* remains unchanged.

Thus, a pertinent question to raise is how should one proceed with the adjustment of the actuation in the orifices as a given precipitation peak remains constant in time: it is sufficient to adjust the actuation values only after the first 5 minutes, or otherwise, should it be performed earlier, hoping to soften its impact on the drainage system? To answer this question further investigation and experiments were performed, to be detailed in the subsection that follows.

5.4.2 Constant Intensity With Variable Actuation During Peak

Seeking to answer the question raised at the end of the previous subsection, further investigation and tests were conducted. In this sense, rainfall events that required different actuations to handle either 5 and 10-minute peaks or 10 and 15-minute peaks, during which measured precipitation intensity remained unchanged, were selected. Thus, for rainfall events considered at this stage, the total duration of the observed peaks was either 10 or 15 minutes.

In the performed tests, the actuation in both orifices was adjusted at different moments in time. For 10 minutes peaks, adjustments performed after 4, 4.5 and 5 minutes were made. For 15 minutes peaks, these adjustments were performed after 9.5 and 10 minutes. More detailed information concerning these experiments can be found in the tables presented in appendix D.

Analysing the data presented in such tables, one can verify that adjusting the actuation in the orifices slightly before the critical transition moments (10 and 15 minutes) does not offer any benefit regarding the maximum water level measured in downstream conduits. Even analysing the evolution of the depth in the system's underground conduits, making this adjustment earlier in time does not offer any kind of significant improvement.

5.4.3 Variable Intensity During Peak

The third and final set of experiments to be covered in this section consists, as already mentioned, of rainfall events with peaks of 10 minutes in which the measured precipitation intensity varies after 5 minutes of the peak. This variation can correspond to either an increase or decrease in the measured intensity.

In the performed experiments smoother intensity variations (of about *10 mm/h*) were chosen over more abrupt ones. However, in these experiments, only pairs of intensity values for which different actuations were defined in subsection 5.4.1 were used. In figures 5.6 and 5.7 two examples of rainfall events used at this stage are presented. In figure 5.6 an intensity increase is witnessed during the peak, while in figure 5.7 a decrease is observed.

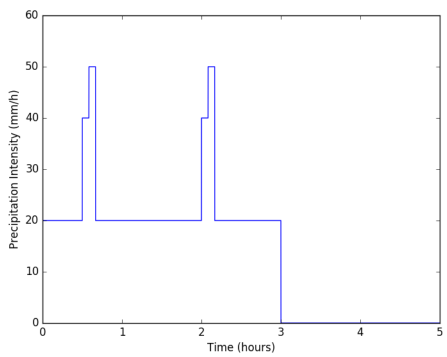

FIGURE 5.6: Example of rainfall event with an intensity increase during a peak.

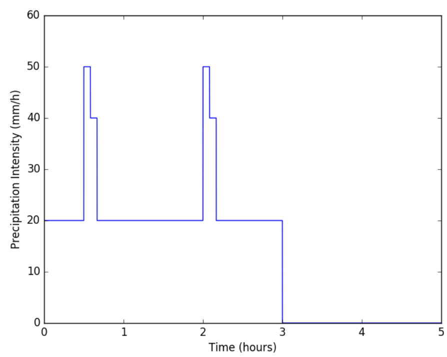

FIGURE 5.7: Example of rainfall event with an intensity decrease during a peak.

At this point two different approaches were compared when reacting to the observed variations: in the first one, actuation on the barriers would be performed according to the rules defined in subsection 5.4.1. In the opposing approach, actuation values defined for the period of higher intensity were kept during the 10 minutes of the peak.

An important objective of this comparison was to complement the discussion of the previous subsection, seeking to understand to what extent adjusting actuation values on the barriers in response to intensity variations during a peak allows the retention of a more considerable amount of water, or not.

In appendix E more detailed results of the tests performed at this point can be found.

Analysing the provided tables in appendix E and their respective graphical representations of the evolution of water levels and flow in the conduits of the system, neither of the two mentioned approaches seems to overlap the other in both scenarios (with an increase and decrease in the intensity during precipitation peaks).

Indeed, it was found that, in general, when the rainfall intensity increased performing a more relaxed actuation during the entire peak (corresponding to the highest measured intensity value during the peak) allowed for slight higher retentions in upstream conduits, resulting in reduced water levels in downstream conduits. However, when the rainfall intensity decreased adjusting the actuation produced better results.

These conclusions are supported by the following ideas: when facing a scenario in which, during a peak, the measured precipitation intensity increases, if a more relaxed actuation is carried out during the 10 minutes, in the first 5 a smaller amount of water will be stored in upstream conduits (when comparing to what should be used, based on the developed rules). However because a smaller amount of water was stored in upstream conduits, when the precipitation intensity increases, the available volume to store excess water in these sections is higher, meaning that they will be able to store a considerable volume of water in the last 5 minutes, contributing to a more significant reduction in water levels in downstream sections of the system.

On the other hand, when the precipitation intensity decreases during a peak, if a more relaxed actuation on the barriers is kept then more water will flow through them, meaning that a larger volume of water reaches downstream sections. This happens because when precipitation intensity decreases water levels in the system's conduits (namely in upstream ones) also tend to decrease. This means that actuation on the barriers can be tighter because its available volume will tend to increase, therefore upstream conduits can store more water.

Although it was found that, when precipitation intensity increases during peaks, performing a more relaxed actuation in existing barriers leads to smaller levels in downstream conduits, adopting this approach in a real situation raises some problems, especially since there is no way of knowing with absolute certainty future precipitation values[7]. Because of this, when faced with a peak, there is no way of knowing if, given the current precipitation intensity, it will remain constant, increase or decrease, and for how long will it behave like that.

Nevertheless, even though a more relaxed actuation may produce better results in some scenarios, adjusting the actuation on the barriers as intensity and duration variations are registered does not produce results that inferior. Thus, with all these facts in mind, it was decided that, in the developed controller, actuation values on the barriers will be adjusted based on measured duration and intensity values of the rainfall event.

5.5 Developed Controller

In previous sections of the current chapter it was mentioned that a series of experiments were performed from which important knowledge was obtained, useful in the development of the controller. By testing rainfall events of the same nature (even though clear differences in their profiles

[7]It can be argued that the controller could, at this point, make use of rainfall predictions. However since, in the course of this work, no access to such data was obtained, this path was not properly explored, and therefore it was not included in this document.

can be identified) a series of actuation values that enabled water retentions in the presence of a given set circumstances were determined.

As it was also mentioned in previous sections, these actuation values were obtained with the intention of being directly used in the controller under development. In fact, these values were used to define the actions to be taken on the barriers installed on the system, even when it was subjected to rainfall events with different characteristics.

Therefore, it is natural to consider rainfalls with other characteristics, distinct from those used so far in an attempt to obtain more generic rules to guide the behaviour of the controller. Although performing more tests with different rainfall events can contribute to a greater generalization of the resulting rules, it is important to consider the following ideas:

On one hand, the number of experiments needed to be performed and their respective timing requirements. In fact, if one starts to extend the list of scenarios to be explored the time required to complete all tests grows sharply, making the completion of the work infeasible.

On another hand, it is important to remember that as the amount of water transported by upstream conduits increases (and by extension their level), the amount of water in them that can be retained with the barriers and the time period during which they may be activated is significantly reduced. Therefore, by considering more aggressive rainfall scenarios, in which water levels in upstream conduits would be higher, more relaxed actuations would have to be carried out in the barriers, which can considerably reduce resulting water level reductions in downstream sections of the system.

Alternatively, it would also be possible to work with actuation values previously documented which, in the presence of more aggressive rainfall events, would lead to an overload of upstream conduits. When this scenario is observed preserving water retention in upstream conduits would no longer be beneficial in any way, meaning that it should be aborted by the control system.

This was the solution chosen to be adopted and implemented in this work, as it allowed the system to attempt to handle all sorts of precipitation events as best as it could, without requiring extra experiments and rules to be developed. Thus, accurate identification of these situations is of major importance at this stage.

With these ideas in mind a decision was made to implement the controller in different modules, each with distinct functions and hierarchical positions regarding its importance and power with respect to the remaining components. Thus, the controller proposed and developed in this work is composed of the following modules:

- A *Supervisor* module, responsible for supervising water levels and flows in all underground structures of the drainage system, with a special focus in the identification of overload states in upstream sections of the system. The supervisor is also responsible for enabling and disabling the *Actuation* module and can order certain actuations to be performed on the installed barriers.

- An *Actuation* module which contains the developed control rules presented in tables 5.4, 5.5 and 5.6. These rules govern the actuation on the barriers, based on measured base precipitation intensities and water levels in downstream conduits and in current precipitation intensity values.

- An *Application* module, where all actuation values to be performed on the installed barriers are processed. This module is composed of part of the *MatSWMM* library, which implements the necessary mechanisms to adjust the settings in the orifices included in the processed hydraulic models.

In figure 5.8 a small diagram which briefly summarises the interactions between these modules is presented.

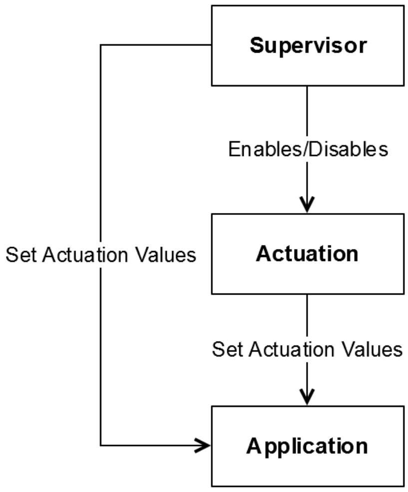

FIGURE 5.8: Modules which compose the controller and their respective interactions.

With the addition of this *Supervisor* module, instead of simply applying the rules obtained from previous experiments the current state of the system is continuously being monitored and undesired overload scenarios can be identified as early as possible. Thus, precipitation events to which the system has not been properly prepared are not being disregarded or ignored but, on the contrary, are being properly identified and dealt with, by adopting immediate measures to mitigate them.

In the current work, whenever the developed rules proved insufficient to properly retain all water inflows in upstream conduits all excess water in those structures is re-routed outside of the conduits, into a water retention tank.

From the three modules that compose the controller, the *Supervisor* constitutes the most important one, but also the one with the more complex description and implementation. For this reason, a more detailed coverage of this module will be provided in subsection 5.5.1. Following that, a brief description of how the communication between the different modules is assured will also be addressed, in subsection 5.5.2.

5.5.1 Supervisor

In order to properly implement the *Supervisor* this entity was modelled with a state machine. This way it is possible to define a series of states in which the *Supervisor* can be at any given time during the simulation, each with very clear and distinct actions and transition rules.

In figure 5.9 the state machine that describes the idealized behaviour for *Supervisor* module is presented. Table 5.7 contains the description of the events contained in the state machine.

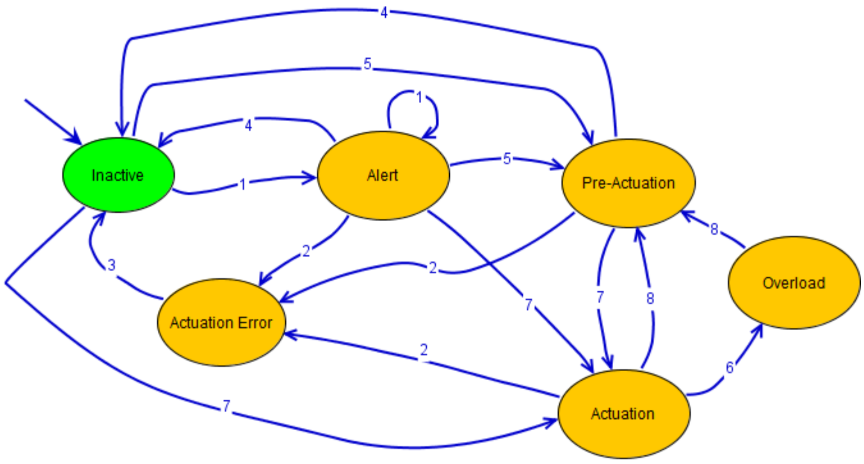

FIGURE 5.9: State machine describing the *Supervisor's* behaviour.

In figure 5.9 the following states were defined for the *Supervisor*:

- An *Inactive* state, which is set during a normal scenario when small volumes of water (or none at all) are flowing through the system and the current precipitation state and predictions are of low risk. When this state is set the Supervisor is in a passive state, simply monitoring the system without performing any actions on the barriers.

- An *Alert* state, which is set upon the arrival of a prediction of medium or high risk of intense precipitation. When this state is set the Supervisor performs a simple test on the barriers, to verify that they are working properly. In this test the Supervisor simply changes the setting of both barriers to 0.5, keeping it at that level for two straight minutes, moving it up again to a full opening.

- During the barrier tests performed in the Alert state, if any problem regarding actuation on the barrier is identified (for example, even though the Supervisor gave the instruction to set the setting of the barrier to 0.5 the system failed to do so) then the Supervisor stops all actuation and monitoring task and enters in the *Actuation Error* state, issuing a warning to the entity responsible for carrying out maintenance of the structures in the system, indicating the problem identified so that it can be resolved as soon as possible. Only when this problem is successfully repaired and duly certified by that entity the supervisor resumes its normal operation.

- Whenever measured precipitation intensity raises, as a consequence of its predicted intensity (or not), the Supervisor enters a *Pre-Actuation* state. The main goal of this state is to reduce the time needed to respond to a more intense rainfall event. Thus, in this sense, when the Supervisor enters in this state the setting in both barriers is set to 0.5. If base water levels in upstream conduits of the system are considerably higher then this value then, to avoid start significant retentions at this stage, the target setting for the barriers may be relaxed to values higher than the current water base levels.

- Whenever an increase in the measured precipitation intensity is registered, a similar behaviour tends to be observed in upstream conduits of the drainage system, with a brief time delay. When water levels in upstream conduits raise more significantly, the Supervisor must start implementing measures to retain water in these conduits, in order to prevent it to reach its downstream sections. At this point the Supervisor enters in the *Actuation* state, in which the previously defined rules that govern the actuation performed in the barriers are applied.

When the precipitation ceases and current water inflows in the system's underground conduits are small enough the Supervisor can return to its Pre-Actuation state, as the current precipitation peaks has passed.

- If, during the Actuation state, the application of the previously defined rules leads to an overload scenario in upstream conduits of the drainage system then the Supervisor shifts to the *Overload* state. In this state actuation on orifices *O-1* and *O-2* ceases, resulting in their full opening. Additional water that may enter in the system will be re-routed to an existing water retention tank, outside of the drainage system.

Regardless of its current state, the *Supervisor* will be responsible for, in every iteration of the simulation, collecting and processing series of information necessary to its normal operation. Such information consists of the current state of the system (including current and base water levels and flows in each structure of the drainage system, among other relevant information) and of daily precipitation predictions that classify the current risk of intense rainfall events in one of three states: high risk, moderate or low.

TABLE 5.7: Description of the events defined for the *Supervisor's* state machine.

Event	Description
1	Collected daily precipitation prediction with moderate or high risk of intense rainfall events
2	Failure during actuation in one barrier (Identifies problem in the system)
3	Previously identified problem in the system successfully fixed (Triggered by entity responsible for the maintenance of the system)
4	Collected daily precipitation prediction with low risk of intense rainfall events
5	Current Precipitation Intensity >= 40 mm/h OR Current Precipitation Variation >= 10mm/h
6	System Overload during execution of Actuation Module (Water Flow in an ascending direction, in at least one outlet flow)
7	Current Precipitation Intensity >0 AND (Upstream conduits water level >= 10% base level for conduit OR Water inflow in J-1 increases by at least 10%)
8	Current Precipitation Intensity = 0 OR Water inflow in J-1 <= 0.1 m3/s

Even though figure 5.9 describes the behaviour idealized for the Supervisor, in the performed experiments a slight different version of this module was implemented. Because only a model of a real drainage system is being used in this work, problems such as errors in the execution of instructions sent to the barriers will not occur in the performed simulations. Furthermore, daily predictions regarding the current risk of intense rainfall events are also not considered, simply because no access to such data was granted.

Even though these events were not included in the performed simulations they were still mentioned in the initial definition of the controller, considering that they complete its definition and, in a real application, such situations should be accounted for.

With this in mind, the implemented Supervisor only features four states: *Pre-Actuation*, *Actuation*, *Overload* and a *Normal* state, that encompasses the *Inactive*, *Alert* and *Actuation Error* states.

All transitions between the states presented in figure 5.9 are maintained, with the exceptions of all events related with either rainfall predictions or errors in the actuation of the barriers.

5.5.2 Communication between the Modules

Considering that all the key components of the controller are implemented in the same programming language (Python) the communication between the different modules can be established by simple function or method invocations.

In the performed simulations all operations start from a main class, composing the application's starting point. In this class the function *swmm.initialize()*[8] is called to load and validate a file containing the hydraulic model to be used in the simulation.

When this operation finished successfully, both *Supervisor* and *Actuation* classes are properly instantiated. It is worth mentioning, at this point, that the *Actuation* class is not instantiated in the main class, but as a private attribute of the *Supervisor*.

The simulation is then started and executed step-by-step, by iteratively calling the functions *swmm.run_step()*[8] and *swmm.is_over()*[8].

During each iteration of the simulation relevant information regarding the state of the system (water levels and flows in existing junctions and conduits, current precipitation intensities, among other relevant information) is collected using the *swmm.get()*[8] function. Furthermore, in each of these iterations the current state of the *Supervisor* is updated, allowing it to trigger any action defined for its current state. This is accomplished by a simple invocation of a public method implemented in the *Supervisor* class.

If, after updating its state, the *Supervisor* needs to delegate actuation responsibilities to the *Actuation* module it will do so by performing a simple invocation of a public method implemented in that class, which will process the collected information describing the current state of the system and determine the actuation values for the two barriers. To actually perform the actuation in the barriers the method *swmm.modify_orifice_setting()*[8] needs to be called.

Alternatively, if the current state of the *Supervisor* may require it to perform some sort of previously defined actuation on the barriers (as, for example, if it is in the *Pre-Actuation* state). In that scenario the *Supervisor* will simply call the respective method implemented in the *MatSWMM* library to perform the desired actuation.

5.6 Remarks

In the current chapter the experimental setup adopted throughout this work was presented, along with a series of experiments carried out on it, which resulted in the development of a control system. The main goal of this control system is the continuous monitoring of a given drainage system's state, and consequent *reaction* to the occurrence of more aggressive precipitation events. This reaction consists in the execution of a series of actions that aim to retain water in upstream sections of the drainage system, by manipulation of a series of barriers installed in those locations.

In the scope of the collaboration established with a research group at *DEC*, as mentioned in section 1.5, both the hydraulic model presented in the beginning of this chapter and the actions performed on the barriers were properly validated, as well as all monitoring and verification routines included in the control system. Therefore, as a result of this validation, it is guaranteed that a valid scenario is being considered in this work which meets all the legal requirements imposed and that all the actions and decisions taken by the control system are also legally valid.

At this point, it is also important to emphasize once again the fact that this control system was designed to be applied in a set of drainage systems where possible overloads (and consequent surface flooding events) are triggered only in their downstream sections. Indeed, as evidenced

[8]Implemented in the *MatSWMM* library.

in some situations reported in appendix B, the approach followed in this work will not produce the desired results if, instead, upstream sections of the drainage system become overloaded and originate these flood phenomena.

Furthermore, in the development of the presented control system and hydraulic model some important points need to be emphasized:

First of all, with respect to the layout of the barriers, these should not be placed too closed to each other. Indeed, two barriers installed in underground conduits must be separated in depth by a distance at least equal to their height. The reason for this has to due with the fact that, when a barrier is used to retain water in a conduit (or in a series of conduits) upstream of its location it has an impact in the conduits, starting from its immediately upstream one up until the last conduit placed at a depth equal to the difference between the barrier's depth and height. Thus, if a barrier with a height of *1 m* is placed at a depth of *20 m* (for example), then another barrier should only be placed in a conduit with a depth of, at least, *21 m*.

The general behaviour assumed for the barrier also has a strong impact in the development of the control system and on its idealized behaviour. Considering that current legislation prevents water flow in underground sections of drainage systems to be null, in this study vertical barriers were always considered, moving from the conduit's top to its bottom. Nevertheless, barriers with alternative behaviours could also be idealized, however they would require the development of a different control system.

The profile of the rainfall events considered in these experiments is also worth mentioning. Indeed, in the tests performed rainfall events with a constant low precipitation intensity, momentarily contradicted with shorts periods of very intense precipitation, were adopted. As previously mentioned in this chapter, this kind of events is very characteristic of a popular phenomena known as *"flash floods"*. Furthermore, as proved by the experiments performed, the approach adopted in the development of the control system allows for more significant flow and level reductions in downstream sections of the drainage system when precipitation peaks are shorter (i.e., with a smaller duration). In fact, this result comes as no surprise, as longer precipitation peaks require the retention of higher water volumes in order to produce significant level and flow reductions in downstream sections. When this cannot be accomplished, actuation on the barriers needs to be relaxed, meaning that smaller reductions will be accomplished.

Another point worth highlighting at this stage, regarding the development of the control system, has to due with sensory material needed for its implementation in a real environment. As can be understood from an analysis of the behaviour and constitution of this control system, it is highly dependent on a series of information regarding the infrastructures of the drainage system being monitored, collected by a series of sensors:

- *Hydraulic Flow* and *Level Meters*, installed in each underground conduit and junction of the drainage system. These equipments are responsible for collecting, respectively, information regarding water flows and levels in the structures where they are installed.

- *Precipitation Intensity Predictions*, provided on a daily basis to the control system and built by certified entities (such as, for example, *IPMA - Instituto Português do Mar e da Atmosfera*[9]), in the already specified format: low, moderate or high risk of intense rainfall events.

- *Rain Gauges*, usually installed on very tall locations, are responsible for measuring current precipitation intensity values in a given location. If they are installed near the location of the drainage system that is being monitored then information regarding current precipitation intensities at surface levels on the drainage system, with a certain degree of accuracy, can be obtained.

[9] `https://www.ipma.pt/en/`

In a final note, the behaviour of the developed control system can be summarised to the following ideas: it is mainly composed of two major operation levels, described in the listing that follows:

- On a higher level, a *Supervisor* entity defines all actions performed by the control system. Its main responsibilities are related with the detection and prevention of abnormal situations which may present, in any way, a risk to the normal functioning of the drainage system (for example a sudden increase in precipitation intensity values). In many scenarios, when such events are identified they have not yet been observed in the drainage system:

  For example, when a sudden precipitation peak is registered there is a certain time delay from the moment it is measured in the rain gauges to the instant when a rise in water inflows in the drainage system is registered. During this period of time, since a rise in precipitation intensity has been registered (which will result in a rise in water levels inside underground conduits) there is a chance that actuation on the barriers will be required. In this sense, from the moment that a peak in precipitation intensity values is registered, the control system will actuate over the barriers in order to start lowering them, without actually starting any water retention in upstream conduits.

  This way, when a rise in water inflow in the drainage system is observed, the required time to lower the barriers to a point where they actually start retaining water is considerably reduced.

- In a lower level, an *Actuation* module defines the actions performed by the control system. This module can issue different control operations to be performed on the barriers that depend on previous indications issued by the *Supervisor*, the current state of the drainage system and a series of previously defined rules.

Chapter 6

Experiments

After the definition and implementation of the controller a series of tests were performed. The primary goal of these experiments was to test the developed system, finding and correcting possible errors in its conception and implementation which might have gone unnoticed in debugging tasks performed during its development.

Therefore, in an initial stage, the scenarios presented and tested in section 5.4.1 were used. Next, scenarios featuring precipitation intensity changes during a precipitation peak, which had also been tested in section 5.4.3, were also subjected to experimentation. In section 6.1 a more detailed presentation and discussion of these experiments and their results can be found.

With this initial series of tests it was intended to verify that the supervisor was behaving has expected. In fact, since all these scenarios had already been used to generate some of the controller's rules, its expected behaviour was already known. Therefore, it was possible to confirm that, in fact, the implementation of the controller was performed according to what was intended and planned, as will be shown further in this chapter.

Finally, two distinct and new sets of scenarios were also introduced and tested with the developed controller, covered in sections 6.2 and 6.3:

In the first one a constant incoming water flow was added to the system, seeking to force downstream conduits to become full, forcing water out of the system through its outlets and on to the surface. By executing the developed supervisor under these circumstances it was expected that the observed ascending flows were removed, or at least considerably attenuated, by performing water retentions in upstream conduits. Naturally, at this point, the *Overload* state of controller's supervisor component was disabled, thus making it possible to determine to what extent can the developed controller remove water ascending flows through the system's outlets.

In order to properly simulate this scenario, a constant inflow with a given value was added to junction *J-7* (immediately downstream of orifice *O-1* and right before the steepest slope). This was accomplished using a specific functionality available in the *SWMM* software, meaning that, at this point, no visual changes were made to the hydraulic model presented in figure 5.1.

In the second set of scenarios an overload situation was promoted in upstream conduits, seeking to force the system to correctly identify such state, stopping all developed control measures and forcing water in its initial sections to be removed from the drainage system.

In order to properly allow water to exit the drainage system a small addition to the scenario presented in figure 5.1 was made. After its first section a conduit connecting junction *J-2* to an outfall node was added, controlled by a third orifice (orifice *O-3*). This orifice will only be opened when the Supervisor is in the *Overload* state, being fully closed in all remaining moments.

Figure 6.1 shows the hydraulic model with the mentioned changes.

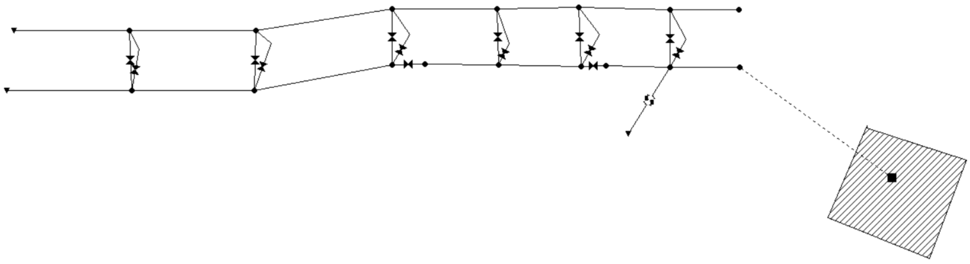

FIGURE 6.1: Scheme of the modified hydraulic model.

6.1 Comparison with Previous Scenarios

The current section is reserved to a brief discussion and presentation of the results obtained when simulating the developed controller with the scenarios presented in sections 5.4.1 and 5.4.3.

For the sake of simplicity, and to avoid too long references, scenarios presented in section 5.4.1 will be mentioned as *"Constant Intensity"*, while scenarios presented in section 5.4.3 will be mentioned as *"Variable Intensity"*. Tests performed with the controller for each set of scenarios will be covered in their own subsection, starting with the ones for *Constant Intensity*.

6.1.1 Constant Intensity

Similarly to the experiments performed in section 5.4.1, table 5.1 summarises the tests executed at this point. Following a similar approach, constant rainfall events where generated in order to produce different base water levels in conduit *C-6*. As in previous experiments, base water levels ranging from *0.1 m* up to *0.4 m* were considered, with a step of *0.05 m*.

In these rainfall events two precipitation intensity peaks were also included, which would lead to a water level rise, to be mitigated by the developed controller. For each base water level, and therefore for each type of rainfall event, different intensity values were considered during precipitation peaks. Rainfall intensity increases ranging from *10 mm/h* up to *80 mm/h*, compared to base precipitation intensities, were considered. Again, precipitation intensity values significantly higher than *80 mm/h* were not allowed.

In appendix F.1 a series of tables summarising key results of these tests were included, similarly to what was already performed with previous tests. In these tables the obtained results are also compared with those of the experiments performed in section 5.4.1.

As expected for these tests, the results obtained with the developed controller are very similar to the ones already obtained in previous experiments. In all the performed simulations actuation on barriers with previously defined values and determined by the controller are identical.

Nevertheless, when making use of the controller it was possible to, in some cases, reduce the duration of the observed peak in water levels in downstream conduits. Even though in some of these situations maximum water levels in downstream conduits were higher when the controller was being used, being able to reduce the duration of level peaks in downstream conduits seems a very promising result.

As an example, figure 6.2 supports this idea.

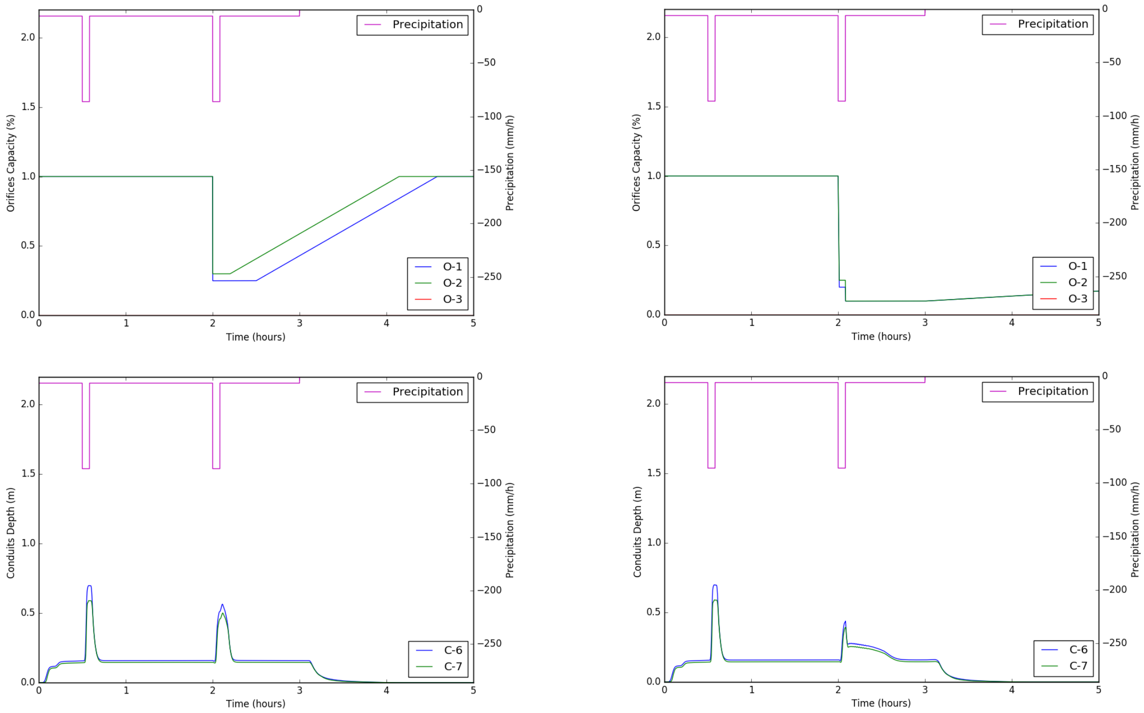

FIGURE 6.2: Example of a simulation in which the developed control system was able to reduce the duration of the observed peak in water levels in downstream conduits (*C-6* and *C-7*): In the leftmost plots it is possible to observe actuation history and water levels obtained in previous experiments, while the rightmost plots contain actuation history and water levels obtained with the developed control system.

In simulations featuring rainfall events with peaks lasting for 5 minutes, only for a base level of *0.1 m* and smaller intensity increases did the controller failed to further reduce maximum registered water levels in downstream conduits (when compared to experiments with hard-coded rules). In fact, this was only observed for intensity increases of 10, 20 and 30 mm/h.

When considering longer peaks (lasting for 10 or 15 minutes) there seems to be no pattern or rule that determines which approach provides better results.

In figure 6.2 it can be seen that the actuation carried out by the developed controller does not achieve a full orifice open state by the end of the simulation. In fact, one could consider this a sign of a problem in the conception of this system or in its implementation. However, that is not the case. As stated in section 5.3, a considerably slow orifice opening speed was defined in order to avoid significant increases in water flows through the orifices.

Because of this slow orifice opening speed, orifices *O-1* and *O-2* are still in their opening process when the simulation finishes. Furthermore, by taking advantage of the fact that these orifices only perform water retentions when their setting is lower than water levels, promoting slow orifices' opening speeds allows for a faster response in the scenario of a potential recurrent precipitation event.

6.1.2 Variable Intensity

As in the previous subsection, table 5.3 summarises the tests executed at this point. In these tests rainfall events similar to the ones already tested were used, with the difference that only precipitation peaks of 10 minutes were used, and measured intensity values during those peaks were changed (either increased or decreased by 10 mm/h) after the initial 5 minutes of the peak.

In appendix F.2 a series of tables summarising key results of these tests were included, similarly to what was already performed with previous tests. In these tables the obtained results are also compared with those of the experiments performed in section 5.4.3.

Considering the results obtained in the tests performed in 5.4.3, it was expected that the developed controller presented slightly inferior results in situations featuring an intensity increase during precipitation peaks, when compared to the best approach tested in 5.4.3.

However, analysing the results obtained at this point and comparing them with the experiments performed in section 5.4.3, none of the two approaches being compared seems to produce better results in a consistent way, when being compared to the other.

For some water base levels (and precipitation intensities during peaks) the developed controller produces better results, being able to perform more significant reductions of water levels in downstream conduits. But in other scenarios this approach simply fails in improving on what was already accomplished.

Nevertheless, regardless of which approach produces better results, with both of them very similar maximum levels in downstream conduits are always achieved. The main difference between them, favouring the approach using the developed controller, has to due with the duration of water level peaks, which as already mentioned is smaller with the controller. Generally, this tends to occur with higher water base levels in conduit *C-6* (rounding *0.3 m*). Again, as mentioned, this seems like a very promising result.

6.2 Overload in Downstream Conduits

In the current section the first of the final two sets of scenarios to be tested will be covered. As already mentioned in this chapter, in the scenarios presented in this section a constant incoming water flow was added to junction *J-7*, raising water levels in downstream conduits of the drainage system.

With this incoming flow an overload scenario in these conduits is sought to be obtained in such a way that an ascending flow of water through outlets *L-9* and/or *L-11* (located in the last two sections of the drainage system) is observed. The goal of these experiments is, therefore, to determine the maximum incoming flow for which the developed controller is able to successfully remove this overload scenario.

As in previous experiments conducted so far, the hydraulic model presented in figure 6.1 was used. Regarding the selected rainfall events, the same precipitation data sources used in the experiments discussed in sections 5.4.1 and 5.4.3 were selected: All considered precipitations events were composed of a long period with a constant intensity and two relatively short peaks in which measured precipitation intensity increased significantly, either remaining constant or varying during the peak.

Again, different rainfall events leading to distinct water base levels in downstream conduits were considered. For each water base level only rainfall events used in 5.4.1 and 5.4.3 with higher intensities during peaks were considered. The reason and motivation for this choice has to due with the fact that, for each base level, these are the most aggressive precipitation events collected. Therefore, if the controller is able to remove or mitigate the effects of this overload scenario with a more aggressive rainfall, then it will also be able to accomplish this in a less aggressive environment.

Similarly to the approach adopted in the previous section, the experiments covered at this stage were divided in two distinct groups: Those in which precipitation intensity remained constant during peaks, also referred to as *"Constant Intensity"* (section 6.2.1); and those in which precipitation intensity was allowed some variation during peaks, also referred to as *"Variable Intensity"* (section 6.2.2).

In an initial approach, a constant incoming flow of $1\ m^3/s$, similar to flow values registered in the drainage system's conduits during precipitation peaks in previous experiments, without any control operation being performed in the barriers was tested. Since, in the performed experiments, no overload scenario in downstream conduits was observed, the selected incoming flow value was increased until such situation was observed.

Whenever, for a given incoming flow that lead to an overload scenario, the developed controller managed to properly retain water in upstream conduits, removing the observed ascending flows, the test in question was repeated but for a higher constant incoming flow value (increased by $0.1\ m^3/s$), until the controller proved unable to mitigate this situation.

Table 6.1, presented below, contains a summary of the experiments performed at this stage. More detailed information regarding these tests can be found in appendices G.1 and G.2.

TABLE 6.1: Summary of the experiments performed with the developed controller at this stage.

Initial Base Level (m)	Final Base Level (m)	Base Level Step (m)	Peak Duration	Peak Variation	Incoming Flow Start (m3/s)	Incoming Flow End (m3/s)	Incoming Flow Step (m3/s)
0.1	0.4	0.05	5 min	Constant	1.0	1.5	0.1
0.1	0.4	0.05	10 min	Constant	1.0	1.5	0.1
0.1	0.4	0.05	15 min	Constant	1.0	1.5	0.1
0.1	0.4	0.05	10 min	Increase	1.0	1.5	0.1
0.1	0.4	0.05	10 min	Decrease	1.0	1.5	0.1

6.2.1 Constant Intensity

Simulations in which precipitation intensity remained constant during registered peaks will be covered in this subsection. As supported by table 6.1, three different precipitation peak durations were considered at this point: 5, 10 and 15 minutes, covered in sections 6.2.1.1, 6.2.1.2 and 6.2.1.3, respectively.

6.2.1.1 5-minute Peaks

In the overwhelming majority of the experiments performed with the smallest duration allowed for precipitation peaks significant reductions of maximum flow values registered in downstream conduits of the drainage system were obtained.

When no control operations were being adopted measured water flows in downstream conduits were between $0.9\ m^3/s$ and $1.0\ m^3/s$. However, when the developed controller was being applied these measured values decreased, prowling $0.7\ m^3/s$ (Resulting in a flow reduction of about 78 %). Occasionally, for water base levels of *0.1 m* and *0.3 m*, it was possible to further decrease these measurements, to about $0.5 m^3/s$ (totalling a flow reduction of about 50 %).

For constant water inflows of $1.1\ m^3/s$ and $1.2\ m^3/s$ the developed controller was able to remove the observed ascending flows in outlet *L-9*, for all base levels considered. Only for constant inflows higher of equal to $1.3\ m^3/s$ did these situations were again observed, even though in a much more smaller scale. As more intense incoming flow were considered the registered ascending flow in outlet *L-9* increased not only in intensity but also in duration.

The plots in figures 6.3 and 6.4 support these ideas, presenting an example of both scenarios for a base level of *0.4 m*.

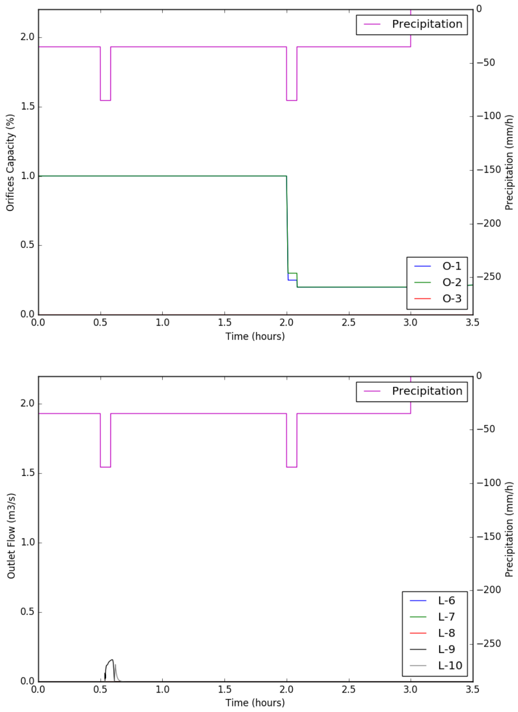

FIGURE 6.3: Actuation history in the barriers and resulting effects in water flow in downstream outlets (*L-6* to *L-10*) for a constant inflow of 1.2 $m^{3/s}$, considering a base level of *0.4 m*.

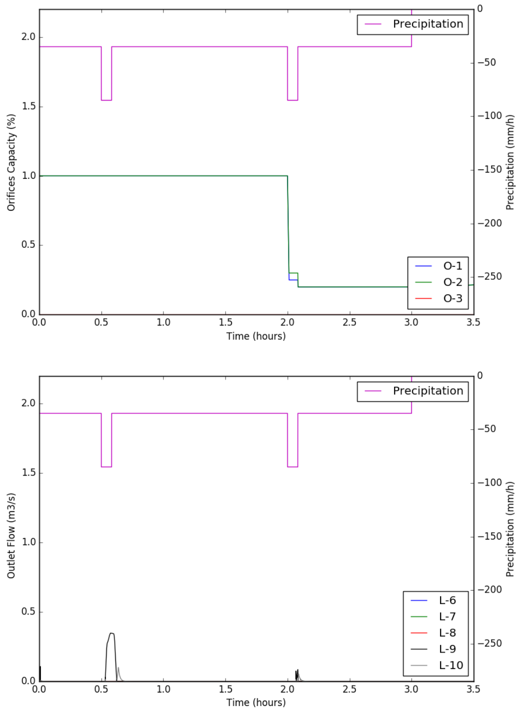

FIGURE 6.4: Actuation history in the barriers and resulting effects in water flow in downstream outlets (*L-6* to *L-10*) for a constant inflow of 1.4 $m^{3/s}$, considering a base level of *0.4 m*.

Because a continuous incoming flow is being considered throughout the entire simulation, even when precipitation ceases, considerable high water levels are measured in downstream conduits, forcing the control system to remain in its *Actuation* state.

Nevertheless, even for constant inflows of $1.3\ m^3/s$ and $1.4\ m^3/s$ the controller was able to perform significant water retentions in upstream conduits, freeing up volume in downstream sections to store most of the additional water volumes reaching the system, therefore resulting in a considerable attenuation of the water flow that passed through outlet *L-9*, and by extension, water stored at a surface level as presented in figure 6.5.

However, when the inflow was raised to $1.5\ m^3/s$ this scenario worsen, as the controller was unable to increase water retentions in upstream conduits enough to allow downstream sections to store all excess water. As a consequence of this, water flows in outlet *L-9* increased, as well as water levels at surface conduits, as presented in figure 6.6. Furthermore, with an incoming flow of $1.5\ m^3/s$ maximum values measured for ascending flows and water levels at surface conduits are superior to those of previous tests by an order of magnitude.

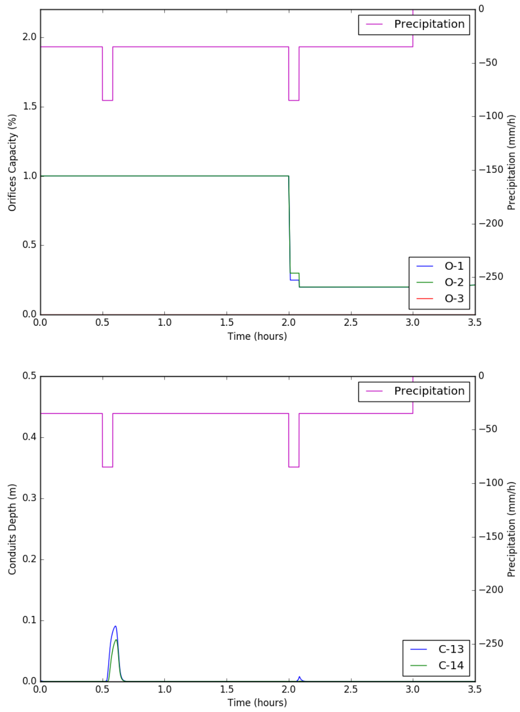

FIGURE 6.5: Actuation history in the barriers and resulting effects in water stored at a surface level (Conduits *C-13* and *C-14*) for a constant inflow of $1.4\ m^{3/s}$, considering a base level of *0.4 m*.

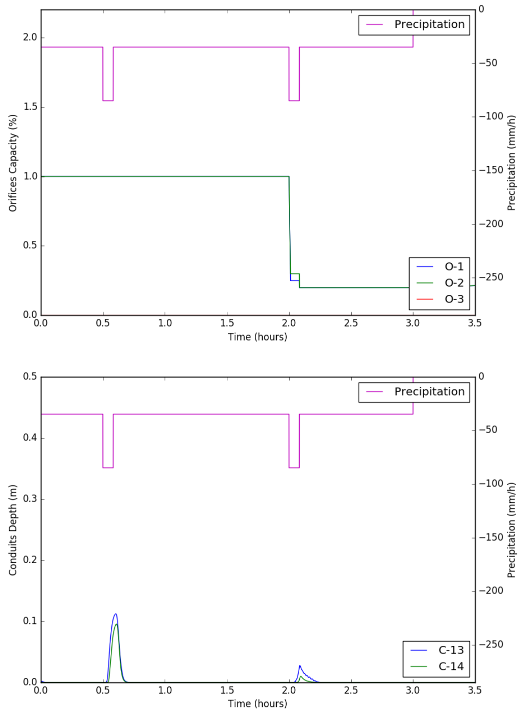

FIGURE 6.6: Actuation history in the barriers and resulting effects in water stored at a surface level (Conduits *C-13* and *C-14*) for a constant inflow of $1.5\ m^{3/s}$, considering a base level of *0.4 m*.

As in this last scenario it became clear that the controller could not avoid the occurrence of ascending water flows if even more water volumes were added to the system, no more experiments were conducted with higher values for the incoming water flow.

6.2.1.2 10-minute Peaks

If the duration of precipitation peaks is to be increased then the controller is expected to only properly remove and mitigate observed ascending flows for smaller incoming flow values.

In the performed tests with a peak duration of 10 minutes these expectations were confirmed: For incoming flows with a constant flow intensity higher or equal to $1.2\ m^3/s$ the developed controller became unable to remove observed ascending flows in outlet *L-9*.

In some experiments, even though the controller performed a significant reduction in water levels at surface conduits, maximum registered values for the ascending flow were close to a scenario with no control operations. However, in these situations, the registered higher incoming flow values occur in a timely manner and during a very short time period. At times only one, high, positive ascending flow is registered. Naturally, in these situations the registered positive ascending flow is a simulation outlier, meaning that in those scenarios the controller proved able to remove the overload scenario observed in the drainage system.

6.2.1.3 15-minute Peaks

Considering even longer precipitation peaks the tendency observed so far regarding the controller's inability to deal with higher incoming flows for longer periods was, again, verified.

With precipitation peaks lasting for 15 minutes, the developed controller was only able to remove observed ascending flows when a constant water inflow of $1.1\ m^3/s$ was considered. If more intense inflows were used (*1.2*, *1.3* m^3/s and so on) then it failed to remove these overload scenarios, or even to significantly mitigate them.

6.2.1.4 Final Remarks

Analysing the results of the performed experiments it can be seen that, for rainfall events with smaller intensity peaks (5 minutes) the developed controller provides a clear improvement, supporting and mitigating the effects of additional inflows in the system of up to 1.5 times higher than measured flows in the system during most intense rainfall periods.

Furthermore, by analysing the drainage system's response when no control measures are applied it can be seen that considerable volumes of water (rounding a few centimetres) tend to quickly accumulate at surface levels, during more intense rainfall events with longer peaks. When the controller is used this situation tends to be observed less frequently, as it promotes water retention and distribution in upstream sections of the drainage system, therefore reducing water flows that reach overloaded downstream sections of the system.

In fact, performed tests suggest that this type of approach could be successful in drainage system of considerable higher dimensions and extensions, namely when they are subjected to precipitation events with brief intensity peaks. In these type of drainage system, such as that of an entire city, several junction points in which two or more distinct conduits are merged together can be found. When this happens, the conduit in which all incoming flows are merged shall transport a flow equal to the sum of all its incoming flows.

By applying the developed and studied techniques in those junctions points one can reduce the resulting flow in the final conduit. Moreover, if this is performed in several junction points across the drainage system then substantial flow reductions can be accomplished, namely in its downstream conduits where water flows tend to be higher and junction points more frequent.

With this alternative management of existing drainage systems their longevity may be extended, allowing them to accommodate the effects of both urbanization and climate changes: As already mentioned, these two phenomena are responsible for an increase in residual water volumes drained to existing drainage systems, namely to their upstream sections, resulting in significantly higher water levels and flows in downstream sections.

As a result, underground conduits in such downstream sections may prove unable to store all water flows that reach them, leading to their overload and consequent flood occurrences. In order to prevent these events, external interventions are carried out in these systems, with the goal of increasing the height of their downstream conduits. Besides bearing a considerable impact on cities' finances, such interventions also bear a social hazard, as they often require certain roads to be blocked which can, for example, force people to take alternative routes to their destinations in daily commutes.

If an approach based on what was proposed and studied is applied such interventions may be delayed in time or, in the best scenario possible, they could become no longer necessary.

6.2.2 Variable Intensity

Simulations featuring rainfall events with precipitation peaks lasting for 10 minutes, during which measured intensity changed after the initial 5 minutes (it could either increase or decrease) will be covered in this section. More detailed information regarding the tests performed at this stage can be found in appendix G.2.

Similarly to the scenarios presented in section 6.2.1, different base levels for downstream conduits were considered in the performed experiments. For each base level the precipitation with the most intense peaks was selected, featuring two variations:

In the first one measured peak intensities would start with a the second highest intensity values during the initial 5 minutes, after which they would rise to their maximum registered values; on the other hand, in the second variation the opposite situation was considered - During the initial 5 minutes precipitation intensity was at its maximum, after which it would decrease to the second highest values.

Analysing the performed experiments there is one result that stands out immediately from the rest. This has to due with the fact that, when precipitation intensity during a peak decreases the controller seems capable of mitigating, or even removing, registered ascending flows for higher values of constant incoming flows.

In the previous section, when precipitation peaks lasting for 10 minutes with a constant intensity were considered the controller failed to remove the overload scenario in downstream conduits for constant incoming flows equal or higher to $1.2\ m^3/s$. However, if a decrease in precipitation intensity is observed during the peak incoming flows equal to $1.2\, m^3/s$ are properly dealt with by the controller. In some situations it is even possible to remove the observed overload for constant incoming flows of $1.3 m^3/s$ or even $1.4 m^3/s$ (even though, for incoming flows of $1.4 m^3/s$ this was only observed for one particular combination of base level and precipitation series).

Typically, when the measured precipitation intensity increases during a peak the controller becomes unable to remove overload scenarios for more intense incoming flows. However, when one simply increases water base levels in downstream conduits (keeping the same incoming flow intensity) this decrease in performance does not seem to occur.

In fact it could be expected that, as water base levels in downstream conduits increase, the controller would become less and less capable of removing the observed overload scenarios for more intense incoming flows. Nevertheless, as can be seem in tables in appendix G.2, if one considers base levels of $0.25\, m$ and $0.3\, m$ this is not verified: for a base level of $0.3\, m$ the controller only fails to remove the observed overload scenario for incoming flows with an intensity equal or higher than $1.4\ m^3/s$, while for base levels of $0.25\ m$ this phenomena starts to occur for less intensive incoming flows (of about $1.2\, m^3/s$).

Still, as showed in tests performed in the previous section, the controller proved to always be able to remove overload scenarios in downstream conduits of the drainage system for constant incoming flows at least equal to water flows transported in its conduits during more intensive precipitation periods (that is, water flows of about $1\ m^3/s$). In some scenarios the controller considerably improved on this, only failing to properly accommodate such phenomena for constant incoming flows of about 1.5 times higher than it *"normal"* maximum registered flow.

As stated in section 6.2.1.4, these are very interesting and promising results, that if maintained when these techniques are applied to larger drainage systems, could considerably increase their current usage and reduce the frequency of some maintenance operations. This could ultimately result in fewer flood events in urban environments triggered by an overload of underground conduits, with clear benefits of economical, environmental and social natures for urban areas.

6.3 Overload in Upstream Conduits

In the current section a series of experiments were performed in which only one goal was sought: subjecting the drainage system to precipitation events that force its controller to overload upstream sections of the system when, in response to such rainfall events, attempting to retain in then as much water as possible.

When this phenomena is registered the controller's supervisor must correctly determine that performing any sort of control operations in existing barriers installed in the drainage system will no longer be feasible, and therefore it must enter in the *Overload* state mentioned in section 5.5.1.

It is important to note that the behaviours aiming to be reproduced in this section are quite different from those discussed in section 6.2: in this case it is intended to give more focus to situations in which ascending flows in upstream outlets are observed. This occurs as a result of the actuation command issued by the supervisor while attempting to react to a more intensive and aggressive rainfall event.

Therefore, for these experiments, the 5-year, 20-year and 100-year return period rainfall events were used. A graphical representation of the 5-year return period rainfall can be found in figure B.1. Both 20-year and 100-year return period rainfalls follow a similar shape, only featuring higher precipitation intensity values.

In all the three experiments performed (one for each return period rainfall) the controller succeeded in identifying the overload scenario in upstream conduits, fully-opening orifice *O-3* in response. As a consequence of this, considerable volumes of water were removed from the system and the witnessed overload scenario in upstream sections of the drainage system ceased.

Figures 6.7 and 6.8 show an example of the controller's response to this situation. When a positive flow was registered in outlet *L-1*,around *0.7* hours of simulation, the controller immediately changed to the *Overload* state in which excess water was removed from upstream conduits even though, at that point, a considerably intense precipitation was still being registered.

As defined in figure 5.9, when water levels in upstream and downstream conduits reach lower values (coinciding with the end of the precipitation event) the control system returns to its *Pre-Actuation* state, where the setting of orifices *O-1* and *O-2* is set to 0.5.

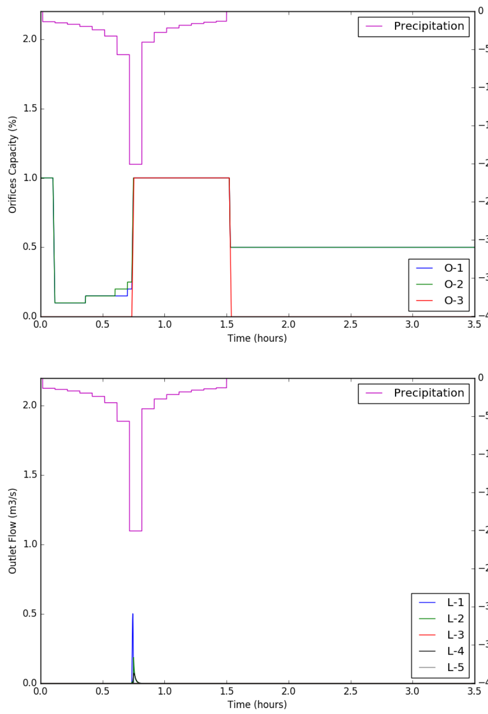

FIGURE 6.7: Actuation history in the barriers and resulting effects in water flows in upstream outlets (*L-1* to *L-5*) of the system, when subjected to a 100-return period rainfall.

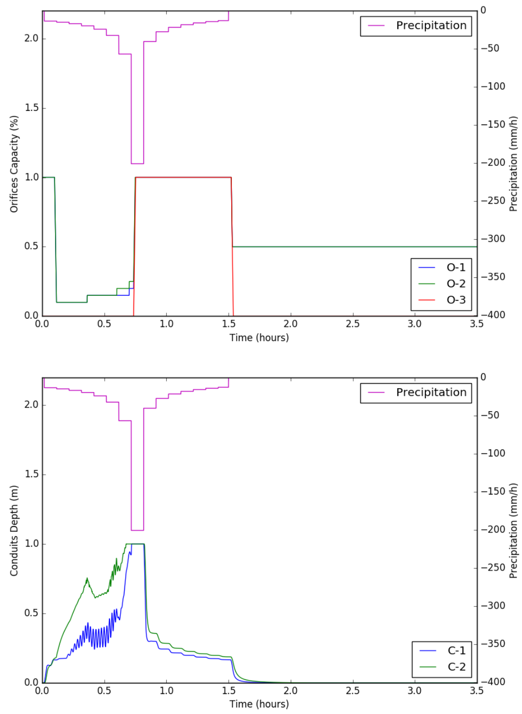

FIGURE 6.8: Actuation history in the barriers and resulting effects in water levels in upstream conduits (*C-1* and *C-2*) of the system, when subjected to a 100-return period rainfall.

From these experiments it was possible to conclude that the developed controller presents a behaviour consistent with what was initially defined, since in a variety of situations it successfully identified the existence of an overload in upstream conduits, resorting to an external water retention tank to remove excess water from the system. Additionally, it also ceased all control actions while this situation occurred.

6.4 Remarks

In this chapter a series of experiments with the developed control system were performed. In general, the results obtained were promising, as the developed control system was able to perform considerable water level and flow reductions and avoid overload scenarios in the drainage system for considerable intense rainfall events.

In an initial stage, rainfall events generated for the experiments conducted in the previous chapter, and of which resulted the development of the control system, were used as precipitation data sources. These experiments had two major motivations: on one hand they were intended to be an initial test to the control system and a way of verifying that it was behaving according to what was intended and planned. On the other hand, understanding to what extent was this approach able to perform better compared to an approach where the control rules were *hard-coded* was also intended.

When the control system was tested with the same precipitation events used in its development slight better results were obtained in some simulations, when comparing to a scenario in which actuation and control rules were previously defined, in *hard-coded* values. In fact, and even though both approaches produced very similar results in all tests, when the control system is being used the actuations performed on the barriers are more adjusted to different variations observed in the profile of the precipitation events. Contrary to what one might think when actuation rules are *hard-coded*, accurately predicting and adjusting actuation measures on the barriers can be a very complex and tricky task, made far simpler with the adoption and implementation of the control system.

Nevertheless, this alternative usage of the existing drainage systems appears very promising, as considerable flow reductions in downstream sections were achieved: starting from a maximum flow ranging from $0.9\,m^3/s$ when no control measures were being taken, in a vast set of scenarios this value was reduced to about $0.7\,m^3/s$, and in some special cases it was even possible to reduce it to about $0.5\ m^3/s$. These reductions are, obviously, easily obtained for shorter precipitation peaks. When longer peaks are considered maximum flow values tend to rise, in some cases to values closer to the original $0.9\ m^3/s$.

Even when constant incoming flows were added downstream of the installed barriers, the developed control system proved able to perform considerable water retentions that had a significant impact in downstream sections: when no control actions were being taken, for more intense incoming flows, downstream sections of the drainage system became overloaded, resulting in the occurrence of ascending water flows to the surface. However, when the developed control system was used a wide range of these situations were removed, and even when downstream sections remained overloaded, a considerable smaller amount of water was able to reach the surface.

In fact, in the vast majority of the situations, for constant incoming flows of $1.2\,m^3/s$ the developed control system easily attenuated water levels in downstream sections of the drainage system. In some simulations this remained the case for constant incoming flows of $1.4\ m^3/s$. Considering that, for all the rainfall events generated and used in this work, the maximum registered water flows in downstream conduits ranged $0.9\ m^3/s$, it was proven that, with the help of the control system, it is possible to deal with external junctions and derivations in which constant water inflows similar to the maximum values registered for this system are constantly being drained. As mentioned in previous sections of this chapter this is, without a doubt, also a very interesting and positive result.

When, during the course of the actuation measures taken by the control system on the barriers, this proved unable to retain excess rainfall water in the drainage system's upstream sections, ascending water flows tend to be observed as a result of an overload in these sections.

Since, when there is an increase in the rainfall intensity it is necessary to act on the barriers in order to restrict water flows that reach downstream sections of the drainage system, the occurrence of these ascending flows (when the control system is acting on the barriers) is an indicator that no more water can be stored in upstream sections. At this point, there is a need to maintain or increase water retention, counteracted by the availability of free space to do so. When this happens the control system has exceeded its limit, and maintaining any control actions ceases to be a wise choice, as it will only increase the amount of water that exists initial sections of the system, worsening the situation.

In fact, in the tests performed with this goal, the control system was always able to correctly identify these situations, ending all control actions issued on the existing barriers. In its place, the control system promoted the opening of all orifices in the drainage system in order to remove all excess water from upstream conduits as fast as possible.

If, on one hand, performing the correct actions on existing barriers in response to precipitation intensity peaks is critical to the success of the approach proposed this thesis, on the other hand correctly detecting, and as soon as possible, that the implemented control system is no longer able to serve its purpose is as or more important, since the last thing one wants to do in these situations is to worsen the problem that one is trying to solve.

Chapter 7

Conclusion

In the current document, the usage of barriers in underground conduits of a drainage system as a way of retaining water in its upstream sections, reducing water inflows to downstream conduits, was presented and studied.

A control system responsible for monitoring a given drainage system, determining when and how to actuate on a given set of barriers in order to attenuate as much as possible the effects of intensive rainfall events in downstream sections of the drainage system, was developed. This development focused on a particular group of drainage systems, with an architecture similar to that of the system presented in section 5.2.

The developed control system was tested under a certain set of distinct rainfall events, with different impacts on the drainage system. As no previous exploration of this sort of approach and techniques had been performed, an initial exploration stage focusing on how to properly use the mentioned barriers to perform water retentions in a given drainage system was conducted. In total, more than a thousand experiments were conducted in this work, considering different drainage systems, precipitation profiles and actuation strategies.

Precipitation profiles featuring sudden and short intensity peaks were extensively used in experiments throughout this work. Among these rainfall events the ones with shorter intensity peaks were considerably easier to accommodate, allowing for considerable level and flow reductions even when significantly higher intensity values were being considered. As the duration of these peaks increases the performance of the control system starts to drop when subjected to precipitation events with higher peak intensities.

Foreseeing the application of this approach to drainage systems of higher dimensions and extensions it was found that the developed control system can accommodate the effects of incoming flows significantly higher than the maximum flow values recorded for the tested precipitation profiles. As in other scenarios, this accommodation capacity is largely dependent on the duration of precipitation peaks, being higher for shorter peaks.

Based on what as stated in previous paragraphs it can be concluded that, on all the scenarios studied, the developed control system is able to prevent flood events up to certain limits, which are consistent with the effects of climate change and the problems of urban expansion. Moreover, these results can be achieved without the need to perform any type of intervention in the drainage system aimed at modifying in any way its existing infrastructures (which typically involves increasing the height of underground conduits or, when this is no longer possible, performing a duplication of the existing conduits).

Even when water retention in upstream sections reaches its limit, water withdrawals can be promoted in those locations, redirecting excess water into external retention basins, again without the need to perform any type of intervention in the drainage system.

Even though the results obtained in the course of this work show serious promises regarding the application of this novel approach in real urban drainage systems, they are not enough to justify (or allow, for that matter) their adoption. In fact, promising and important first steps have been taken in that direction of which resulted a series of valuable contributions, detailed in section 7.1. As mentioned in section 1.5, under the collaboration established with the research group at

DEC the knowledge and intelligence collected from this work will be provided to this group and used to enrich their collaboration and participation in the *H2020* European project.

However, additional validation stages are still required, featuring precipitation events and drainage systems with different profiles for instance, before moving on to a production stage. More detailed considerations and reflections regarding this stages, and other possible and valuable extensions and complements to this work, are presented in section 7.2.

7.1 Contributions

In the current section the main contributions that resulted from this work are listed and briefly discussed:

- Extensive study on the usage of barriers installed in underground conduits to, for a drainage system with a given set of characteristics and properties, determine under what conditions are these infrastructures capable of properly retaining water in upstream sections of the drainage system, reducing water inflows that reach downstream sections.

  As already presented and discussed, the results obtained so far in this work are promising, motivating the exploration of the usage of these techniques in additional scenarios, with drainage systems of distinct dimensions and characteristics.

- Additionally, the barriers in underground conduits should be placed at a safe distance between each other, thus avoiding to be installed in a conduit that is affected by another orifice, downstream of its location.

  As mentioned in 5.6, two barriers installed in underground conduits must be separated in such a way that the upstream barrier is not affected by retention effects caused by the downstream one.

- Development and implementation of a control system specified in a state machine, instead of having to resort to a sequence of *IF-THEN-ELSE* instructions to specify the controller's behaviour.

  The *SWMM* software only allows the definition of control rules for barriers in the form of a sequence of *IF-THEN* instructions. However, as already mentioned, considering the complexity wrapped around the definition of all possible sequence of events that can occur it is far more convenient to make use of a state machine to define the behaviour of the control system.

  Even though it was not implemented in the course of this work, providing end users with a graphical interface in which the behaviour of a control system could be specified in the form of a state machine would constitute a major contribution to this work.

  In fact, with an interface that required no knowledge of how the software works or how the control system should be implemented would, certainly, be extremely useful for this scientific community. For starters, many of the most popular and used software of this nature do not allow for much dynamism in the parameters of their simulations. Furthermore, in the few parameters of the simulations that can be changed in the course of a simulation, there is little freedom regarding what can be changed and how can it be changed.

  Thus, even though this topic is far from being closed, important initial steps have been performed in this work towards the implementation and specification of this new functionality.

- Improving on the ideas presented in the previous point, in the current work some dynamism was introduced in the simulation of these hydraulic models which, as already mentioned, was not supported in current simulation applications of this nature. Even though, in the scope of this work, no formal and official contributions have been made with respect to this point, the work accomplished so far can be seen as a proof of concept, potentially becoming a trigger for a more generalised implementation and adoption of these contributions and concepts.

Besides studying and implementing ways of managing barriers in underground conduits as a way of storing excess water in them, the current work introduced more dynamism to other simulation components, such as allowing for variations in actuation speeds on the barriers or in water inflows at different junctions points in the drainage system.

By taking advantage of direct access to the software's source code, it was possible to modify these parameters at any time during the simulation. In practice the entire simulation environment can become much more dynamic, which as of right now does not seem to be fully supported in *SWMM* and other similar applications. Obviously, the introduction of this dynamism has great interest to the scientific community in this field, as it allows these models to better represent the reality of these phenomena.

7.2 Future Work

In future iterations of this work a wide variety of improvements on what has already been achieved can be sought, some of them playing a key role in a possible application of this approach to real drainage systems in urban environments. From them, the more important ones are presented in the listing that follows:

1. As already mentioned in the previous section, implementing an interface for the definition of the control system, responsible for its validation and implementation, without the need for user interaction (except in its specification) is definitely an idea to be explored.

2. Extending the proposed approach to other urban drainage systems.

   All the simulations executed and documented in previous chapters of this document made use of the same hydraulic model. From these experiments resulted the development of a control system that, inevitably, is considerably tuned to this particular environment and model.

   In fact, testing the developed control system in different environments, in which the existing drainage systems have other architectures, extensions and morphologies is an important task. Only by performing such experiments is it possible to tune some parameters of the developed control system, aiming to produce a more general solution.

   In this particular point, there is a great interest in studying the application of these techniques and approaches in hydraulic models of more extensive drainage systems.

   In this study the approach to be followed would be similar to the one adopted throughout this work: starting with *well-behaved* and less aggressive rainfall events, a first version of the control system would be developed. Once it was finished, rainfall events with more intensity variations would be used seeking to adjust some of its parameters.

3. When extending this approach to larger drainage systems, such as the ones installed in a city, it might be appropriate to reformulate the developed control system, allowing it to best fit the size and extension of the territory that is being monitored.

   Therefore the territory to be monitored should be divided into several areas of similar dimensions. In turn, the control system should be composed of a series of *Supervisors*, each of which in charge of monitoring a given area and controlling all barriers there installed. Additionally, an higher level entity would also be required, responsible for monitoring all the *Supervisors*, collecting and processing a general overview of the behaviour and state of the drainage system.

4. When working with more extensive scenarios it is expected that all *Supervisors* have similar state machines. Nevertheless, because water flows in different areas may have distinct behaviours small variations and changes in these state machines are expected. Similarly, the actuation rules defined in the *Supervisor's Actuation* state should also expected to remain similar.

   However, the main difference between these areas should lie in the control objectives of their *Supervisors*. Throughout this entire work, all simulations and experiments have been conducted over a relatively small area, always with the same control objective: prevent water flows from reaching the surface level as a result of pressure flows.

   Nevertheless, when working with a drainage system of considerable larger dimension there may exist some areas in which preventing these pressure ascending flows is more critical, when compared to others. In this sense, there may be some scenarios in which these situation cannot be avoided, meaning that it would be better to promote their occurrence in a given area, thus preventing it from occurring in another, in which the resulting impact would be worse.

   In this context machine learning based approaches could make a strong contribution, meaning that their exploration would also be interesting, as a way of automatically determining the better distribution of control objectives across the different areas of a drainage system. In fact, in quite extensive environments, performing this task manually is an intractable problem, given its size and complexity.

5. Furthermore, also when working with considerable larger and extensive drainage systems, the integration of the control system in a *Geographical Information System (GIS)* could really bring an additional and very interesting complement to this work.

   When simulating the application of these control approaches to such systems, or even in a real scenario where the controller is deployed directly in the drainage system under monitoring, a GIS platform would allow for a better integration and visualization of all the collected information (both with respect to the current state of the drainage system and current actuations performed on the barriers).

   In this sense, by building an application supported by a GIS platform it would be possible to easily combine information from the different areas of the drainage system in a simple and interactive way: a map of the drainage system being monitored could be shown, in which the different areas composing it would be highlighted. It would then be possible to select one of those areas for further inspection. When this is done a more detailed scheme of the drainage system in that area would be shown, along with information of the current state of the system, collected in real-time from that area.

   In this concept, one could also think of the GIS application as a connecting platform between the control system and the physical actuators installed in the drainage system, responsible for performing the issued actuations on the barriers.

   Furthermore, *Virtual Reality* techniques could also improve the proposed representation component, targeting underground conduits, namely in areas where their dimensions are higher (specially in height). Thus, when a given area was selected in the GIS platform for further inspection a *3D*, fully immersive representation of the underground conduits of the drainage system in the selected area would also be available.

6. Further investigation regarding the positioning of the barriers in systems of this type is also a topic of great interest to be studied.

In fact, locations for underground barriers are dependent on the length, height and slope of existing conduits. In the scenario considered in this work, when faced with a gentle slope in upstream sections the need to consider more than two barriers was not felt. However, if such sections had a more steep slope and longer conduits, an extra barrier could have allowed for an increase in water retention capacity.

Therefore, further research in this topic is also very important, as it allows for better and more considerable water retentions using these barriers.

7. Finally, following this work scientific articles are being prepared with the intention of being submitted in international conferences in this field, such as *IWA/IAHR International Conference on Urban Drainage*[1].

[1] `http://www.icud2017.org/important-dates.htm`

Appendices

Appendix A

Related Concepts

As mentioned in 7.2, in future iterations of this work a wide variety of improvements with contributions from distinct areas can be idealized and pursued. From these contributions, the integration of current *Geographical Information Systems* and *Virtual Reality* technologies are seen as potential major players and contributors.

With this is mind, in the current chapter these concepts are briefly covered, with a special focus and emphasis in the *Virtual Reality* technologies.

A.1 Concepts Definition

In the current section a series of important concepts adopted throughout this chapter will be defined.

A.1.1 Geographical Information System

A *Geographical Information System (GIS)* can be defined as a system designed to capture, store, manipulate, analyse and present geographically distributed data [94]. However, this is a broad term that has distinct meanings.

A *Geographical Information System* consists in any software, hardware, information system, computational procedure or even human resources capable of collecting, storing, processing and/or presenting data in a way accessible to others. Many *GIS* applications can be seen as tools capable of analysing, editing and presenting spatial data in different map types, allowing users to perform interactive queries on the data being processed.

Geographical Information Systems are also very popular when dealing with remotely sensed data. Many *GIS* applications support real time data acquisition [95], processing and representation, enabling a visualization of the data collected and its evolution over the time.

As stated in section 1.2, in the scope of this thesis *Geographical Information Systems* will not be used to perform the core data analysis. Such task will be performed by the *Decision System* to be built, while *GIS* applications will mainly be used to display the information provided by such intelligent system.

The main reason to use *GIS* to represent different flood scenarios is related with existing map integrations of current *GIS* applications, enabling a geographical visualization of the data processed, enhancing one's perception and understanding of the phenomena under study.

Regarding the topic of floods, many applications using *Geographical Information Systems* can be found in the literature. Emmanuel Opolot has published a state of the art paper review of *GIS* and remote sensor applications in flood management [96]. In this paper, *GIS* and remote sensors applications are often coupled with hydrological and mathematical models. *Digital Elevation Models (DEM)* are also referenced as a popular source of input data in such applications.

A.1.2 Virtual Reality

The term *Virtual Reality (VR)* can be defined as a computer-generated environment that allows a given user to interact with it in a completely immersed way, without being able to distinguish it from the real world [97, 98, 99].

To accomplish this, *VR* systems usually require that its users wear specific hardware, such as an *Head-Mounted Display (HMD)*, promoting their complete immersion in the virtual world.

Currently, *VR* systems have many different fields of application, ranging from education, to entertainment, healthcare or even military applications. Their success in so many areas is mostly due to their flexibility, intuitive interaction, complete abstraction of the reality and inherent safe environment in which they can be used.

Although it may be difficult to categorise all forms of *VR*, most can be classified in one of three categories [100]:

1. *Non-Immersive Systems*, providing the least immersive implementation of *VR*.

   Typically they consist in a computer desktop system, where the virtual environment is viewed through a portal or window using a standard high resolution monitor. The user can only interact with the environment in conventional means, such as using the keyboard or mouse.

2. *Fully Immersive Systems*, giving the user the closest experience to reality possible by providing a field of view of 360º and requiring a *HMD* or any other form of head-coupled display to exclude the user from the real world that surrounds him.

3. *Semi-Immersive Systems*, which fall between *Non-Immersive Systems* and *Fully Immersive Systems*. Usually these systems provide a field of view ranging from 100º to 150º and are combined with virtual reality inducing equipment.

A.1.3 Augmented Reality

A well-known variation of virtual environments that often gets mistaken with this technology is *Augmented Reality (AR)*.

AR can be defined as a real time direct or indirect interactive representation of a real environment where virtual 3D objects are integrated with the purpose of enhancing one's perception of reality [97, 101, 102].

An *AR* system can be seen as a blending between virtual reality and real life, as users can interact with a computer-generated virtual content, placed in the real world, while being able to distinguish between reality and virtual contents.

FIGURE A.1: Comparison between Virtual and Augmented Reality [103].

A.2 Virtual Reality Literature Review

The current chapter contains a brief overview of the virtual reality field, focusing on the *Oculus Rift* device. This study was conducted in an initial stage of this work, before the explosive growth period in these technologies, mostly observed during the 2016 year.

This study was mainly motivated by the fact that, at that time, the incorporation and integration of *Geographic Information Systems (GIS)* in this work was being equated. In this way, *Virtual Reality* technologies were seen as valuable additions and complements to GIS solutions, as they allowed for a fully immersive representations of underground conduits, thus providing an additional representation layer.

At that time, the *Oculus Rift* was the most popular *VR* technology available in the market, which highly contributed to its choice to be the main feature in this study. As the work was being developed, new challenges emerged, with potential greater contributions to the final outcome of this thesis, motivating a gradual abandonment of this component. Because of this, the literature review study presented in this chapter is only updated until the fall of 2015. All major trends, updates and new players in this field registered during the 2016 year have, therefore, not been included.

Nevertheless, and because the integration of *VR* and *GIS* technologies in future iterations of this work important contributions may result, the previously conducted *VR* literature review was added as an appendix to this document.

A.2.1 Introduction

Considered by many as the next mass trend, Virtual Reality is becoming integral part of our lives. The *Oculus Rift* in 2012 by Palmer Luckey triggered a revolution in this field, with a lot of companies deciding to invest in VR devices and software.

However, Virtual Reality is not an entirely new concept. Its earliest traces point back to 1929 when Edwin Albert Link developed a simple flight simulator, which as patented in 1931 [104]. A few years later, in 1938, Artaud introduced the term *Virtual Reality* in his collection of essays, *Le Théâtre et son double*. Since then, the field of VR has experienced successive changes, leading to today's popular *Head-Mounted Displays* devices.

In 2012 Palmer Luckey and Brendan Iribe founded *Oculus VR* and launched the first prototype version of the *Oculus Rift*, a fully immersive VR HMD device. This device was a success, convincing *Facebook* to purchased the company in 2014 for $2 billion [105]. In the same month the device's second prototype version was announced. Recently, the release date of the first consumer version of the device was also revealed, targeting the first quarter of 2016 [106].

After the *Oculus* release there was a major boom in the VR market. Many VR *startups* were created, and like *Oculus VR*, some of them were bought by larger and more dominating companies in the technological market [107, 108]. Besides leading to an increase in the number of virtual reality applications available in the market, this enthusiasm was also responsible for the release of other HMD devices by *Oculus VR's* competitors. As of 2015, *VR afficionatos* have plenty of devices to choose from, with more expected to be launched in the coming years. A more detailed list with popular HMD devices under development is available in [109].

Since 2012 there has also been a big enthusiasm in bringing Virtual Reality to web pages and mobile phones: by using libraries and API's such as the *WebVR*[1] , developers have been able to access popular VR devices in applications running on the user's browser. With the release of *Samsung VR*, *Zeiss VR One* and *Google Cardboard* [110, 111], it was also possible to develop applications for some mobile platforms, most of them targeting *Android* and *iOS* devices.

Nevertheless, it is important to understand that there is more to Virtual Reality than the popular *HMD* devices. With the recent boom in the wearable technology industry there is a natural

[1] `http://webvr.info/`

tendency to assume that all VR environments must make use of *HMD-like* devices and provide a fully immersive experience. As presented in section 2 , this assumption is completely wrong. A simple computer game that creates a new, virtual, environment where users interact with it using the mouse and keyboard is a VR System, just not a fully immersive one.

The remainder of this chapter will be organised as follows: a short presentation of the *Oculus Rift* will be made in A.2.2 , followed by an overview of the most popular and used tools for developing applications for the *Oculus* device in A.2.3 . Finally, in A.2.4 some applications of this device will be covered.

A.2.2 Inside the Rift

As stated in A.2.1 , the *Oculus Rift* is a fully immersive *Virtual Reality Head-Mounted Display* device, mainly developed by Palmer Luckey in the company he founded, *Oculus VR*. At the time of writing *Oculus VR* had already launched two developer versions of this hardware, the *Oculus DK1* and *DK2*, and announced the launch of the device's first commercial version for the first quarter of 2016.

In both released versions, the *Oculus Rift* makes use of head tracking sensors and optical lenses to give its users the illusion of actually being within a given virtual environment. To support this illusion the device renders the same image to each lens in a slightly different perspective, covering the user's field of view, so that the only thing he/she can see is the image processed by the device and rendered to the lenses.

In order to provide a realistic feedback to some head movements, the device is also equipped with head tracking sensors that continuously report the user's head orientation quaternion, allowing the device to rotate the image at almost the same time the user rotates his/hers head.

The second version of the device also features positional tracking. This was accomplished by building an IR-LED[2] array into the device, which is tracked by a CMOS[3] camera positioned about 1.5 meters away from the user [112]. With this setup the device is able to capture more subtle movements of its users, like leaning in to inspect a given object, and therefore is able to provide an even more realistic and immersive experience.

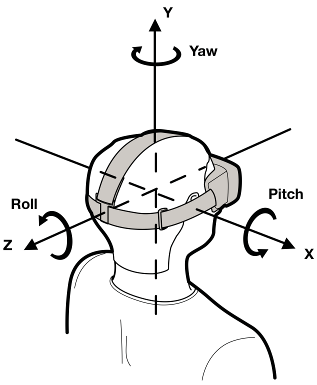

FIGURE A.2: Oculus Rift head tracking [113].

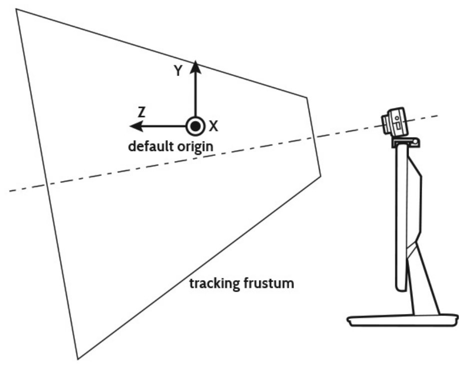

FIGURE A.3: Oculus Rift positional tracking [113].

Another major improvement in the *Oculus DK2* was the display resolution. One of the main critics of the DK1 was related with the resulting image. With a resolution of 640x800 on each lens (resulting in a total resolution of 1280x800), the generated image appeared distorted and with an imperfect overlap between the eyes. This problem was addressed in the second prototype of the

[2]Infrared Light-Emitting Diode.
[3]Complementary Metal-Oxide Semiconductor.

device, which featured a resolution of 960x1080 on each lens (total of 1920x1080), providing an improved immersive experience.

A detailed list with the specifications of each device are presented in table A.1 :

TABLE A.1: Oculus DK1 and DK2 Comparison.

	Oculus DK1	Oculus DK2
Resolution (per lens)	640x800	960x1080
Refresh Rate	60Hz	75Hz, 72Hz or 60Hz
Persistence	3ms	2ms, 3ms, full
Field of View	110º Nominal	100º Nominal
Display Technology	LED	OLED
Display Size	4.08″	5.7″
Head Tracking	Yes	Yes
Positional Tracking	No	Yes
Internal Tracker Sensors	Gyroscope Accelerometer Magnetometer	Gyroscope Accelerometer Magnetometer
External Tracker Sensors	None	CMOS Sensor Near Infrared
Weight	380g	440g

A.2.3 Developing for the Oculus Rift

Since the launch of Oculus' first prototype version Palmer Luckey has stressed that this is primarily a gaming device, meaning that the main target of his company will be the gaming industry.

This had an immediate impact in such industry, with several companies seeking to develop software applications compatible with the device. It also encouraged other companies to develop their own VR devices, attempting to compete with *Oculus VR*. As a result many alternative VR devices have already been introduced in the market, and even though most *VR-compatible* software is still targeting developers an increase in the number of applications available in the market for consumers is expected in 2016.

With this increasing need to develop applications for VR devices, specially in the gaming industry some existing software tools, such as game engines, were adapted to support these new devices. Furthermore, in some cases new software tools supporting *VR programming* were created, introducing VR to new environments. Because the Oculus VR was a pioneer in this field, many of this tools fully support the rift.

A.2.3.1 Oculus SDK

A few months after the *Oculus Rift* first release, *Oculus VR* launched an official driver for the device, which included *LibOVR*, the core library needed to develop applications for this device [114, 115]. Additionally, sample integrations for the *Unity* and *Unreal Engine* game engines were also supplied.

The *LibOVR* is a *C++* library that provides an interface to interact with the rift without forcing developers to implement mechanisms to communicate with the hardware. However, this is still a low-level library in the sense that the developer must handle everything in the scene that is being rendered: for example, in a first person view, in order to make the scene's camera rotate while the user is rotating his/hers head, the developer must detect the user's head movement and apply the appropriate rotation to the camera, as *LibOVR* only provides the information from the device's sensors.

Even though the *LibOVR* gives almost complete control to developers, most of them prefer to use current Oculus integrations with modern game engines as they allow faster development, therefore enabling developers to create more complex applications in less time than if they used *LibOVR*.

Furthermore, when using such engines developers do not have to worry about handling the input provided by the device (no more rotating the camera according to the user's head orientation. That is already handled by the engine internally) or about writing any *device-specific* code: the same application, running the same code, can be executed in a standalone computer, with no VR devices connected, or in any VR device that is compatible with the game engine that is being used. As the reader can understand, this is a strong advantage in favour of modern game engines.

A.2.3.2 Game Engines

Game Engines are software tools specifically designed to create and develop video games. Their main purpose is to simplify and speed up development by implementing core functionalities common to all video games, allowing developers to focus on implementing *game-specific* content.

With the recent growth of the VR industry, and consequent focus in the gaming sector, developers of both game engines and VR hardware hurried to create stable integrations of these devices in popular game engines, thus providing an adequate and familiar development environment with a minimum learning curve.

Regarding the *Oculus Rift* three game engines are currently being officially supported [4] : *Unity4*, *Unreal Engine4* and *CryEngine*. Even though integrations for alternative game engines are also available[5], the remainder of this section will be reserved to the three game engines presented.

When it comes to graphical content, *Epic Games' Unreal Engine 4 (UE4)* is the next generation game engine, with an outstanding performance in nearly all possible graphical representations: terrain, dynamic lighting, shadows and post processing effects. UE4 can also handle up an enormous number of particles in a scene at one time.

As of March, 2015 *Unreal Engine 4 (UE4)* is available to everyone for free, including the engine's source code. The default programming language is *C++*, but there is also support for a *Blueprint Visual Scripting System* (based on a node-interface to create gameplay elements from within the engine's editor). UE4 software can be released on PC, Mac, Linux, iOS, Android, Xbox One and PlayStation 4, supporting popular VR devices, like the *Oculus Rift*, *Samsung Gear* and *Steam VR*. This is clearly an advantage for many developers in this industry.

Even though UE4 has quite an extensive documentation it is very difficult to master, mostly because of its many features and complexity. Another issue with this engine is regarding its *Asset Store*[6] . When compared with other competing engines, UE does not provide much starter content, nor there are many assets available to purchase or download, forcing developers to implement extra features, already provided in other competing engines.

Nevertheless, UE4 is still one of the best game engines available in the VR market, and a key tool for experienced developers.

Unity is probably the best game engine for 2D content and even though it does not have UE4 3D graphical power, it is still a strong competitor and a good choice for creating 3D content. In Unity developers are free to choose between *C#* and *Javascript* as their development language, but for more complex tasks C# is generally recommended.

Unity software can be released on PC, Mac, Linux, iOS, Android, BlackBerry, Windows Phone 8, Xbox One, Xbox 360, PlayStation3, PlayStation 4, PlayStation Vita and PlayStation Mobile. It also supports many VR devices, such as *Oculus Rift*, *Samsung Gear* and *Steam VR*.

[4]According to `https://www.oculus.com/en-us/dk2/`

[5]However they are not officially recognised and supported by *Oculus VR*.

[6]Or *Marketplace*, as it is called by Epic Games.

Unlike UE, *Unity* is fairly easy and quick to learn, providing users with a lot of starter content, most of it from the engine's asset store. This store is one of *Unity's* strongest features, as it contains many useful assets available for free, thus relieving developers from the need to implement extra features.

The engine suffers in the amount of editing capabilities it provides. *Unity* has no real modelling or building features outside a few primitive shapes, meaning that more advanced processing must be handled in a third party application.

CryEngine (CE) is another extremely powerful and popular game engine in the gaming and VR industry. The graphical capabilities of this engine are equivalent to those of UE4, surpassing other engines like Unity. Like UE, *C++* is the main programming language in this engine. *LUA* is also available, more as a complement to *C++* than as an alternative. In fact, most CE developers use *C++* for implementing the applications' core mechanics, and *LUA* scripting for less important tasks. The main reason for using *LUA* in these situations is the ability of debugging the application, without having to restarting it.

Of the three game engines presented in this section, CE is probably the one with the hardest learning curve, meaning that it is the most difficult to learn and master. The main reason for this is the fact that, unlike Unity or UE, CE was not built to be used by a single person. This engine is more suited for small groups where each person can focus on a specific task (such as programming, level design, among other) and therefore more complex.

Deciding which game engine to use in a given application is a little bit like deciding which operating system to install in our machine: some excel at a given set of tasks while others are more suitable for different contexts. Some people only use one of them, while others are constantly trying all of them. With game engines there are developers who only work with a given engine, regardless of what they need to do, and there are others who keep trying different ones, identifying which engines are more suitable to what they need to do. Of course, both of these approaches have their advantages and their problems.

At the end of the day there is no right or wrong answer that suites all possible scenarios: ultimately, it relies on the software to be built, the team that is going to build it, the experience that such team has with the different tools and, of course, the individual preference and productivity with each solution.

A.2.3.3 WebVR

A few months after the release of the *Oculus Rift* a new trend started to appear in the *Virtual Reality* industry: *WebVR*, mainly consisting in fully immersive *Virtual Reality* applications executed on the browser and shared on the Internet.

In fact, this idea is not entirely new, as the *World Wide Web* and browsers had already been used to share *Virtual Reality* content. The real novelty in this approach is the ability to connect an HMD device to the browser, providing a fully immersive experience to the users.

Browsing popular WebVR repositories one can find two major implementations of such concept: the *vr.js* library [116] and a modified version of the *THREE.js* library [117, 118, 119].

The first one, *vr.js*, is a *JavaScript* library that uses a native browser plug-in to exposes VR devices, handling all the communications and operations required for rendering lens distorted and responsive scenes in the device. *vr.js* offers support for two VR devices, the Oculus Rift and the Razer Hydra[7] .

The second major implementation is built on top of *THREE.js*, a popular JavaScript 3D library, adding extra features to handle stereo rendering and process input received from the device. Examples of this implementation can be found at [117] and [119].

[7] http://www.razerzone.com/gaming-controllers/razer-hydra-portal-2-bundle

The main idea behind WebVR is to create a *JavaScript API* that provides access to *Virtual Reality* devices in the browser. Currently, only two browser are providing support for WebVR applications: Firefox Nightly builds[8] and Chromium Experimental VR builds[9] .

Even though WebVR may not be as popular as other VR applications, interest in this topic has increased since it was initially introduced. With the current developments in MobileVR, in which users can run fully immersive Virtual Reality applications in their Android phones connected to VR devices like Google Cardboard or Samsung Gear VR, we could potentially witness a merge between these two concepts that could for ever change the way we see VR today.

A.2.4 Oculus Rift Applications

Virtual Reality can be used in almost any area to which computers are applied, if not all. Of all *Virtual Reality* applications those in the gaming industry are the most popular, and one particular set of devices has captured the attention of most developers and users in the last few years: the *Head-Mounted Display (HMD)* devices, specially Palmer Luckey's *Oculus Rift*.

Allowing its users to fully immerse themselves in the content that is being displayed, HMD devices are being used in all sorts of scenarios, varying from entertainment, to educational or even military tasks. In the remainder of this subsection some of the trending and most popular applications of the *Oculus Rift* will be addressed.

Right after the *Oculus Rift* launch in 2012, many developers predicted that such devices could be used to increase the user's immersive experience in areas besides computer games. One of which was the film industry, namely *360º Video* [120]. Thanks to *Total Cinema 360* [121], a proprietary software, it is possible to watch live action video content in HMD devices like the *Oculus Rfit*. *Oculus VR* has also released its own application for watching video content in the Oculus Rift device, called *Oculus Cinema*. In this application users are placed in the middle of a virtual theatre where a movie is playing on a simulated screen [122]. As of March, 2015 *Youtube* also joined the *Immersive VR* experience, officially supporting 360º video uploads whose reproduction is already supported in HMD devices such as the *Oculus Rift DK2* or the *Google Carboard* [123, 124].

In the entertainment field, a relatively new application for HMD devices like the Oculus Rift is *Virtual Turism*. Thanks to applications like *OculusStreetView*[10] users can now explore different places in the world, in a fully immersive *first-person view*, without having to actually travel to their destination.

Virtual Reality also has an extensive background in militar applications [98, 125]. Making use of realistic simulations to train soldiers in combat, handling vehicles, flying planes and helicopters, studying targets and preparing missions, among other examples, a strong investment in this field was made, contributing to a bigger development in this field. For the special case of the Rift, they are mostly used in soldier training [126], along with *Augmented Reality* technologies [127] and Haptic devices [98, 128, 129].

Medical applications of *Virtual Reality* have also proved effective in both clinical research and training. Given the practical nature of this field, there are few applications using only fully immersive devices, like the Oculus Rift. The vast majority of medical applications are paired with *Augmented Reality* scenarios and Haptic devices. Most common examples of such applications are surgery simulators [125, 130] and virtual environments to help patients overcome traumatic events [125, 131], but there have been some experiments using Virtual Reality and the Oculus Rift as a way to distract patients from their pain [132].

Another field where Virtual Reality and *Rift-like* devices can make a real contribution is in remote equipment operation. Computers and robots have been used to replace humans in many tasks, specially in those representing some sort of danger to humans or where physical access

[8] `https://nightly.mozilla.org/`

[9] `https://vr.chromeexperiments.com/`

[10] `http://oculusstreetview.eu.pn/index.html`

is very limited or non-existent. With humans safely controlling machines, dangerous tasks have become easier, safer and sometimes even faster to perform, as machines take less time to execute some assignments when compared to humans. By making use of fully immersive devices, operators can control their equipments in a safe environment and still have a realistic experience, as if they were physically operating the equipment. Some application examples where the Oculus Rift and other HMD devices play this important role are remote control of construction and underwater robots [133, 134].

Browsing the literature one can also find a general interest in applying virtual reality to *Data Visualization*, namely *Big Data Visualization*. In fact, in July 2015 a Big Data Virtual Reality challenge [135] sponsored by the *Wellcome Trust* and *Epic Games* took place in Brighton, England, offering a prize of $20,000 to the team that better explored how can these two concepts be merged and applied to the scientific community.

Current Big Data analytics rely on pattern recognition techniques to identify hidden patterns in the datasets and thus extracting relevant information from them. As it is known in this field, a certain level of sensitivity and familiarity with the dataset is recommended, as it usually leads to better results. In most cases, this can be accomplished by *"playing"* a little bit with the data at hand in order to understand it better and the effects some deviations can have on the data and on the conclusions extracted from such data. As pointed out in [125] this experimenting phase with the dataset is possible in a virtual environment, namely, in a fully immersive one. For datasets with multiple dimensions, visualizing them in a 3D immersive environment, where they can move the data, resize it, etc, can help then understand and acquire some sensitivity towards the data, leading to better results in the information they extract from it.

Regarding applications of the Oculus Rift in *Flood Monitoring* systems, there appears to be very few work published. Browsing the literature one can find multiple examples of flood monitoring systems, most of them making use of computer simulations and mathematical models to monitor and predict the evolution of the water level.

The closest example found to a flood monitoring system making use of some sort of fully immersive Virtual Reality device is the work of Ibrahim Demir [136]. In this work a web-based interactive simulation environment was developed, aiming to introduce hydrological concepts to Civil and Environmental Engineers. To accomplish this the system provided its users with a series of simulations of different flood scenarios, supporting their representation with augmented reality, as well as non-immersive and immersive virtual reality techniques.

Appendix B

Hydraulic Model Exploration

The current chapter was introduced in this document to provide a brief overview of the study and exploration of the hydraulic models performed in the scope of this work.

With this exercise, in an initial phase, it was intended to acquire experience and sensitivity in the parametrization of the models and in the effects of subtle variations to such parameters. More specifically, since this work intended to explore the usage of barriers in underground pipes to store water, assessing the feasibility of this solution became a critical step. In fact, it would be impossible to attempt any kind of control operations without having a clear picture of the system's operation mode and, above all, its boundaries.

Initially, historical rainfall time series were used in the performed tests and experiments, namely 5-year, 20-year and 100-year *return period rainfalls*[1]. Furthermore, historical rainfall data from a heavy rain event registered in Coimbra, on June 9, 2006 was also used.

The usage of rainfall events from 5-year and 20-year return period rainfalls is related with the fact that, per current legislation, urban drainage systems need to be built so that they do not overflow when subject to rainfall phenomena equivalent up to 20-year return period rainfalls. Therefore, when a new drainage system is being designed, Civil engineers make use of these sources to assess the system's resistance to this events.

A graphical representation of the 5-year return period and the June 9, 2006 rainfall events is presented in figures B.1 and B.2, respectively. Regarding more intense rainfall events, such as 20-year and 100-year return period rainfalls, their shape is very similar to that of the 5-year return period, only with higher precipitation intensity values registered. For this reason a decision was made to only include in this document a plot of the 5-year return period rainfall.

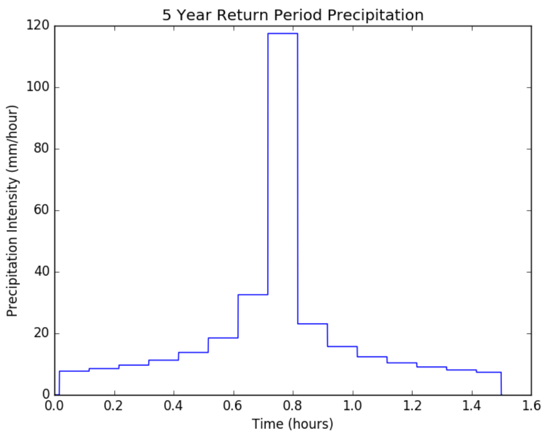

FIGURE B.1: Plot of a typical 5-year return period rainfall for the city of Coimbra.

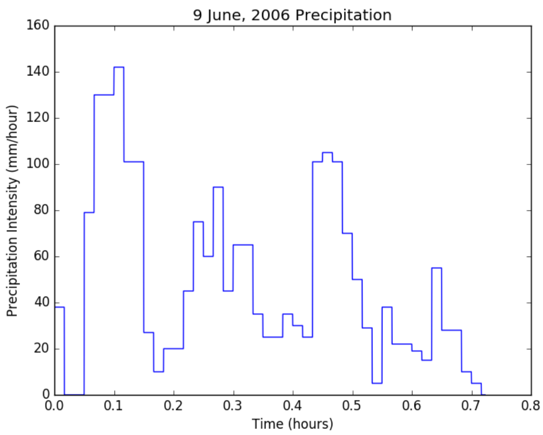

FIGURE B.2: Plot of the heavy rain that took place in Coimbra on June 9, 2006.

[1]That is rainfall events that, on average, only occur once every 5, 20 years or 100 years, respectively.

The remainder of this chapter is organised into a series of sections, in which each group of experiments will be detailed. The analysis is intended to evolve from simpler scenarios to more complex ones, highlighting the most relevant results and conclusions and their implications on the path pursed in this work.

B.1 Experiment 1

The scheme of the first scenario to be studied in this work is presented in figure B.3. In order to keep the scenario, and consequently the developed hydraulic model, as simple as possible a single road was represented and studied. Furthermore, only a faint slope was considered, small enough to assure water drain through gravity.

As suggested by the figure, the developed hydraulic model is a dual model, as it considers both surface and subsurface components of the drainage system.

Both surface and subsurface components are divided into four sections, each with a 30-meter length. The communication between both components in each section in ensured with the addiction of a series of outlets. Since in the simulation software used in this work these structure only allow for unidirectional communication, a pair of outlets was added in each section, to allow water flow in both directions.

The height difference between the two components was set to 5 meters and a constant slope of around 0.3% was defined. In the subsurface component an initial height of 0.4 meters was chosen for the underground conduits and a barrier was placed at the end of the second section. In the modelling and simulation software used in this work such a barrier is represented by a side orifice, already introduced in 4.2.6.3.

In the surface component a subcatchment structure was added to collect precipitated water, directing it to the surface of the street, where part of it would be forwarded to the system[2]. In the subsurface component such a structure was also added but with a different purpose. In this case the goal of the subcatchment structure was to represent water inflow from upstream sections of the drainage system, which were not represented in this scheme.

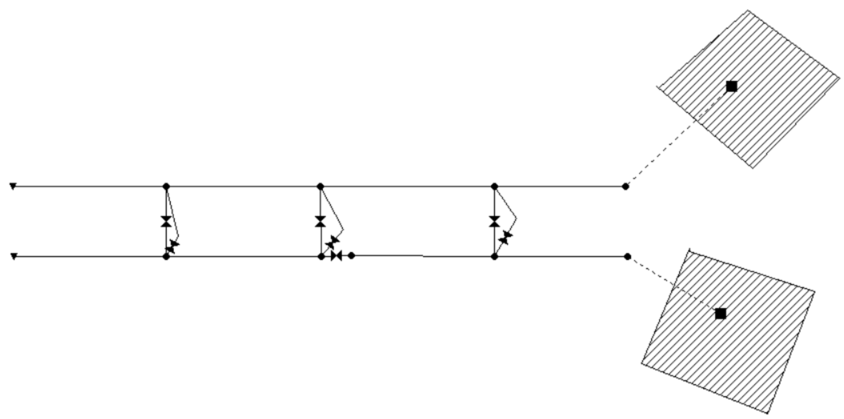

FIGURE B.3: Scheme of the hydraulic model of the first studied scenario.

Before attempting any control action on the installed orifice the presented scenario was simulated with the rainfall events of June 9, 2006 in order to assess the *standard response* of the system to a severe rainfall event. In these experiments the following outlet names were adopted when identifying the outlets in each section:

Outlets *L-1* and *L-2* were placed in the connection between the first two sections. Outlets *L-3* and *L-4* were in placed in the connection between the second and third sections. Finally, outlets

[2]Through its communication with the underground component, represented by the vertical outlets in the figure.

L-5 and *L-6* were in placed in the connection between the third and fourth sections. All outlets with an odd number assured water flow from the underground to the surface, while outlets with even numbers assured the flow from the surface to the underground conduits.

Analysing the results of this experiment one can conclude that the system fails to drain all the water that reaches it, as there are periods in time in which an upward flow of water through outlet *L-1* is registered, presented in figure B.4. Furthermore, even though a graphical representation was not included in this document, all underground conduits are overloaded during almost all the simulation time.

When facing this situation it is not possible to remove the upward flow of water by manipulating the orifice placed in the conduits. In fact, even if the orifice was placed in the first section this phenomena would still occur, as it is located upstream of all possible locations for the orifice.

In an attempt to promote upward flows in downstream sections of the system, so that it would be possible to attempt to mitigate them by actuating on the orifice, the height of the conduits in the first two sections was doubled, while structures in the last two sections remained unchanged. Performing the simulation with the same historical rainfall data the system behaved exactly in the same way, promoting upward water flows in the first group of outlets, as presented in figure B.5.

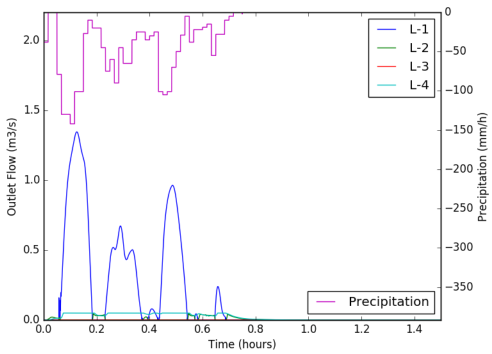

FIGURE B.4: Outlet Flow for rainfall events of June 9, 2006.

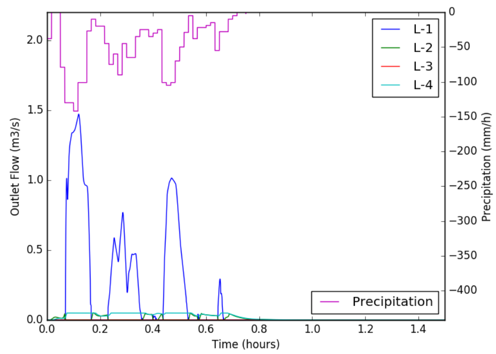

FIGURE B.5: Outlet Flow for rainfall events of June 9, 2006 when doubling the height of the first conduits.

Indeed, one can conclude that an orifice can only prove useful if this upward flow occurs downstream of its location, allowing it to be used to reduce the flow of water that reaches this area. The fact that this system proved unable to retain water from the rainfall events of June 9, 2006 it made no sense to test it with precipitation data from more intensive events, such as the return period rainfall (even the 5-year return period).

Moreover, studying the theory that governs the mechanics of these fluids, it was possible to found that even if a certain set of conduits is overloaded there is no guarantee that a upward flow will be witnessed. For this to occur the energy of the water circulating in depth has to be superior to that of the water circulating at surface and sufficiently high to overcome the difference in height between the two recorded locations.

This concept is referenced in the literature as *hydraulic head* [92] and its computation depends on the height of the conduit or channel in which the water is flowing, on its flow speed and its pressure. In fact, analysing the scenario under study, it was found that in the downstream sections the hydraulic head in the underground pipes is not enough to promote upward water flows.

Therefore it was concluded that the simplest and most effective way of promoting these upward flows would be to increase the flow speed in downstream sections, which could be achieved by the introduction of a new section with a greater slope. This new scenario was subject of study in the second set of performed experiences, to be detailed in the section that follows.

B.2 Experiment 2

Analysing what has been said in the previous section of this chapter modifications to the original scheme of the hydraulic model were performed in order to increase the flow speed in the downstream sections. Taking into account that water flow in these drainage systems occurs by gravity it comes as no surprise that a simple way of increasing the flow speed is to increase the underground conduits' slope.

Therefore, instead of considering a constant slope of around 0.3% in all sections of the underground component of the drainage system, a new section was added between the second and third sections, with a slope of about 12%.

Additionally, a section upstream to the orifice was also added in order to increase the available storage space. The developed system then consists of six sections, with the first three and the last two featuring a constant slope of around 0.3%, while the fourth possess a steeper slope, with values rounding 12%.

Finally, in an attempt to facilitate the control task to be performed the subcatchment connected to the surface component of the system was also removed. Thus, in this new scenario only one subcatchment can be found, in which precipitated water is routed to the subsurface component of the drainage system.

In figure B.6 a scheme of the hydraulic model in this new scenario is presented.

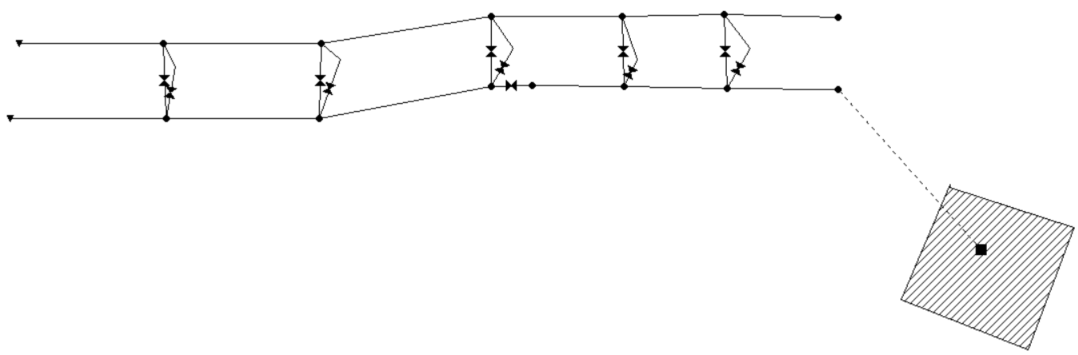

FIGURE B.6: Scheme of the hydraulic model developed for the second studied scenario.

Similarly to the previous experiment a naming convention was adopted when identifying outlets, conduits and junctions in each section:

Outlets *L-1* and *L-2* were assigned to the first section, *L-3* and *L-4* were assigned to the second section, and so on. Starting on the rightmost part of the subsurface component to its leftmost part, following the same logic for the surface component of the system conduits and junctions were named starting with *C-1* and *J-1*, ending with *C-12* and *J-13*, respectively.

Initially, the rainfall event of June 9, 2006 was used as the precipitation source for the simulations executed with this scheme.

In the performed experiments no *real* ascending water flow was registered in the outlets. In fact, in two distinct moments an ascending water flow was registered in outlet *L-7* (located in section 5, right after the sharp slope). However, as can be seen in figure B.7, such values were only registered in distinct and isolated moments, which may constitute residual simulation values, not necessarily meaning the existence of an ascending flow in this outlet.

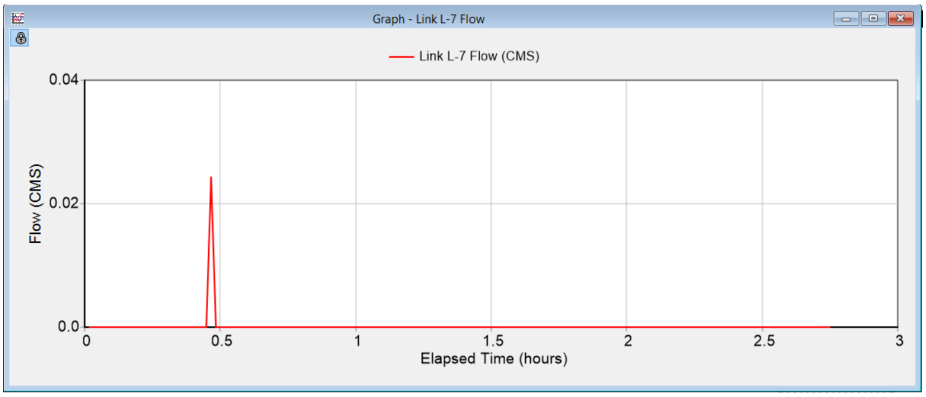

FIGURE B.7: Outlet Flow for Outlet *L-7* for rainfall events of June 9, 2006.

These results suggest that, even promoting an increase in the flow speed as a means of increasing the hydraulic head, such increment is not sufficiently high enough to overcome the difference in height between the surface and subsurface components of the drainage system at the section in question.

Furthermore, these results are supported by the plot in figure B.8. It is important to note that, in this plot, node *J-6* (represented in blue) corresponds to the junction in the subsurface component of the penultimate section of the system, and the node *J-12* (represented in red) is the corresponding junction in the surface component of the system.

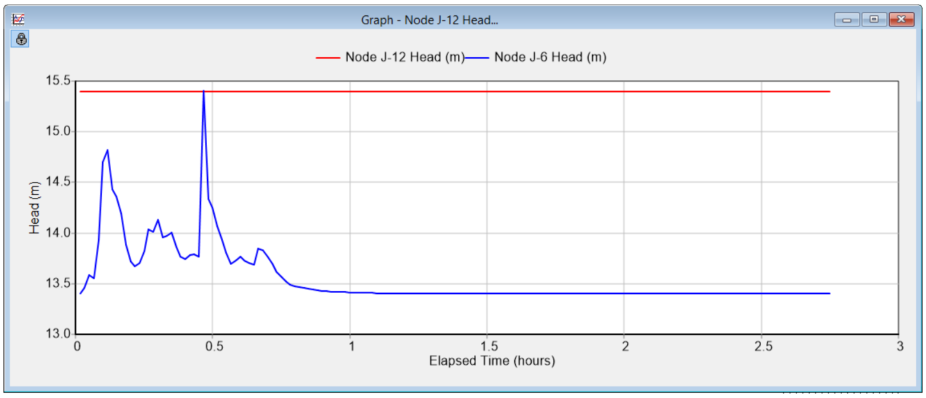

FIGURE B.8: Evolution of the hydraulic head in the surface and subsurface junctions in the fifth section of the drainage system.

As can be seen, the values of the hydraulic head for the two nodes have the same value only for a simulation time slightly below 0.5 hours, at which moment the residual flow values can be measured. During the remainder of the simulation the hydraulic head in the surface node is always higher than the hydraulic head in the subsurface node.

Considering that similar results were also achieved when using the 5-year and 20-year return period precipitation data, the presented scenario was also simulated using data from 100-year return period rainfall events.

As could be expected, submitting the system to more intense rainfall events lead to an increase in the flow measured at outlet *L-7*.

Additionally a new behaviour was witnessed: as a consequence of the increase in precipitation, the level in the conduits and junctions in upstream sections of the subsurface component of the system promoted an ascending flow in outlets *L-1* and *L-3*, although the measured flow in outlet *L-3* is momentarily, which probably is only measured due to a simulation error.

Figures B.9 and B.10 present the measured flows in the mentioned outlets.

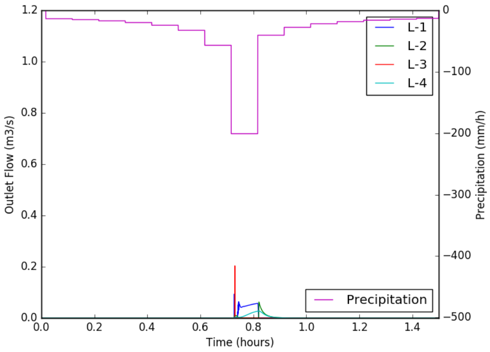

FIGURE B.9: Outlet Flow for rainfall events of a 100-years return period precipitation, for outfalls *L-1*, *L-2* and *L-3*.

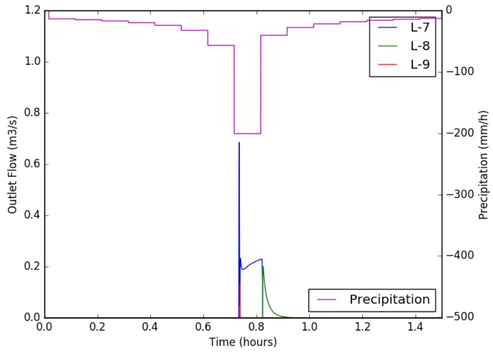

FIGURE B.10: Outlet Flow for rainfall events of a 100-years return period precipitation, for outfalls *L-7*, *L-8* and *L-9*.

In light of the results obtained and presented, yet another unsatisfying outcome was achieved. Even though introducing a significant slope in the scenario provided a boost in the hydraulic head in downstream junctions of the system, it proved insufficient to recreate the desired scenario for control: ascending flow in downstream junctions and outlets of the subsurface component of the system while there was still space available in conduits and junctions upstream of the orifice.

Even when considering variations of the proposed scenario, in which different area values were assigned to the subcatchment draining water into the system, this goal could not be achieved.

In fact, in all performed experiments with this scenario when an ascending flow of water was measured in downstream outlets of the system only one of two things was happening: either upstream outlets were also experiencing ascending flows, or their corresponding conduits and junctions were almost full, meaning that any control operation performed in the orifice would raise water levels in those structures, possibly promoting its rise to surface levels and consequent flood.

Nevertheless, and in order to acquire more sensitivity and experience when manipulating orifices in underground conduits some experiences with this scenario were performed. In these experiments, even when no downstream ascending flow was being registered the orifice was operated in some way.

In one of those experiments a constant precipitation of 15 mm/hour, with a total duration of 90 minutes, was used. Initially, as already performed in other scenarios, the developed model was simulated with no actuation in the barriers. As can be seen in figures B.11 and B.12, water levels in conduits and junctions upstream of the orifice remained close to 0.2 m.

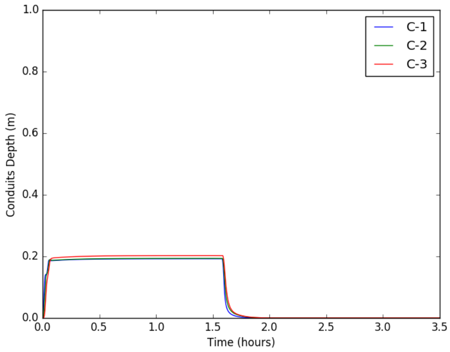

FIGURE B.11: Water level in conduits upstream of the orifice for a constant precipitation of 15 mm/hour, lasting for 90 minutes.

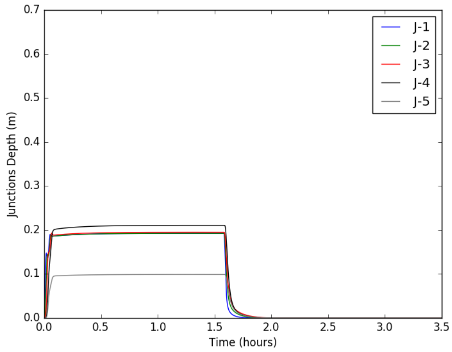

FIGURE B.12: Water level in junctions upstream and immediately downstream of the orifice for a constant precipitation of 15 mm/hour, lasting for 90 minutes.

When manipulating the orifice's opening it is important to understand what happens when an action is triggered to reduce its setting. Some insight in this matter has been provided in section 4.2.6.3.

Therefore, for very low water levels in the orifice, small reductions to its default setting will have no effect in the amount of water that flows through it, as its opening is still above water level. In order to produce any effect in the flow of water through the orifice it is necessary to drastically reduce its setting.

In this sense, the orifice setting was reduced to 20% of its capacity during the rainfall event. As a result, water levels in the conduits and junctions upstream of the orifice rose somewhat rapidly (considering its relatively low level and precipitation intensity, when compared to more aggressive scenarios), as shown by figures B.13 and B.14. However, in this experiment no major differences were found in water levels in the conduits and junctions downstream of the orifice.

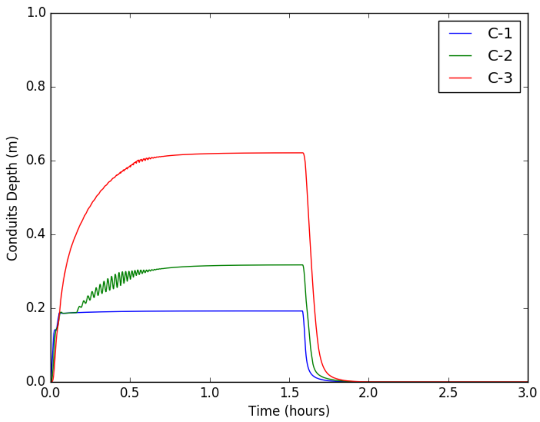

FIGURE B.13: Water level in conduits upstream of the orifice after the changes in its setting.

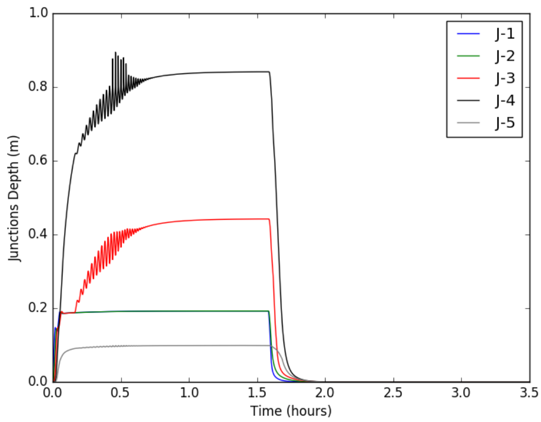

FIGURE B.14: Water level in junctions upstream and immediately downstream of the orifice after the changes in its setting.

Further experiments were conducted, in which different options for the type of actuation in the orifice, its duration and the period of time during which the orifice's setting remained unchanged after an actuation were tested.

As an example, approaches seeking to maintain a reference depth or flow value in a conduit downstream of the orifice were tested. When testing these approaches different actuation speeds in the orifice were considered (that is, different rates at which the orifice's setting was increased or decreased). Typically, the baseline actuation speed for these experiments was *1 m/min*, with faster and slower variations being considered (in the worst case instant actuation speeds were adopted, and in the opposite side values in the order of magnitude of *1 dm/min* were also tested).

Further experiments updating the system's state and the actuation in the orifice at different frequencies were also conducted. As it could be expected, these experiments featured scenarios in which this analysis was performed at every simulation step and scenarios in which the orifice setting, once changed, remained constant for a given time period. In this last scenario the orifice's setting remained constant for periods that ranged from several minutes (half an hour in the worst case) to only a few seconds.

In these tests it was found that when the orifice setting remained unchanged for a certain period of time, after an actuation had been made, potentially worst consequences could occur, especially during more intense precipitation events.

The reason for this is that, when the orifice's opening is reduced if the amount of water that flows through it is smaller than the amount of water that arrives at it, then an excess water volume will be stored in the upstream conduits and junctions. If this volume is sufficiently high then an ascending flow of water upstream of the orifice may occur.

In figures B.15 and B.16 the flow in the conduits upstream of the orifice before and after the changes in its setting are represented. When the orifice's setting changes the flow in its immediate upstream conduit suffers considerable variations. In some moments the flow in this conduit is decreased in half, when compared to its *normal* behaviour. Thus, these are the moments in which excess water volumes need to be stored in the conduits, justifying the registered increase in their water levels, presented in figures B.13 and B.14.

Naturally, if the orifice setting is only updated in certain periods of time this phenomena may occur without the adoption of any action to mitigate it.

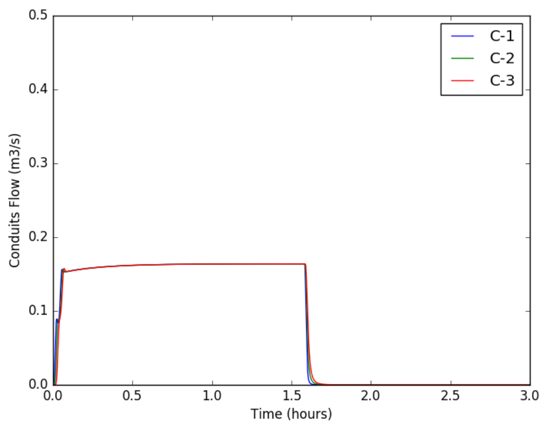

FIGURE B.15: Water flow in conduits upstream of the orifice when no changes in its setting are being performed.

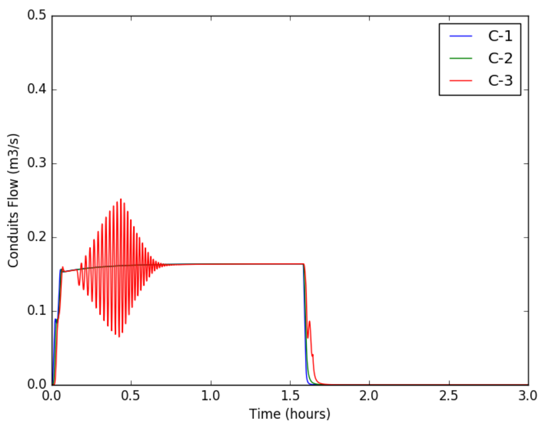

FIGURE B.16: Water flow in conduits upstream of the orifice after the changes in its setting.

An obvious attempt to solve this problem is to increase the frequency in which the orifice's setting is updated and to perform changes in its setting as smooth as possible. Unfortunately, in some scenarios, even the slight reduction to the orifice setting lead to a sudden rise in water level in the conduits and junctions upstream of the orifice. Even though this is more noticeable in simulations with more intense rainfall events, it is an important factor that affects the handling of the orifice and its success in mitigating the effects of a flood.

B.3 Experiment 3

With the results of previous experience problems and difficulties encountered while trying to retain water in pipelines by means of a manipulation of an orifice became evident. As supported by figures B.13 and B.14, even when only a small fraction of the water flowing through the system is retained its level tends to rise very fast in locations immediately upstream of the orifice.

In the scenario presented and studied in the previous section this effect was also strengthened by the fact that only one water entry point was being considered in the system. Indeed, in a real scenario water reaches these systems through multiple entry points arranged along its surface component.

However, when only one entry point into the underground drainage system is being considered for simulation purposes, it has to be representative of the full extent of the terrain being studied. Thus, in the scenarios studied so far it implies that the volume of water drained to the system through that point needs to be considerably high, which may condition all control operations.

It was based on this reasoning that the scenario presented in figure B.17 was created and proposed in the initial study carried out in this work.

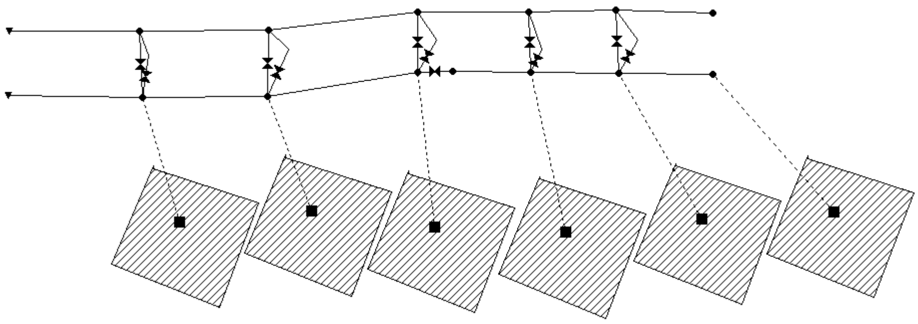

FIGURE B.17: Scheme of the hydraulic model developed for the third studied scenario.

In this scenario each section in which the system was composed possesses an entry point through which water can drain and enter in the subsurface component of the system, as in a real system.

The main goal of the set of experiments presented in this section is to investigate if introducing water into the system in a more gentle way (and distributed throughout a series of entry points, instead of concentrating it in a single entry point) can ease the control process and water retention capabilities of underground conduits.

However it is noteworthy that, from the standpoint of the development of hydraulic models, this approach is much more demanding than previous ones. The main reason for this is that a larger number of subcatchments is being considered, whose attributes need to be properly adjusted. This can turn out to be a rather time consuming and demanding process, particularly as the number of subcatchments increases. For this reason, considering only one water entry point, or a slight higher number of entry points when modelling very large and extensive systems, is a solution widely used by many members of the scientific community in this field.

Nevertheless, the model presented in figure B.17 was used in an initial setup, along with the precipitation data source of the rainfalls of June 9, 2006. Figures B.18 and B.19 present water levels in upstream and downstream conduits for this simulation.

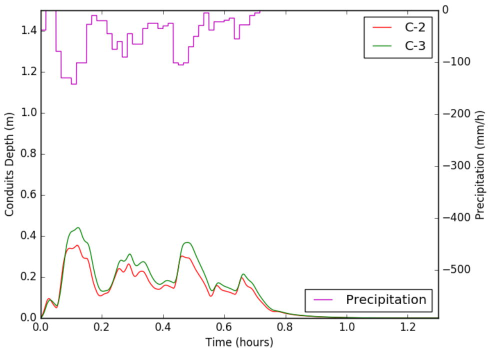

FIGURE B.18: Water level in upstream conduits for the rainfall events of June 9, 2006.

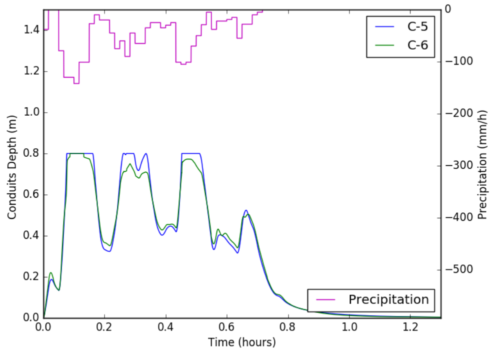

FIGURE B.19: Water level in downstream conduits for the rainfall events of June 9, 2006.

As can be seen in both figures, while upstream conduits still have a lot of free space to store water, downstream conduits are easily overloaded whenever a precipitation peak is registered.

Analysing the first peak in downstream conduits' levels, further experiments were conducted to attempt to retain as much water as possible in upstream conduits, in order to reduce the overload situation. Considering that this peak is registered from 0.07 hours of simulation up to 0.16 hours, the tested rules sought to actuate right at the start of the simulation up until about 0.2 hours of simulation (thus covering the time window in which the peak is observed).

Even when severely promoting water retention at upstream conduits, which was accomplished by keeping the orifice setting with a value of 0.1 throughout the mentioned actuation time window, no considerable level reduction was accomplished in downstream sections of the system. As presented in figures B.20 and B.21, upstream conduits became overloaded, promoting ascending flows of water in those sections, while only slight level reductions were observed in the second of the two downstream conduits.

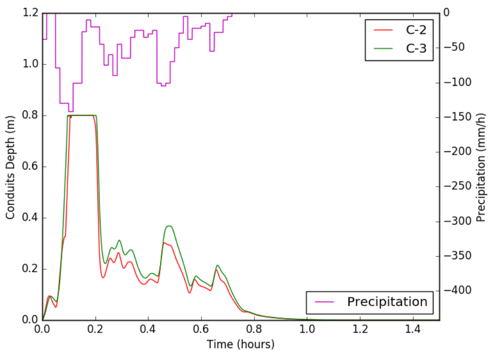

FIGURE B.20: Water level in upstream conduits for the rainfall events of June 9, 2006 when aggressive control actions were being used.

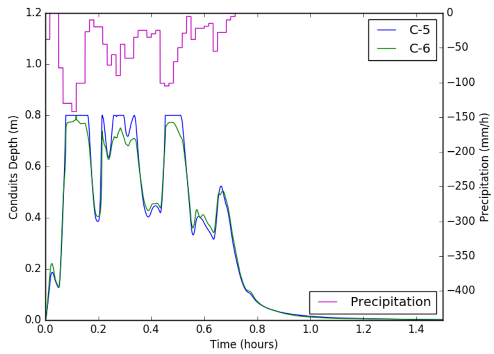

FIGURE B.21: Water level in downstream conduits for the rainfall events of June 9, 2006 when aggressive control actions were being used.

Even with the less aggressive return period rainfall data used in this work, the 5-year return period precipitation, similar results were achieved.

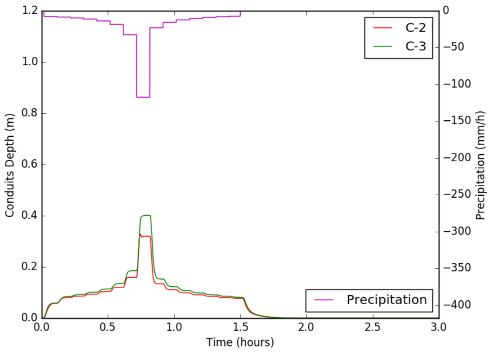

FIGURE B.22: Water level in upstream conduits for the 5-year return period rainfall events.

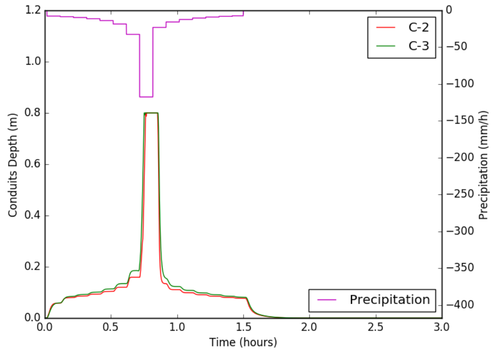

FIGURE B.23: Water level in upstream conduits for the 5-year return period rainfall events when aggressive control actions were being used.

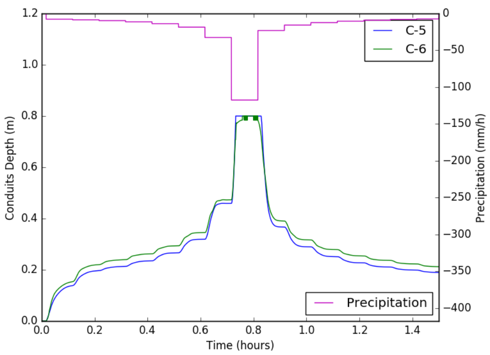

FIGURE B.24: Water level in downstream conduits for the 5-year return period rainfall events.

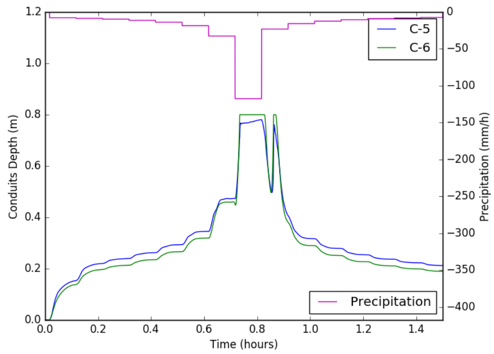

FIGURE B.25: Water level in downstream conduits for the 5-year return period rainfall events when aggressive control actions were being used.

Analysing the presented experiments and their respective results a general idea emerges, which defends that whenever many entry points are being considered in the system, a tremendous retention would have to be performed in upstream conduits in order to assure a drastic reduction in water levels at downstream conduits.

However, the overall scenario may not be as dramatic as the previous paragraph may have drawn it. In fact, it is important to point out that water enters the system through junctions, connected to subcatchments. As presented in 4.2.4, one of the characteristics of a subcatchment is its area, whether it is pervious or impervious. If a given subcatchment is created with a greater area than another, and all other characteristics are the same, then the first one will drain more water than the second one (obviously, when submitted to the same precipitation).

Thus, a possible way of turning this scenario into something closer to what is desired in this work is to manipulate the subcatchment's areas (within a reasonable limit) in order to reduce inflows downstream of the orifice.

Later in this chapter, namely in section B.5, a scenario inspired by these ideas will be presented and further studied.

B.4 Experiment 4

With the previously mentioned experiments the difficulty in controlling a given water flow in a very small and restricted area, such as an underground conduit, became clearer and clearer. After some reflection and analysis of the situation that is being created it seemed that, in order to witness an ascending flow of water in downstream sections, a significant water flow would have to be drained to the system in its upstream sections and accelerated during its flow.

The main problem that this approach seems to have is that, because significant flows are being drained to the system and the storage capacity of its earlier sections is quite small, considerable water retention in upstream conduits and junctions is only possible if these structures become so overloaded that, instead of witnessing ascending flows in downstream sections of the system, they are witnessed upstream of the orifice.

Thus, the next hypothesis tested in this work aimed to explore an alternative scenario, similar to the one presented in figure B.6. The main modification performed in this scenario relies on the fact that instead of having only one water entry point in the system, a secondary inflow is being considering, linked to the system in the beginning of its forth section. This inflow could represent a derivation from another part of the drainage system that would transport a given amount of water into the system being represented and modelled.

Figure B.26 presents the mentioned scenario.

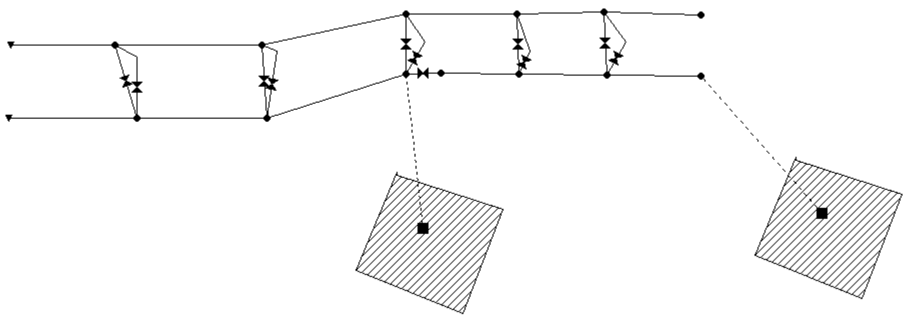

FIGURE B.26: Scheme of the hydraulic model in the fourth studied scenario.

As in previous scenarios, the adopted naming convention remains the same: names for underground conduits and junctions start with *C-1* and *J-1* on the rightmost side, ending with *C-6* and *J-7*. Names for surface conduits and junctions start with *C-7* and *J-8*, finishing with *C-11* and *J-13*. Finally, existing outlets start with *L-1* and *L-2*, also on the rightmost side, and finish with *L-10*. Again, odd outlets assure for ascending flow, while descending flow occurs in even ones.

Regarding the dimensions of the structures considered in this scenario, all underground conduits were given a height of $0.8m$ and a length of $40m$. Their respective junctions were given a depth value of $2m$.

The main motivation and hope with this approach was that by adding a second inflow that only affects the last two sections of the system, one would be able to obtain a *more controllable* scenario, in the sense that it would be possible to reduce the amount of water that was being drained to initial sections of the system, while keeping an ascending flow of water in its downstream sections, thanks to this additional inflow.

The two water inflows would have to be coordinated in such a way that by actuating on the orifice, and thus retaining some of the inflow in the upstream conduits and junctions, the remaining water flow that reaches downstream sections would not have enough hydraulic head to reach surface levels.

In order to assess the feasibility of this idea some experiments were conducted in the presented scenario. Considering that water retention would be performed in upstream conduits, initial tests featured less aggressive rainfall events as the precipitation data source for the subcatchment connected to the first junction of the drainage system. In an attempt to keep all interactions in the system as simple as possible a constant precipitation event was used as the incoming flow's rainfall data, with a total duration of 90 minutes.

In further exercises, more aggressive rainfall data were used as the main precipitation's data source, consisting of the smoother return period rainfall, of the three presented previously in this document.

The remainder of the current section is thus divided in three subsections. The first two are reserved to the two main groups of experiments performed with this scenario, while in the last one the main conclusions and ideas withdrawn from this experiment are presented.

B.4.1 Simulations With June 9, 2006 Rainfall

Of all the historical rainfall data collected and used in this work, the precipitation intensity dataset from the heavy rain witnessed in Coimbra on June 9, 2006 appears as the less aggressive rainfall data for the drainage systems used in this work[3]. For this reason, the mentioned dataset was selected as the starting point of these experiments, as it would use a smaller percentage of upstream conduits' available volume. In figures B.27 and B.28 water level in conduits immediately upstream of the orifice and in downstream conduits is presented, for simulations featuring only the rainfall events of June 9, 2006 (without taking into account any additional inflow).

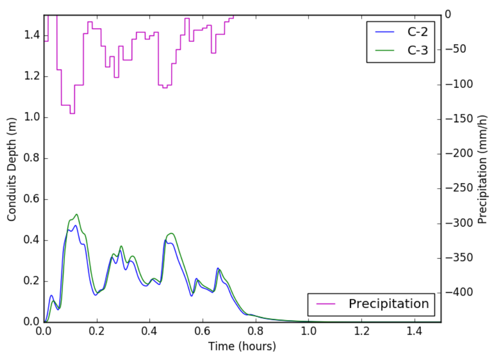

FIGURE B.27: Water level in conduits immediately upstream of the orifice, for a simulation with the rainfall events of June 9, 2006.

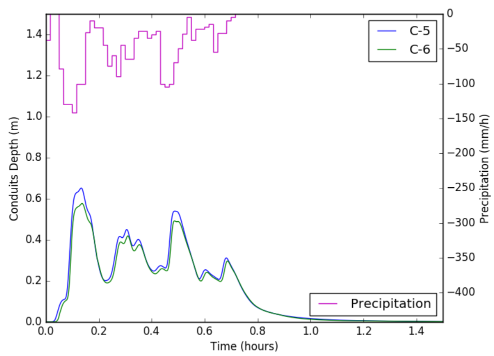

FIGURE B.28: Water level in downstream conduits, for a simulation with the rainfall events of June 9, 2006.

Before performing any simulation featuring the mentioned incoming flow, it was crucial to determine whether or not the current scenario allowed for any kind of upstream retention when simulated with the selected precipitation dataset.

Although it was not possible to attenuate too much the effects of the first peak of precipitation, with the rules developed and tested for this purpose it was possible to reduce water levels in downstream conduits during the second precipitation peak, which took place around the 0.5 hours of simulation. However, to avoid extending the current discussion too much further details about such experiments will not be covered in detail in this subsection.

Regarding the water flow to be further added to the system, or the *incoming flow* as more often used for convenience, a series of different precipitation time series were tested, each with higher

[3] Indeed, the fact that this event is characterised by strong variations, both positive and negative, of measured precipitation intensity poses a lesser threat to the system, since it allows following a period of heavy precipitation of any accumulated water to be drained through it.

intensity values, until one was found which allowed for an ascending flow in downstream outlets. Of all the different and successively bigger values tested, ascending water flows in downstream outlets were only registered for constant incoming flows of $100mm/h$.

Indeed, this incoming flow value came with some surprise considering that, as presented in figure B.28, when only the rainfall of June 9 is being considered water levels in downstream conduits already surpass half its capacity.

When this incoming flow is also taken into account, water levels in these conduits increases, although not as much as expected. Only with constant flows of $100mm/h$ is it possible to overload downstream conduits long enough to witness an ascending flow of water in outlet *L-7*, as presented in figures B.29 and B.30.

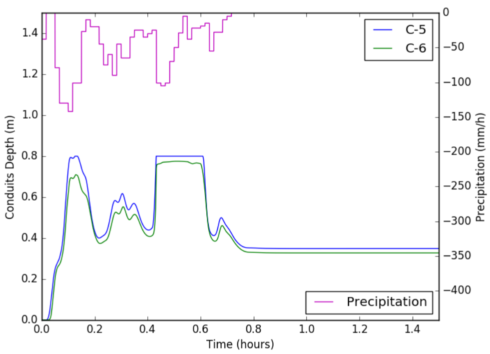

FIGURE B.29: Water level in downstream conduits when incoming flow of $100mm/h$ is also considered.

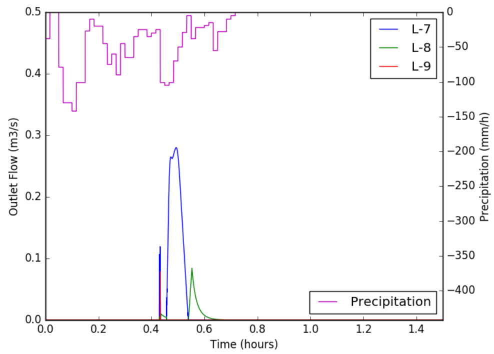

FIGURE B.30: Outlet flow in downstream outlets when incoming flow of $100mm/h$ is also considered.

Analysing the mentioned value of $100mm/h$, it is important to point out that this is tremendously high value and, as shown in B.30, it only leads to an ascending flow half an hour after being continuously introduced in the system. In fact, this value suggests that the idea and motivation behind the current scenario may prove infeasible, as tremendous inflows of water seem to be required to promote ascending water flows in downstream sections of the system, in the conditions of the current experiment.

Nevertheless, some control rules that could retain as much water as possible in upstream sections of the system were sought. In order to increase their performance, time periods in which low precipitation intensities were measured were used to promote water drain through the system. However, in all performed tests no significant reduction in the ascending water flows were obtained[4].

In figures B.31 and B.32 water levels in upstream and downstream conduits are shown for one of the performed tests.

As can be seen, even though there are some periods in which upstream conduits are at full capacity, in one of those (around 0.5 hours of simulation) downstream conduits still remain overloaded.

[4]Even though it was not stated, in these experiments situations in which ascending water flows in sections upstream of the orifice as a result of its control actions were discarded for obvious reasons.

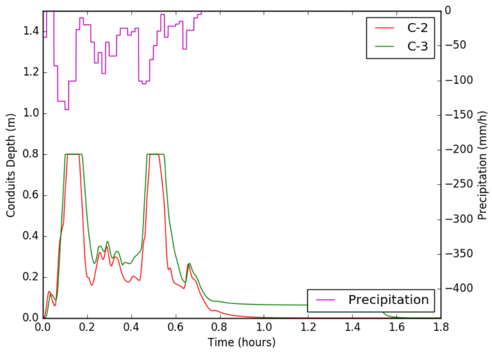

FIGURE B.31: Water level in upstream conduits (*C-2* and *C-3*).

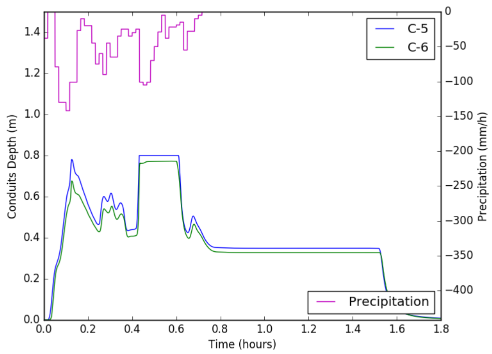

FIGURE B.32: Water level in downstream conduits (*C-5* and *C-6*).

The corresponding actuation history for the orifice is presented in figure B.33.

As shown in this subsection the initial idea of using rainfall events less aggressive to the system appeared to suggest that larger water volumes would be retained in upstream conduits of the system, thus increasing the chances of actually making an impact in downstream water levels.

However, because less aggressive water flows were being drained to the system, the only way of promoting the rise of water in downstream sections was to introduce a tremendous amount of water in a later section of the system. By doing so the verified ascending flow occurs much more as a result of this incoming flow than of the precipitation inflow, which dooms the success of any control actions performed in upstream sections of the system.

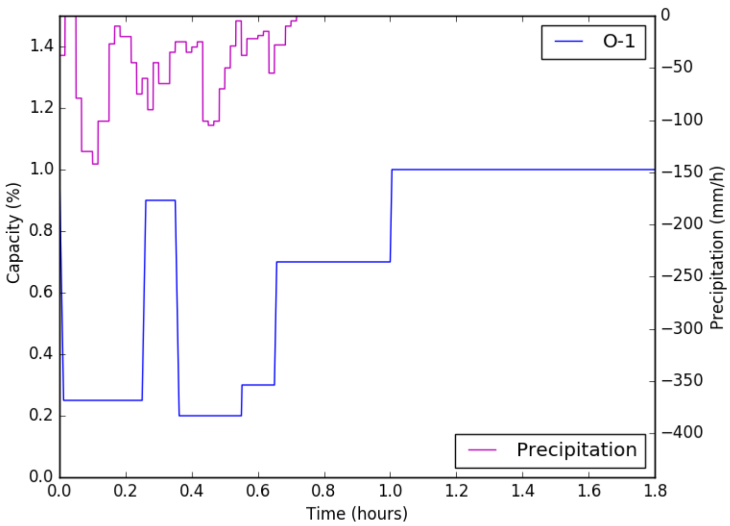

FIGURE B.33: Actuation history on the orifice.

B.4.2 Simulations With Return Period Rainfall Events

As shown in the previous subsection, smoother rainfall events proved to be unsuccessful in producing a controllable scenario as they require a tremendous incoming flow to be considered in order to assure an ascending flow in downstream sections, thus dooming all prospects of success of any control actions.

In light of these results, more aggressive rainfall events were considered, hoping that they would allow to reduce the intensity of the incoming flows while allowing for considerable water retentions in upstream sections of the system.

Initially the 5-year return period rainfall data was used as the main precipitation source in this scenario. Successively higher constant incoming flows were tested until one was found that could promote ascending flows of water in downstream sections of the system. Similarly to the situation described in the previous subsection only extremely high incoming flows were able to overload downstream conduits and promote water pressure flows to the surface, making useless all efforts to retain water in upstream sections of the system.

Attempting to enable ascending flows in downstream sections of the system with incoming flows of smaller intensity, experiments were also conducted using the 20-year return period rainfall data as the main precipitation source. Like in previous experiments, before performing additional tests - that is, before adding the incoming flow downstream of the orifice - some control rules for the orifice were developed and tested in order to verify whether or not the amount of water that could be stored in upstream conduits was enough to have an impact in downstream water levels.

It was found out that, during more intensive precipitation periods, considerable reductions in downstream levels were only achieved by performing significant restrictions to the orifice's setting (with values around 0.2).

However, when the orifice's setting was reduced to such small values, upstream conduits were not able to retain all excess water. By relaxing the orifice's setting those upstream ascending flows were removed, but again at the cost of not being able to retain enough water in upstream conduits to have repercussions in downstream levels. These conclusions are supported by figures B.34, B.35 and B.36.

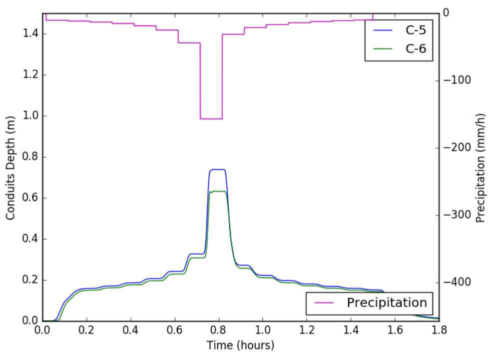

FIGURE B.34: Water level in downstream conduits while no control actions are being performed.

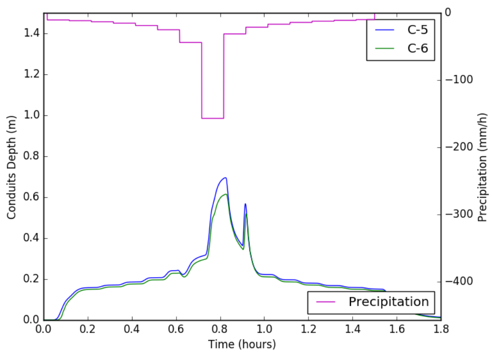

FIGURE B.35: Water level in downstream conduits while water retention is being made in upstream conduits.

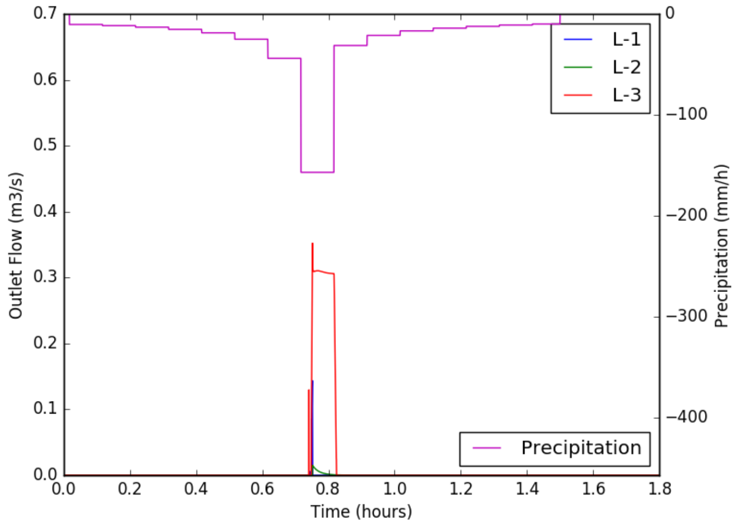

FIGURE B.36: Flow in upstream outlets.

As these results point out, controlling water flows in this scenario for more intensive rainfall events has proven unsuccessful.

B.4.3 Remarks

When this scenario was first idealised, its main motivation and hope was that the inclusion of a derivation that introduced water in the system could, together with that which was already in circulation, cause an overload of downstream conduits in such a way that by retaining some water before the derivation this overload situation could be avoided.

Unfortunately, this was not possible to reproduce, since a small amount of water being drained in the initial sections is required in order to allow some degree of actuation and control of the orifice's opening. But following this logic, there would only be an ascending flow of water downstream of the orifice if a large amount of water was transported by the second inflow. However, in this case the witnessed ascending flow was due to this additional volume of water added to the system, making it useless to act on the orifice, since it would have no effect on the final behaviour of the system.

Similarly, if a larger amount of water was introduced in the initial section of the system, as evidenced in the previous experiments, no feasible set of control actions on the orifice was found that allowed to reduce the flood events in the downstream sections, while keeping the upstream sections free of ascending flows to the surface.

In short, although the motivation and idea for this scenario were, in theory, interesting, its application to this work has proved to be useless since when water rose to surface levels in downstream sections the actuation on the orifice could not reduce its effects while keeping upstream sections free of any repercussions.

B.5 Experiment 5

In a last attempt to produce a situation where it would be possible to store a considerable volume of water in upstream sections of the system an alternative scenario was developed and tested, which will be presented in the current section. This scenario is somewhat similar to those presented in the last three experiments, with the difference that a considerable extension has been added to the system's initial section.

Furthermore, by considerably extending the initial sections it is also possible to explore the usage and the effects of more than one orifice when retaining water in upstream sections of the system, thus avoiding concentrating the storage of all excess volume in only a small set of conduits.

Thereby, three orifices were arranged along the initial sections of the system, as presented by figure B.37. Even though no subcatchment is shown in this scenario, an approach similar to the one presented in section B.3 was followed, in which multiple subcatchments and water entry points in the system were considered.

The reason why no subcatchment is visible has to do with the fact that, because of their number, the subcatchments were generated and added to the model in an automated way (that is, with the help of a script), without being manually drawn in *SWMM*. Nevertheless, although they are not visible, the subcatchments are represented in the model.

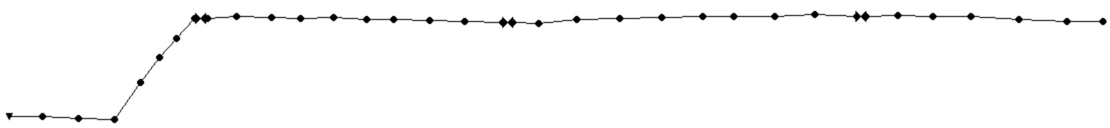

FIGURE B.37: Scheme of the hydraulic model in the fifth studied scenario.

With respect to the adopted naming conventions regarding orifices, junctions and conduits the following were considered: for conduits, starting from upstream to downstream sections the objects were given the names *C-<conduit_number>*. Orifices followed the opposite trend: starting from downstream to upstream with the names *O-<orifice_number>*. Regarding the junctions, a similar convention to the one followed for orifices was adopted, with the exception of the junctions immediately upstream of the orifices, which were the last ones to be added.

Since we have multiple orifices it is also important to analyse what is happening in the conduits and junctions upstream of them.

Therefore, regarding orifice *O-1* the junction directly upstream of it is junction *J-42*, which is connected to conduit *C-24*, and consequently junction *J-18* and conduit *C-23*. Regarding orifice *O-2* the junction directly upstream of it is junction *J-44*, which is connected to conduit *C-15*, and consequently junction *J-27* and conduit *C-14*. Finally, the junction directly upstream of orifice *O-3* is junction *J-43*, which is connected to conduit *C-6*, and consequently junction *J-36* and conduit *C-5*.

It is also important to point out that, in the current scenario, all the conduits were created with a length of 60 meters and a height of 1 meter, while the junctions in the upstream sections (where the orifices were placed) had a maximum depth of 1.5 meters, leaving the remainder with a maximum depth of 1 meter.

In order to assess if this scenario would, in fact, allow for a better water retention across its length, a series of tests involving the rainfall events of June 9, 2006 and the 5-year, 20-year and 100-year return period rainfalls were performed. Considering the extension of the tests and their consequent discussion, the remainder of this section is divided into multiple subsections, each reserved to the presentation and analysis of a particular test.

B.5.1 Baseline Experiments

The first set of experiments conducted in this scenario aimed to establish a baseline for all the remainder experiments. Therefore, simulations with the mentioned rainfall events were conducted, with no manipulation of the orifices' settings.

Taking into account the fact that, in the presented scenario, no surface component of the drainage system is included it is not possible to analyse any ascending water flows in the system's outlets. Therefore, instead of focusing the analysis on those structures, it will be redirected to water levels in the junctions of the system, with its total capacity being regarded as the system's limits.

Starting this exercise with the rainfall events of June 9, 2006, it was found that the system was not able to drain wall the incoming flows in its downstream sections. As supported by figure B.38, there are some moments in which the first downstream junction is full with water.

However, the same scenario is not replicated in the upstream sections of the system. As presented by figure B.39 upstream junctions and conduits are far from being at their full capacity, meaning that there is still space in the system to store excess water. In figure B.39 only junctions immediately prior to each orifice were included as these are the locations were water levels will be higher (namely when control actions are issued to the orifices). Therefore, water levels in these locations can be used as upper bounds to water levels in earlier conduits and junctions.

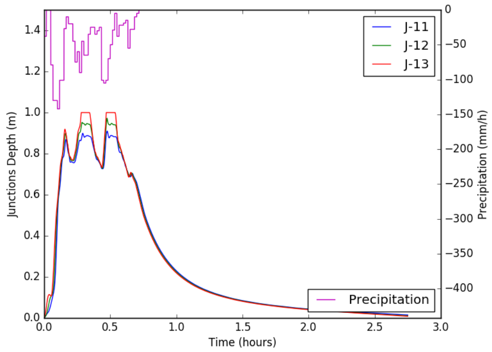

FIGURE B.38: Water level in the downstream junctions, for the rainfall events of June 9, 2006.

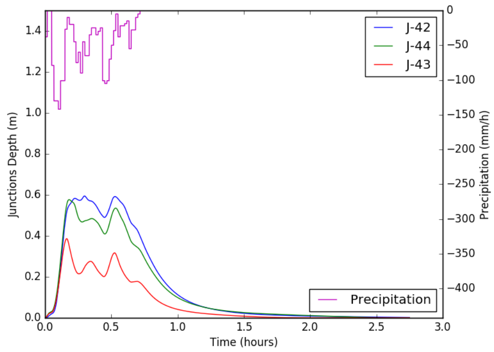

FIGURE B.39: Water level in the junctions immediately upstream of the orifices, for the rainfall events of June 9, 2006.

In fact, considering that water levels in upstream junctions and conduits of the drainage system are quite below their full capacity[5], at least in this scenario, there is some hope regarding the ability to retain water in upstream sections of the system so that this phenomena is controlled.

Moving on to more aggressive rainfall events, such as the 5-year return period precipitation, slight changes in the downstream junction levels have been observed, as presented by figure B.40.

[5]It is important to keep in mind that the conduits have a height of 1 meter and 60 meters of length, while the junctions have and additional 0.5 meters of height.

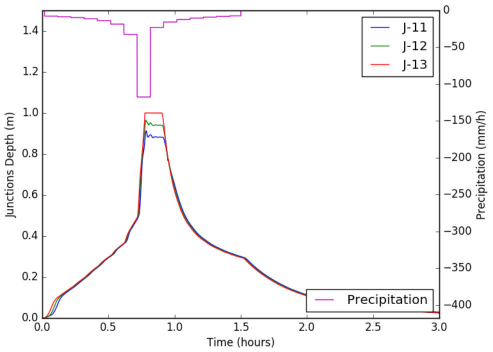

FIGURE B.40: Water level in the downstream junctions, for the rainfall events with a return period of 5 years.

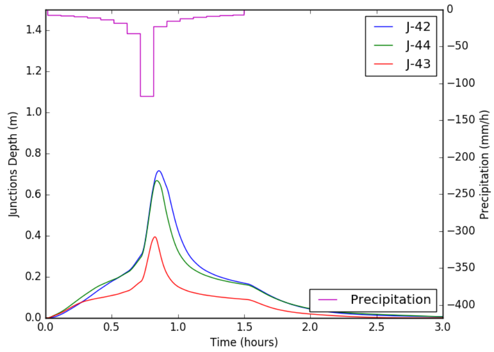

FIGURE B.41: Water level in the junctions immediately upstream of the orifices, for the rainfall events with a return period of 5 years.

Instead of three distinct peaks in registered water levels at downstream junctions, a constant rise is observed, occurring simultaneously with the increase in rainfall intensity. Furthermore, in the junctions directly upstream of the orifices registered water levels follow a similar trend, with higher values than those registered in the rainfall events of June 9, 2006.

As expected, as the rainfall intensity raises so does the width of the peaks in downstream junctions and the level in upstream junctions: for 20-year return period rainfalls, the maximum level registered for the upstream sections of the system was slightly below 0.9 meters, while for the 100-year return period an overflow in junction *J-43* was registered.

Considering the characteristics of the return period rainfall events and the observed state of the system when simulated with that precipitation data, this scenario was mainly tested with the rainfall events of June 9, 2006, attempting to develop a rule set capable of reproducing the desired behaviour.

Because of this, the remainder subsections will focus on different experiments performed with that precipitation data source. In these subsections the orifices will be explored in the *upstream-downstream* direction. That is, initially, only orifice *O-3* was the target of any actuation tasks, progressively actuating upon orifices *O-2* and *O-1*.

B.5.2 Controlling First Orifice

In these experiments, as already mentioned, only control actions regarding orifice *O-3* were issued. As this orifice is placed in an early section of the drainage system the main motivation and goal for these experiments was to retain as much water as possible, understanding how would it affect the overall behaviour of the system.

Considering that, under normal circumstances, water levels in *O-3* stand around 0.3 meters the value of *0.2* was initially set as the minimum allowed setting for the orifice. Further experiments with smaller lower bounds for the orifice setting were conducted, however, they required some adjustments to the orifice setting during the simulation, as they lead to significant level rises in junctions immediately upstream of the orifice.

In the conducted experiments, it was possible to attenuate the second peak observed for junction *J-13*, only with two simple control rules defined for orifice *O-3*:

Modelling its behaviour with a state machine, two distinct states were defined and considered for the orifice: an initial state where the orifice was fully opened, and another one where it was closed up to its limit, retaining as much water as possible in its upstream junctions and conduits.

The lower limit considered for the orifice setting was the already mentioned 0.2 boundary. The results of this actuation in the downstream junctions of the system are presented in figure B.42.

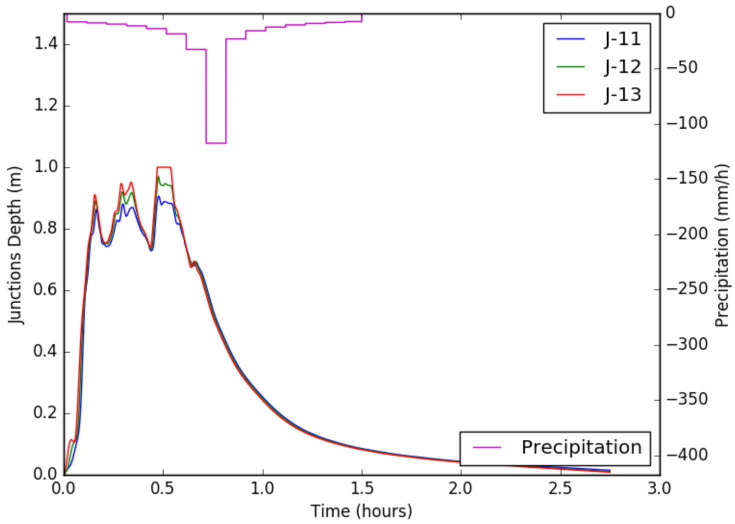

FIGURE B.42: Water level in downstream junctions when applying mentioned control actions in orifice *O-3*.

In figures B.43 and B.44 the machine state diagram for the orifice and its actuation history are, respectively, presented. It is important to state that, even though the actuation in this stage may appear instantaneous, all orifice setting changes were performed at a rate of $1m/min$.

In figure B.43 no verification routine of water levels in upstream junctions and conduits were included. However, this was a major and constant concern when defining and testing new rules, across all the experiments performed. Nevertheless, given the fact that, in this scenario, the actions performed on the orifice did not lead to an overflow of its upstream junctions and conduits, such verifications were not included in the presented diagram for simplicity reasons and to ease its understanding.

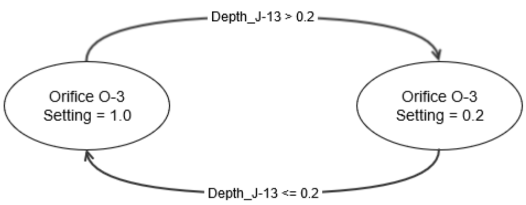

FIGURE B.43: Machine State diagram for the orifice *O-3*.

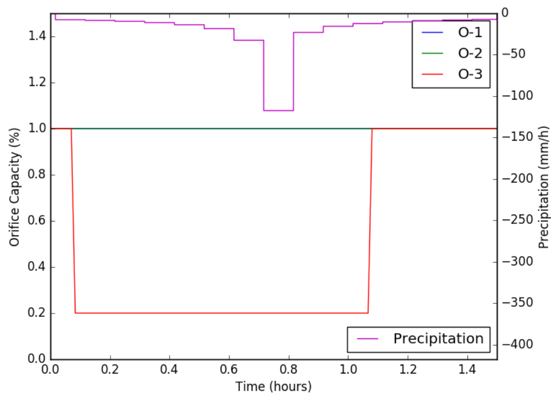

FIGURE B.44: Actuation history on the orifices.

B.5.3 Controlling Two Orifices

Moving on to controlling two orifices simultaneously, all the efforts of these experiments were focused on improving the results obtained in the previous experiment, by actuating on orifice O-2.

With this in mind, and considering that orifice *O-3* is not manipulating considerably high water volumes, the control rules defined for that orifice were maintained. Thus, in this experiment only orifice's *O-2* control rules were adapted.

Instead of following an approach similar to the one chosen for *O-3*, in these tests the rules that govern the operation mode of *O-2* were defined in terms of simulation time, taking advantage of future knowledge regarding precipitation and variations of water levels in the system.

The main idea of this approach was to, in an initial stage, develop rules that would allow us to achieve the desired results, without any concern regarding their feasibility in a real scenario. Once that was accomplished, an effort would be made to develop rules feasible to be applied in a real scenario.

In these experiments, the best results were achieved with the following rules governing the operation of *O-2*'s setting:

- If simulation time ∈ [0.1, 0.25] h, then change orifice's *O-2* setting to 0.25
- Otherwise, if simulation time ∈]0.25, 0.5] h, then change orifice's *O-2* setting to 0.3
- Otherwise, if simulation time ∈]0.5, 1.0] h, then change orifice's *O-2* setting to 0.4
- Otherwise, if simulation time > 1.0 h, then change orifice's *O-2* setting to 1.0.

At this point it is important to state that, just like in the previous subsection, all changes to the orifices' settings were performed at a rate of $1m/min$.

In figure B.45 the actuation history on the orifices is presented.

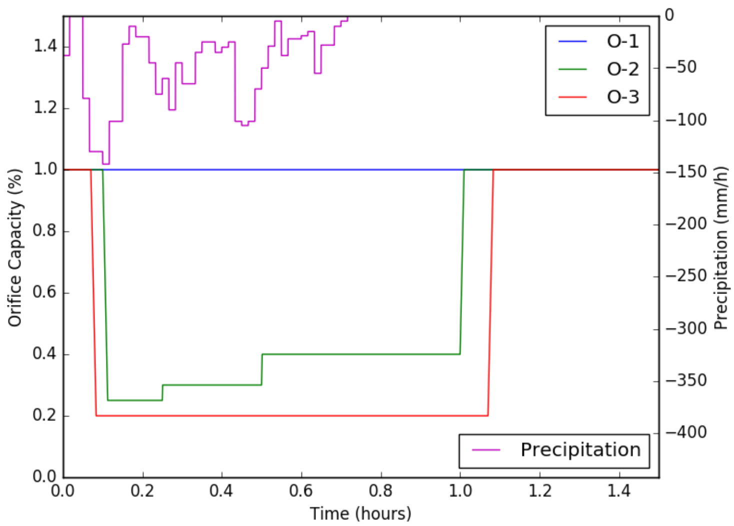

FIGURE B.45: Actuation history on the orifices.

The motivation behind these rules is to attempt to mitigate the effects of some of the intensity peaks identified in figure B.2, while taking advantage of the system's reaction time (that is, the time that it takes for certain precipitation peaks to have expression in water levels in the system).

Therefore, the first defined rule aims to retain water in earlier sections of the system during the first, and biggest, precipitation peak measured in the simulation. When the precipitation intensity drops the orifice remains closed at its smallest value, allowing downstream sections to drain water accumulated during this time period.

Once the second precipitation peak starts (around 0.25 hours into the simulation) the orifice is intended to remain as closed as possible. However, there is a need to increase its setting a little bit, allowing more water to be drained, otherwise upstream junctions would overload.

After the initial half hour of simulation, no major precipitation intensity values are registered, meaning that the orifice's setting can be relaxed. However, considering that there is still water entering the system, and that in the previous 30 minutes of simulation considerably high volumes of water have entered the system, the orifice's setting must not be completely opened.

Finally, once the precipitation has ended, plus an additional time tolerance, the orifice can be fully opened, thus allowing all remaining water to be drained at a faster rate, through the system.

The overall results of this actuation in the downstream junctions of the system are presented in figure B.46.

As presented in this figure, not only is this actuation on the orifices capable of reducing the second peak, but it is also capable of considerable reductions in the minimum levels registered before such peak. Furthermore, by keeping the orifice partially closed in the second half hour of the simulation a slight reduction in the second minimum value registered for *J-13* is also possible.

Unfortunately, in these experiments it was not possible to reduce the last peak, which occurs mostly as a result of the accumulated water over the system, as precipitation intensity values registered during those periods are considerably smaller than previous ones.

With the current configuration, no major improvements in the performance of the system seem to be possible, considering that water level in the junction immediately upstream of orifice *O-2* is very close to its maximum value of $1.5m$. This statement is supported by figure B.47.

Furthermore, other junctions upstream of *O-2* also show signs of overloading, as it is the case of junction *J-26* whose average water levels between 0.5 and 0.7 hours of simulation are greater than 1.0 meters.

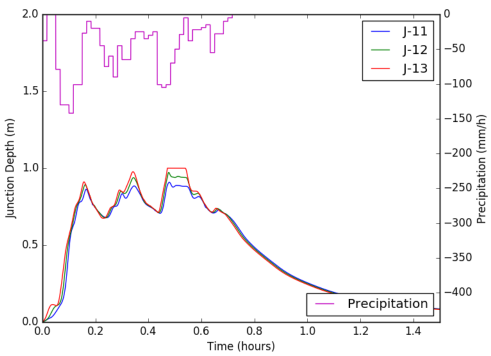

FIGURE B.46: Water level in downstream junctions of the system.

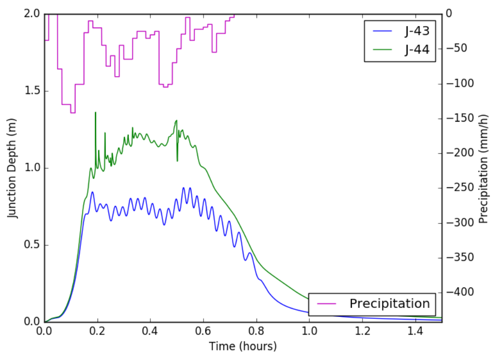

FIGURE B.47: Water level in junctions immediately upstream of orifices *O-2* and *O-3*.

B.5.4 Full Orifice Control

After testing the presented configuration with different rules that govern the actuations on the first two orifices and reaching a configuration with a performance that appeared, in light of the tests performed, to be close to the optimum, it was time to introduce the control of the third orifice of the system.

Like in the previous experiment, the control rules that govern the actuation on orifice *O-3* remained unchanged. Regarding previously defined control rules for orifice *O-2*, during initial tests where the orifice *O-1* was being used to perform heavy water retentions it appeared that when the *O-2*'s setting reached values rounding 0.4 the amount of water flowing through it was enough to overload junctions upstream of *O-1*. However, after some fine-tuning of the rules for both orifices no major improvements were obtained, when compared to the experiment to be presented next.

In this experiment both control rules for orifices *O-2* and *O-3* remained as defined in the previous subsection. The following control rules were set for orifice *O-1*:

- If simulation time $\in$ [0.1, 0.25] h, then change orifice's *O-1* setting to 0.4
- If simulation time $\in$]0.25, 0.35] h, then change orifice's *O-1* setting to 0.5
- If simulation time $\in$]0.35, 0.5] h, then change orifice's *O-1* setting to 0.7
- If simulation time > 0.5 h, then change orifice's *O-1* setting to 1.0

In figure B.48 the actuation history on the orifices is presented.

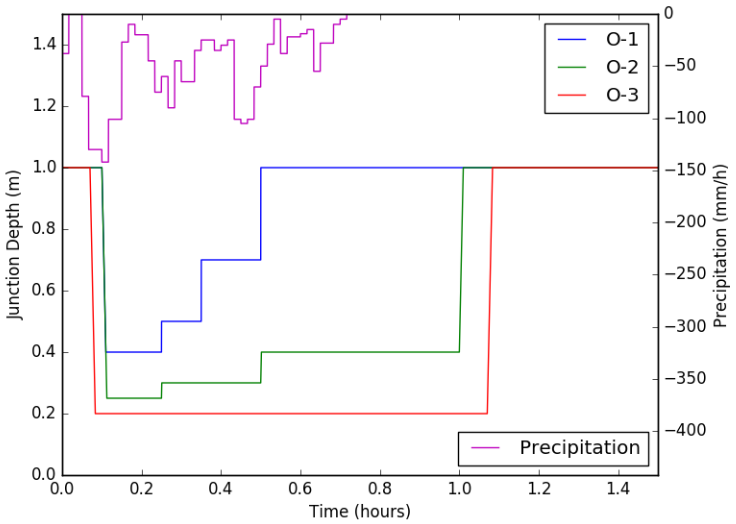

FIGURE B.48: Actuation history on the orifices.

The main motivation behind these rules is to increase water retention in the initial periods of the simulation, in which precipitation intensity is higher, promoting the rise of water levels in the conduits and junctions of the system, namely in the downstream ones.

As presented in figure B.49, with this rules it is possible to further attenuate the second peak in junction *J-13*, as well as the measured minimums before and after such peak. As can be seen in figure B.50, the junction immediately upstream of orifice *O-1*, *J-42*, is not on its full capacity. Looking closely into the depth in other junctions and conduits upstream of this orifice one can find that these structures can still carry more water.

With this in mind, additional tests were performed in which a more aggressive usage of the orifice was performed. However, due to the fact that orifice *O-1* is downstream of orifice *O-2* in this additional set of experiments no significant increase in the overall performance was registered.

Furthermore, in many scenarios, whenever orifice *O-2* was opened a considerable volume of water reached orifice *O-1* which, because it was being used with the goal of retaining as much water as possible, was stored in its upstream junctions, overloading them and promoting the departure of water form the system.

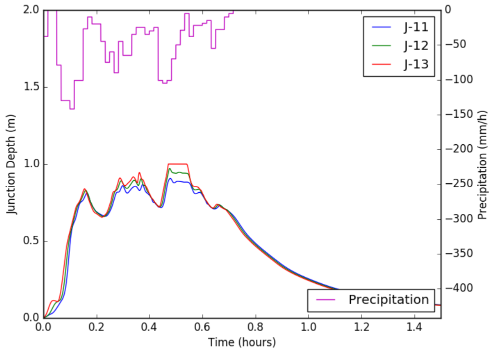

FIGURE B.49: Water level in downstream junctions of the system.

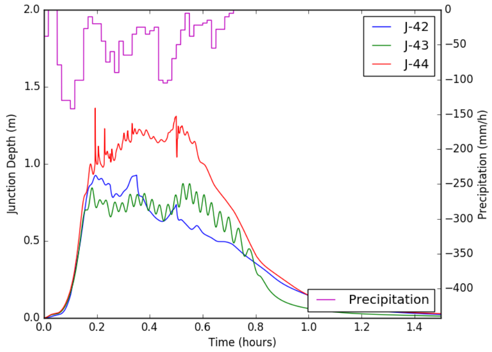

FIGURE B.50: Water level in junctions immediately upstream of the orifices.

Supplementary tests with a slightly different approach were also conducted, although with no major success. In these tests water levels in downstream junctions when no control actions were being taken were analysed, and rules aiming to retain as much water as possible, using the three orifices, during periods in which water levels in downstream junctions decreases abruptly were developed.

The main idea and motivation behind this approach was to promote water drainage during such periods, removing it from the system, so that it would gain storage volume to be used to retain excess water in downstream junctions and conduits, during subsequent heavy rain periods.

B.6 Remarks

In the previous sections many different experimental scenarios were tested and studied. Starting from an initial concept of the system and its behaviour, experiments with different configurations were performed from which conclusions were drawn regarding the behaviour of the system and, above all, the impact of control measures performed in the barriers throughout the entire system.

As this experiments were being carried out alternative solutions were also being considered and tested, from which intermediate conclusions were also extracted and presented.

Thus, and taking into account the extension of previous sections and the multiplicity of tested scenarios, the main goal of the current section is to gather and present the main conclusions withdrawn from this exercise.

As a starting point, it is important to mention that reproducing a scenario that allowed for some kind of control to be performed is not a trivial task. Indeed, underground conduits do not tend to have high available storage volumes.

Additionally, water stored over several conduits is not evenly distributed through these structures, accumulating in higher volumes in downstream sections of the system. Moreover, it is important to point out that for these systems become overloaded considerable amounts of water usually need to be transported by them.

In fact, this last evidence considerably increases the difficulty of the control tasks to be performed since, according to the documented experiments, even when small water volumes are being transported through the system, small actuations in the orifice can lead to sudden and drastic reductions in the water flow that runs through these structures. Even if these control actions are maintained for only a few seconds considerable excess water volumes need to be stored in the conduits, namely in those directly upstream of the orifice, which justifies the sudden rises in water levels in those structures.

Thus, although in some of the performed experiments it was possible to produce a scenario in which downstream conduits were overloaded while upstream sections were not operating at full capacity, the use of barriers in upstream conduits was not always able to successfully retain water in volumes high enough to considerably reduce ascending water flows in downstream sections of the system.

Furthermore, in many cases, even when all conduits upstream of the orifice had enough available space to store excess volumes, they do not always reach those sections, specially the ones in more upstream locations. Thus, water retention is highly conditioned by the capacity of the conduits and junctions immediately upstream of the orifice.

A possible solution that could allow this scenario to be reproduced would consist of increasing the height of upstream conduits, thus boosting its storage capacity and allowing them to tolerate higher water volumes stored when handling the orifice. However, considering the nature and typical locations of the systems under study, current legislation does not provide the construction of structures with greater height, meaning that this option was discarded from this study.

In many of the studied scenarios, developed control rules sought not only to retain water in upstream structures during intense rainfall period, but also to promote their drainage through the system when the same rainfall intensity diminished.

Following this approach, ideally, the system would retain water during heavy rainfall periods, draining out in quieter periods to regain storage capacity to be used again in subsequent heavy rain periods. Nonetheless, in all the developed scenarios and with the considered precipitation time series, it was found that the intervals between recorded precipitation peaks were insufficient for accumulated water to be drained, in a controlled way, through the system.

Appendix C

Constant Intensity With Constant Actuation During Peak

TABLE C.1: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.1 m*.

Base Intensity	Peak Duration	Peak Intensity	Barriers Actuation	C-6 Peak Level (No Control)	C-6 Peak Level (Control)	% Level Reduction
2	5 min	12	0.1/0.1	0.219779	0.181525	17.4056666
2	5 min	22	0.1/0.1	0.306149	0.212523	30.58184087
2	5 min	32	0.1/0.1	0.37706	0.228579	39.37861348
2	5 min	42	0.1/0.15	0.439385	0.263431	40.04551817
2	5 min	52	0.15/0.2	0.497976	0.352021	29.30964544
2	5 min	62	0.15/0.2	0.55466	0.369123	33.45058234
2	5 min	72	0.2/0.25	0.611518	0.46254	24.36199752
2	5 min	82	0.2/0.25	0.672457	0.475781	29.24737195
2	10 min	12	0.1/0.1	0.224617	0.203976	9.189420213
2	10 min	22	0.1/0.1	0.306149	0.238834	21.9876596
2	10 min	32	0.1/0.15	0.37706	0.276504	26.66843473
2	10 min	42	0.15/0.2	0.439385	0.373569	14.97911854
2	10 min	52	0.15/0.2	0.497976	0.39269	21.142786
2	10 min	62	0.2/0.25	0.55466	0.481901	13.11776584
2	10 min	72	0.25/0.3	0.611518	0.570528	6.702991572
2	10 min	82	0.25/0.3	0.672457	0.597466	11.15179112
2	15 min	12	0.1/0.1	0.224957	0.223825	0.50320728
2	15 min	22	0.1/0.1	0.310077	0.265018	14.53155184
2	15 min	32	0.15/0.15	0.377116	0.353146	6.356134452
2	15 min	42	0.2/0.2	0.439385	0.429797	2.182140947
2	15 min	52	0.2/0.2	0.497976	0.466071	6.406935274
2	15 min	62	0.25/0.25	0.55466	0.542283	2.231457109
2	15 min	72	0.25/0.3	0.611518	0.581235	4.9521028
2	15 min	82	0.3/0.3	0.672457	0.661399	1.644417413

TABLE C.2: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.15 m*.

Base Intensity	Peak Duration	Peak Intensity	Barriers Actuation	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
6	5 min	16	0.1/0.1	0.259653	0.203956	21.45055131
6	5 min	26	0.1/0.1	0.33642	0.225377	33.00725284
6	5 min	36	0.1/0.1	0.402135	0.24028	40.24892138
6	5 min	46	0.1/0.15	0.462455	0.271853	41.21525338
6	5 min	56	0.15/0.2	0.519839	0.364718	29.84020052
6	5 min	66	0.15/0.2	0.576194	0.377212	34.53385492
6	5 min	76	0.2/0.25	0.633973	0.471245	25.66797009
6	5 min	86	0.25/0.3	0.697886	0.564201	19.15570738
6	10 min	16	0.1/0.1	0.260744	0.223193	14.40148191
6	10 min	26	0.1/0.1	0.33642	0.257427	23.48047084
6	10 min	36	0.15/0.15	0.402135	0.336809	16.24479342
6	10 min	46	0.15/0.2	0.462455	0.381225	17.56495227
6	10 min	56	0.2/0.2	0.519839	0.457066	12.07546952
6	10 min	66	0.2/0.25	0.576194	0.49369	14.31878846
6	10 min	76	0.25/0.3	0.633973	0.578238	8.79138386
6	10 min	86	0.3/0.3	0.697886	0.662937	5.007837956
6	15 min	16	0.1/0.1	0.260995	0.232478	10.92626296
6	15 min	26	0.1/0.15	0.336447	0.278564	17.20419561
6	15 min	36	0.15/0.15	0.402135	0.359525	10.59594415
6	15 min	46	0.15/0.2	0.462455	0.402095	13.05208074
6	15 min	56	0.2/0.2	0.519839	0.47924817	7.80834643
6	15 min	66	0.25/0.25	0.576194	0.560019	2.807214237
6	15 min	76	0.25/0.3	0.633973	0.600678	5.251800944
6	15 min	86	0.3/0.35	0.697886	0.679162	2.682959681

TABLE C.3: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.2 m*.

Base Intensity	Peak Duration	Peak Intensity	Barriers Actuation	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
10	5 min	20	0.1/0.1	0.292339	0.222542	23.87536388
10	5 min	30	0.1/0.1	0.363579	0.248307	31.70480143
10	5 min	40	0.1/0.15	0.426485	0.268843	36.96308194
10	5 min	50	0.15/0.2	0.48527	0.355125	26.8190904
10	5 min	60	0.15/0.2	0.541865	0.3724	31.27439491
10	5 min	70	0.2/0.25	0.598337	0.463135	22.59629607
10	5 min	80	0.2/0.25	0.657765	0.475641	27.68830813
10	10 min	20	0.1/0.1	0.293011	0.237872	18.81806485
10	10 min	30	0.1/0.15	0.363579	0.277013	23.80940593
10	10 min	40	0.15/0.2	0.426485	0.370387	13.15356929
10	10 min	50	0.2/0.2	0.48527	0.43125	11.13194716
10	10 min	60	0.2/0.25	0.541865	0.478882	11.62337483
10	10 min	70	0.25/0.25	0.598337	0.558464	6.663970304
10	10 min	80	0.25/0.3	0.657765	0.591305	10.10391249
10	15 min	20	0.1/0.1	0.293235	0.24776	15.50803963
10	15 min	30	0.15/0.15	0.363602	0.335731	7.665249366
10	15 min	40	0.15/0.2	0.426485	0.383625	10.04959143
10	15 min	50	0.2/0.2	0.48527	0.460145	5.177530035
10	15 min	60	0.2/0.25	0.541865	0.499918	7.741227058
10	15 min	70	0.25/0.25	0.598337	0.57934	3.174966616
10	15 min	80	0.3/0.3	0.657765	0.652841	0.748595623

TABLE C.4: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.25 m*.

Base Intensity	Peak Duration	Peak Intensity	Barriers Actuation	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
15	5 min	25	0.1/0.1	0.329416	0.271145	17.68918328
15	5 min	35	0.1/0.1	0.395625	0.339005	14.31153239
15	5 min	45	0.15/0.15	0.456047	0.328539	27.95939892
15	5 min	55	0.15/0.2	0.513431	0.368852	28.15938266
15	5 min	65	0.2/0.25	0.569656	0.451326	20.77218532
15	5 min	75	0.2/0.25	0.626922	0.472266	24.66909759
15	5 min	85	0.25/0.3	0.690254	0.565703	18.0442272
15	10 min	25	0.1/0.1	0.329877	0.2748	16.69622314
15	10 min	35	0.15/0.15	0.395625	0.340816	13.85377567
15	10 min	45	0.15/0.2	0.456047	0.381359	16.37725936
15	10 min	55	0.2/0.2	0.513431	0.456236	11.13976367
15	10 min	65	0.25/0.25	0.569656	0.531587	6.682805061
15	10 min	75	0.25/0.3	0.626922	0.577713	7.849301827
15	10 min	85	0.3/0.3	0.690254	0.660238	4.348544159
15	15 min	25	0.15/0.15	0.330079	0.317672	3.758797136
15	15 min	35	0.15/0.15	0.395658	0.357206	9.718494255
15	15 min	45	0.2/0.2	0.456047	0.436193	4.353498653
15	15 min	55	0.2/0.25	0.513431	0.480644	6.385862949
15	15 min	65	0.25/0.25	0.569656	0.556629	2.286818712
15	15 min	75	0.25/0.3	0.626953	0.596268	4.894306272
15	15 min	85	0.3/0.35	0.690392	0.675742	2.121982873

TABLE C.5: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.3 m*.

Base Intensity	Peak Duration	Peak Intensity	Barriers Actuation	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
20	5 min	30	0.15/0.15	0.363655	0.316866	12.8663156
20	5 min	40	0.15/0.15	0.426302	0.332656	21.96705622
20	5 min	50	0.15/0.2	0.484865	0.365689	24.57921277
20	5 min	60	0.15/0.2	0.541303	0.378134	30.14374574
20	5 min	70	0.2/0.25	0.597683	0.467951	21.70582064
20	5 min	80	0.2/0.25	0.657196	0.476856	27.44082435
20	10 min	30	0.15/0.15	0.364073	0.335841	7.754488798
20	10 min	40	0.15/0.2	0.426302	0.374328	12.19182645
20	10 min	50	0.15/0.2	0.484865	0.392736	19.00095903
20	10 min	60	0.2/0.25	0.541303	0.480467	11.2388071
20	10 min	70	0.25/0.25	0.598661	0.559157	6.598726157
20	10 min	80	0.25/0.3	0.657196	0.592741	9.807576431
20	15 min	30	0.15/0.15	0.364259	0.339326	6.844854897
20	15 min	40	0.15/0.2	0.426907	0.383918	10.0698747
20	15 min	50	0.2/0.2	0.485415	0.469985	3.17872336
20	15 min	60	0.2/0.25	0.541819	0.506452	6.527456586
20	15 min	70	0.25/0.25	0.597704	0.580923	2.807576995
20	15 min	80	0.3/0.3	0.657262	0.652959	0.65468565

TABLE C.6: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.35 m*.

Base Intensity	Peak Duration	Peak Intensity	Barriers Actuation	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
30	5 min	40	0.15/0.2	0.426403	0.419555	1.605992453
30	5 min	50	0.2/0.2	0.48485	0.407099	16.03609364
30	5 min	60	0.2/0.2	0.541215	0.426344	21.22465194
30	5 min	70	0.2/0.25	0.597617	0.472437	20.94652595
30	5 min	80	0.25/0.3	0.657242	0.558915	14.96054726
30	10 min	40	0.2/0.2	0.426741	0.406772	4.679419132
30	10 min	50	0.2/0.2	0.48485	0.439334	9.387645664
30	10 min	60	0.2/0.25	0.541215	0.481141	11.09984017
30	10 min	70	0.25/0.25	0.597617	0.559402	6.394563742
30	10 min	80	0.25/0.3	0.657242	0.594322	9.573338283
30	15 min	40	0.2/0.2	0.426909	0.414153	2.987990415
30	15 min	50	0.2/0.2	0.484934	0.462719	4.58103577
30	15 min	60	0.2/0.25	0.541335	0.502286	7.213463013
30	15 min	70	0.25/0.3	0.597763	0.576988	3.475457665
30	15 min	80	0.3/0.3	0.657307	0.653073	0.644143452

TABLE C.7: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.4 m*.

Base Intensity	Peak Duration	Peak Intensity	Barriers Actuation	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
35	5 min	45	0.2/0.2	0.456145	0.430663	5.586381523
35	5 min	55	0.2/0.2	0.513328	0.432117	15.82048904
35	5 min	65	0.2/0.25	0.569464	0.467372	17.92773555
35	5 min	75	0.2/0.25	0.626982	0.47783	23.7888807
35	5 min	85	0.25/0.3	0.690393	0.571308	17.2488713
35	10 min	45	0.2/0.2	0.456466	0.432622	5.223609206
35	10 min	55	0.2/0.2	0.513328	0.463246	9.756335131
35	10 min	65	0.25/0.25	0.569464	0.537247	5.657425228
35	10 min	75	0.25/0.3	0.626982	0.578571	7.721274295
35	10 min	85	0.3/0.3	0.690393	0.661665	4.161108238
35	15 min	45	0.2/0.2	0.456632	0.456632	0
35	15 min	55	0.2/0.25	0.513406	0.482245	6.069465491
35	15 min	65	0.25/0.25	0.569572	0.557458	2.126860169
35	15 min	75	0.25/0.3	0.62703	0.597926	4.641564199
35	15 min	85	0.3/0.35	0.690495	0.67619	2.071702185

Appendix D

Constant Intensity With Variable Actuation During Peak

TABLE D.1: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.1 m* (obtained with a Base Precipitation Intensity of *2 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
32	0.1/0.1 - 0.1/0.15	4 min	0.37706	0.259637	31.14172811
32	0.1/0.1 - 0.1/0.15	4.5 min	0.37706	0.259169	31.26584628
32	0.1/0.1 - 0.1/0.15	5 min	0.37706	0.259124	31.27778072
32	0.1/0.15 - 0.15/0.15	9.5 min	0.377116	0.30107	20.16514812
32	0.1/0.15 - 0.15/0.15	10 min	0.377116	0.300749	20.25026782
42	0.1/0.15 - 0.15/0.2	4 min	0.439385	0.305039	30.57591861
42	0.1/0.15 - 0.15/0.2	4.5 min	0.439385	0.304464	30.70678334
42	0.1/0.15- 0.15/0.2	5 min	0.439385	0.303931	30.82808926
42	0.15/0.2 - 0.2/0.2	9.5 min	0.439385	0.392742	10.61551942
42	0.15/0.2 - 0.2/0.2	10 min	0.439385	0.392756	10.61233315
52	0.15/0.2 - 0.2/0.2	9.5 min	0.497976	0.446902	10.25631757
52	0.15/0.2 - 0.2/0.2	10 min	0.497976	0.446679	10.30109885
62	0.15/0.2 - 0.2/0.25	4 min	0.55466	0.453899	18.16626402
62	0.15/0.2 - 0.2/0.25	4.5 min	0.55466	0.453308	18.27281578
62	0.15/0.2 - 0.2/0.25	5 min	0.55466	0.453061	18.31734756
62	0.2/0.25 - 0.25/0.25	9.5 min	0.55466	0.514099	7.312768182
62	0.2/0.25 - 0.25/0.25	10 min	0.55466	0.513826	7.361987524
72	0.2/0.25 - 0.25/0.3	4 min	0.611518	0.524305	14.26172247
72	0.2/0.25 - 0.25/0.3	4.5 min	0.611518	0.523923	14.32418997
72	0.2/0.25 - 0.25/0.3	5 min	0.611518	0.523678	14.3642542
82	0.2/0.25 - 0.25/0.3	4 min	0.672457	0.587429	12.64437726
82	0.2/0.25 - 0.25/0.3	4.5 min	0.672457	0.587164	12.68378499
82	0.2/0.25 - 0.25/0.3	5 min	0.672457	0.586646	12.76081593
82	0.25/0.3 - 0.3/0.3	9.5 min	0.672457	0.645184	4.055724009
82	0.25/0.3 - 0.3/0.3	10 min	0.672457	0.64518	4.056318843

TABLE D.2: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.15 m* (obtained with a Base Precipitation Intensity of *6 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
26	0.1/0.1 - 0.1/0.15	9.5 min	0.336447	0.26655	20.77504035
26	0.1/0.1 - 0.1/0.15	10 min	0.336447	0.266465	20.80030436
36	0.1/0.1 - 0.15/0.15	4 min	0.402135	0.282819	29.670633
36	0.1/0.1 - 0.15/0.15	4.5 min	0.402135	0.28236	29.78477377
36	0.1/0.1 - 0.15/0.15	5 min	0.402135	0.281925	29.8929464
46	0.1/0.15 - 0.15/0.2	4 min	0.462455	0.323532	30.04032825
46	0.1/0.15 - 0.15/0.2	4.5 min	0.462455	0.323227	30.10628061
46	0.1/0.15 - 0.15/0.2	5 min	0.462455	0.322895	30.17807138
56	0.15/0.2 - 0.2/0.2	4 min	0.519839	0.414123	20.33629643
56	0.15/0.2 - 0.2/0.2	4.5 min	0.519839	0.413733	20.41131966
56	0.15/0.2 - 0.2/0.2	5 min	0.519839	0.413452	20.46537486
66	0.15/0.2 - 0.2/0.25	4 min	0.576194	0.473677	17.7920978
66	0.15/0.2 - 0.2/0.25	4.5 min	0.576194	0.473483	17.82576702
66	0.15/0.2 - 0.2/0.25	5 min	0.576194	0.473	17.90959295
66	0.2/0.25 - 0.25/0.25	9.5 min	0.576194	0.542962	5.767501918
66	0.2/0.25 - 0.25/0.25	10 min	0.576194	0.542934	5.772361392
76	0.2/0.25 - 0.25/0.3	4 min	0.633973	0.55776	12.02148987
76	0.2/0.25 - 0.25/0.3	4.5 min	0.633973	0.557331	12.08915837
76	0.2/0.25 - 0.25/0.3	5 min	0.633973	0.557125	12.12165187
86	0.2/0.25 - 0.3/0.3	4 min	0.697886	0.608343	12.83060557
86	0.2/0.25 - 0.3/0.3	4.5 min	0.697886	0.607995	12.88047045
86	0.2/0.25 - 0.3/0.3	5 min	0.697886	0.607682	12.92532018
86	0.3/0.3 - 0.3/0.35	9.5 min	0.697886	0.680715	2.4604305
86	0.3/0.3 - 0.3/0.35	10 min	0.697886	0.680679	2.465588936

TABLE D.3: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.2 m* (obtained with a Base Precipitation Intensity of *10 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
30	0.1/0.1 - 0.1/0.15	4 min	0.363579	0.263894	27.41770014
30	0.1/0.1 - 0.1/0.15	4.5 min	0.363579	0.263768	27.45235561
30	0.1/0.1 - 0.1/0.15	5 min	0.363579	0.263558	27.51011472
30	0.1/0.15 - 0.15/0.15	9.5 min	0.363602	0.298019	18.0370295
30	0.1/0.15 - 0.15/0.15	10 min	0.363602	0.297504	18.17866788
40	0.1/0.1 - 0.15/0.2	4 min	0.426485	0.305154	28.4490662
40	0.1/0.1 - 0.15/0.2	4.5 min	0.426485	0.304586	28.58224791
40	0.1/0.1 - 0.15/0.2	5 min	0.426485	0.304102	28.69573373
50	0.15/0.2 - 0.2/0.2	4 min	0.48527	0.395708	18.45611721
50	0.15/0.2 - 0.2/0.2	4.5 min	0.48527	0.395373	18.52515095
50	0.15/0.2 - 0.2/0.2	5 min	0.48527	0.395221	18.55647372
60	0.15/0.2 - 0.2/0.25	4 min	0.541865	0.444084	18.04526958
60	0.15/0.2 - 0.2/0.25	4.5 min	0.541865	0.444139	18.03511945
60	0.15/0.2 - 0.2/0.25	5 min	0.541865	0.444393	17.9882443
70	0.2/0.25 - 0.25/0.25	4 min	0.598337	0.516615	13.65818928
70	0.2/0.25 - 0.25/0.25	4.5 min	0.598337	0.516407	13.6929523
70	0.2/0.25 - 0.25/0.25	5 min	0.598337	0.516052	13.75228341
80	0.2/0.25 - 0.25/0.25	4 min	0.657765	0.579524	11.89497769
80	0.2/0.25 - 0.25/0.25	4.5 min	0.657765	0.579279	11.93222503
80	0.2/0.25 - 0.25/0.25	5 min	0.657765	0.578921	11.98665177

TABLE D.4: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.25 m* (obtained with a Base Precipitation Intensity of *15 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
25	0.1/0.1 - 0.15/0.15	9.5 min	0.330079	0.289901	12.17223756
25	0.1/0.1 - 0.15/0.15	10 min	0.330079	0.289913	12.16860206
35	0.1/0.1 - 0.15/0.15	4 min	0.395625	0.2968	24.97946288
35	0.1/0.1 - 0.15/0.15	4.5 min	0.395625	0.296615	25.02622433
35	0.1/0.1 - 0.15/0.15	5 min	0.395625	0.2964	25.08056872
45	0.15/0.15 - 0.15/0.2	4 min	0.456047	0.370999	18.64895504
45	0.15/0.15 - 0.15/0.2	4.5 min	0.456047	0.370891	18.67263681
45	0.15/0.15 - 0.15/0.2	5 min	0.456047	0.370834	18.68513552
45	0.15/0.2 - 0.2/0.2	9.5 min	0.456047	0.405943	10.98658691
45	0.15/0.2 - 0.2/0.2	10 min	0.456047	0.405558	11.07100803
55	0.15/0.2 - 0.2/0.2	4 min	0.513431	0.413703	19.42383689
55	0.15/0.2 - 0.2/0.2	4.5 min	0.513431	0.413213	19.51927328
55	0.15/0.2 - 0.2/0.2	5 min	0.513431	0.412992	19.56231704
55	0.2/0.2 - 0.2/0.25	9.5 min	0.513431	0.479297	6.648215632
55	0.2/0.2 - 0.2/0.25	10 min	0.513431	0.47909	6.688532636
65	0.2/0.25 - 0.25/0.25	4 min	0.569656	0.498825	12.43399525
65	0.2/0.25 - 0.25/0.25	4.5 min	0.569656	0.49865	12.46471555
65	0.2/0.25 - 0.25/0.25	5 min	0.569656	0.498374	12.51316584
75	0.2/0.25 - 0.25/0.3	4 min	0.626922	0.554136	11.61005675
75	0.2/0.25 - 0.25/0.3	4.5 min	0.626922	0.554061	11.62201996
75	0.2/0.25 - 0.25/0.3	5 min	0.626922	0.553914	11.64546786
85	0.2/0.25 - 0.25/0.3	4 min	0.690254	0.620212	10.14727912
85	0.2/0.25 - 0.25/0.3	4.5 min	0.690254	0.619993	10.17900657
85	0.2/0.25 - 0.25/0.3	5 min	0.690254	0.61969	10.22290345
85	0.3/0.3 - 0.3/0.35	9.5 min	0.690392	0.676494	2.013059247
85	0.3/0.3 - 0.3/0.35	10 min	0.690392	0.67638	2.029571606

TABLE D.5: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.3 m* (obtained with a Base Precipitation Intensity of *20 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
40	0.15/0.15 - 0.15/0.2	4 min	0.426302	0.360148	15.51810688
40	0.15/0.15 - 0.15/0.2	4.5 min	0.426302	0.359722	15.61803604
40	0.15/0.15 - 0.15/0.2	5 min	0.426302	0.359637	15.63797496
60	0.15/0.2 - 0.2/0.25	4 min	0.541303	0.450321	16.80796153
60	0.15/0.2 - 0.2/0.25	4.5 min	0.541303	0.449893	16.88703
60	0.15/0.2 - 0.2/0.25	5 min	0.541303	0.449745	16.91437143
70	0.2/0.25 - 0.25/0.25	4 min	0.598661	0.519043	0.519043
70	0.2/0.25 - 0.25/0.25	4.5 min	0.598661	0.518757	0.518757
70	0.2/0.25 - 0.25/0.25	5 min	0.598661	0.518648	0.518648
80	0.2/0.25 - 0.25/0.3	4 min	0.657196	0.581714	11.48546248
80	0.2/0.25 - 0.25/0.3	4.5 min	0.657196	0.581576	11.50646078
80	0.2/0.25 - 0.25/0.3	5 min	0.657196	0.581284	11.55089197
50	0.15/0.2 - 0.2/0.2	9.5 min	0.485415	0.438352	9.695415263
50	0.15/0.2 - 0.2/0.2	10 min	0.485415	0.438072	9.753097865
80	0.25/0.3 - 0.3/0.3	9.5 min	0.657262	0.630156	4.124078374
80	0.25/0.3 - 0.3/0.3	10 min	0.657262	0.629886	4.165157882

TABLE D.6: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.35 m* (obtained with a Base Precipitation Intensity of *30 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
40	0.15/0.2 - 0.2/0.2	4 min	0.426741	0.387748	9.13739247
40	0.15/0.2 - 0.2/0.2	4.5 min	0.426741	0.387479	9.200428363
40	0.15/0.2 - 0.2/0.2	5 min	0.426741	0.3873	9.24237418
60	0.2/0.2 - 0.2/0.25	4 min	0.541215	0.476585	11.94164981
60	0.2/0.2 - 0.2/0.25	4.5 min	0.541215	0.476449	11.96677845
60	0.2/0.2 - 0.2/0.25	5 min	0.541215	0.476365	11.98229909
70	0.2/0.25 - 0.25/0.25	4 min	0.597617	0.521784	12.68923073
70	0.2/0.25 - 0.25/0.25	4.5 min	0.597617	0.52146	12.74344605
70	0.2/0.25 - 0.25/0.25	5 min	0.597617	0.521271	12.77507166
70	0.25/0.25 - 0.25/0.3	9.5 min	0.597763	0.576112	3.622004038
70	0.25/0.25 - 0.25/0.3	10 min	0.597763	0.576039	3.634216236
80	0.25/0.3 - 0.3/0.3	9.5 min	0.657307	0.631014	4.000109538
80	0.25/0.3 - 0.3/0.3	10 min	0.657307	0.630746	4.040881962

TABLE D.7: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.4 m* (obtained with a Base Precipitation Intensity of *35 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
55	0.2/.2 - 0.2/0.25	9.5 min	0.513406	0.48043	6.422986876
55	0.2/.2 - 0.2/0.25	10 min	0.513406	0.480404	6.428051094
65	0.2/0.2 - 0.25/0.25	4 min	0.569464	0.499297	12.32158661
65	0.2/0.2 - 0.25/0.25	4.5 min	0.569464	0.498993	12.37497015
65	0.2/0.2 - 0.25/0.25	5 min	0.569464	0.498568	12.44960173
75	0.2/0.25 - 0.25/0.3	4 min	0.626982	0.558352	10.94608777
75	0.2/0.25 - 0.25/0.3	4.5 min	0.626982	0.558119	10.98324992
75	0.2/0.25 - 0.25/0.3	5 min	0.626982	0.55808	10.98947019
85	0.25/0.3 - 0.3/0.3	4 min	0.690393	0.62982	8.773698459
85	0.25/0.3 - 0.3/0.3	4.5 min	0.690393	0.629593	8.806578282
85	0.25/0.3 - 0.3/0.3	5 min	0.690393	0.629337	8.843658612
85	0.3/0.3 - 0.3/0.35	9.5 min	0.690495	0.677068	1.944547028
85	0.3/0.3 - 0.3/0.35	10 min	0.690495	0.676986	1.956422566

Appendix E

Variable Intensity During Peak

TABLE E.1: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.1 m* (obtained with a Base Precipitation Intensity of *2 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
32/42	0.1/0.1 - 0.1/0.15	5 min	0.438469	0.291088	33.61263852
32/42	0.1/0.15 - 0.1/0.15	None	0.438469	0.287884	34.34336293
42/32	0.1/0.15 - 0.1/0.1	5 min	0.439749	0.271427	38.27683519
42/32	0.1/0.15 - 0.1/0.15	None	0.439749	0.287595	34.60019238
42/52	0.1/0.15 - 0.15/0.2	5 min	0.496493	0.390582	21.3318214
42/52	0.15/0.2 - 0.15/0.2	None	0.496493	0.383961	22.66537494
52/42	0.15/0.2 - 0.1/0.15	5 min	0.498455	0.322241	35.3520378
52/42	0.15/0.2 - 0.15/0.2	None	0.498455	0.383877	22.98662868
62/72	0.15/0.2 - 0.2/0.25	5 min	0.609507	0.516847	15.20245051
62/72	0.2/0.25 - 0.2/0.25	None	0.609507	0.50196	17.6449163
72/62	0.2/0.25 - 0.15/0.2	5 min	0.612276	0.452025	26.17300041
72/62	0.2/0.25 - 0.2/0.25	None	0.612276	0.496501	18.90895609

TABLE E.2: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.15 m* (obtained with a Base Precipitation Intensity of *6 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
36/46	0.1/0.1 - 0.1/0.15	5 min	0.462232	0.303101	34.42665155
36/46	0.1/0.15 - 0.1/0.15	None	0.462232	0.298651	35.38937157
46/36	0.1/0.15 - 0.1/0.1	5 min	0.462803	0.283461	38.75126134
46/36	0.1/0.15 - 0.1/0.15	None	0.462803	0.298054	35.59808385
46/56	0.1/0.15 - 0.15/0.2	5 min	0.519211	0.405757	21.85123197
46/56	0.15/0.2 - 0.15/0.2	None	0.519211	0.394026	24.11062169
56/46	0.15/0.2 - 0.1/0.15	5 min	0.520295	0.32928	36.71282638
56/46	0.15/0.2 - 0.15/0.2	None	0.520295	0.392482	24.56548689
66/76	0.15/0.2 - 0.2/0.25	5 min	0.633798	0.551855	12.9288827
66/76	0.2/0.25 - 0.2/0.25	None	0.633798	0.51964	18.01173244
76/66	0.2/0.25 - 0.15/0.2	5 min	0.634604	0.469796	25.97021134
76/66	0.2/0.25 - 0.2/0.25	None	0.634604	0.512373	19.26098795

TABLE E.3: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.2 m* (obtained with a Base Precipitation Intensity of *10 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
30/40	0.1/0.1 - 0.1/0.15	5 min	0.426634	0.292939	31.33716488
30/40	0.1/0.15 - 0.1/0.15	None	0.426634	0.289879	32.05440729
40/30	0.1/0.15 - 0.1/0.1	5 min	0.426707	0.276895	35.10886861
40/30	0.1/0.15 - 0.1/0.15	None	0.426707	0.288671	32.3491295
40/50	0.1/0.15 - 0.15/0.2	5 min	0.485158	0.388277	19.96895857
40/50	0.15/0.2 - 0.15/0.2	None	0.485158	0.382995	21.05767606
50/40	0.15/0.2 - 0.1/0.15	5 min	0.485596	0.325188	33.03322103
50/40	0.15/0.2 - 0.15/0.2	None	0.485596	0.382166	21.29959884
60/70	0.15/0.2 - 0.2/0.25	5 min	0.598031	0.510535	14.63067968
60/70	0.2/0.25 - 0.2/0.25	None	0.598031	0.49648	16.98089229
70/60	0.2/0.25 - 0.15/0.2	5 min	0.598878	0.444331	25.80609072
70/60	0.2/0.25 - 0.2/0.25	None	0.598878	0.491364	17.95257131

TABLE E.4: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.25 m* (obtained with a Base Precipitation Intensity of *15 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
35/45	0.1/0.1 - 0.15/0.15	5 min	0.456364	0.368905	19.16430744
35/45	0.15/0.15 - 0.15/0.15	None	0.456364	0.355577	22.0847832
45/35	0.15/0.15 - 0.1/0.1	5 min	0.45626	0.295129	35.31560952
45/35	0.15/0.15 - 0.15/0.15	None	0.45626	0.353606	22.49901372
45/55	0.15/0.15 - 0.15/0.2	5 min	0.513613	0.400815	21.96167153
45/55	0.15/0.2 - 0.15/0.2	None	0.513613	0.39441	23.20871941
55/45	0.15/0.2 - 0.15/0.15	5 min	0.513737	0.379354	26.15793684
55/45	0.15/0.2 - 0.2/0.2	None	0.513737	0.39249	23.6009865
55/65	0.15/0.2 - 0.2/0.25	5 min	0.569796	0.492992	13.4792101
55/65	0.2/0.25 - 0.2/0.25	None	0.569796	0.482241	15.36602574
65/55	0.2/0.25 - 0.15/0.2	5 min	0.57006	0.416884	26.87015402
65/55	0.2/0.25 - 0.2/0.25	None	0.57006	0.479149	15.94761955
75/85	0.2/0.25 - 0.25/0.3	5 min	0.692164	0.616158	10.9809236
75/85	0.25/0.3 - 0.25/0.3	None	0.692164	0.598791	13.49001104
85/75	0.25/0.3 - 0.2/0.25	5 min	0.690374	0.55116	20.16501201
85/75	0.25/0.3 - 0.25/0.3	None	0.690374	0.589885	14.55573356

TABLE E.5: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.3 m* (obtained with a Base Precipitation Intensity of *20 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
40/50	0.15/0.15 - 0.15/0.2	5 min	0.485164	0.38823	19.97963575
40/50	0.15/0.2 - 0.15/0.2	None	0.485164	0.385174	20.60952585
50/40	0.15/0.2 - 0.15/0.15	5 min	0.485056	0.368269	24.07701379
50/40	0.15/0.2 - 0.15/0.2	None	0.485056	0.383687	20.89841173
60/70	0.15/0.2 - 0.2/0.25	5 min	0.598035	0.514234	14.01272501
60/70	0.2/0.25 - 0.2/0.25	None	0.598035	0.498604	16.62628441
70/60	0.2/0.25 - 0.15/0.2	5 min	0.598057	0.447477	25.17820208
70/60	0.2/0.25 - 0.2/0.25	None	0.598057	0.493191	17.53444906
40/60	0.15/0.15 - 0.15/0.2	5 min	0.541512	0.426099	21.31310109
40/60	0.15/0.2 - 0.15/0.2	None	0.541512	0.397948	26.51169319
60/40	0.15/0.2 - 0.15/0.15	5 min	0.541576	0.333716	38.38057816
60/40	0.15/0.2 - 0.15/0.2	None	0.541576	0.393338	27.37159697
40/70	0.15/0.15 - 0.2/0.25	5 min	0.597916	0.489756	18.08949752
40/70	0.2/0.25 - 0.2/0.25	None	0.597916	0.478626	19.95096301
70/40	0.2/0.25 - 0.1/0.1	5 min	0.598057	0.420659	29.66239004
70/40	0.2/0.25 - 0.2/0.25	None	0.598057	0.474006	20.74233727

TABLE E.6: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.35 m* (obtained with a Base Precipitation Intensity of *30 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
40/50	0.15/0.2 - 0.2/0.2	5 min	0.485172	0.432429	10.87099008
40/50	0.2/0.2 - 0.2/0.2	None	0.485172	0.4271	11.96936344
50/40	0.2/0.2 - 0.15/0.2	5 min	0.484899	0.391564	19.24833831
50/40	0.2/0.2 - 0.2/0.2	None	0.484899	0.422499	12.86865925
60/70	0.2/0.2 - 0.2/0.25	5 min	0.598039	0.510407	14.65322496
60/70	0.2/0.25 - 0.2/0.25	None	0.598039	0.50093	16.23790422
70/60	0.2/0.25 - 0.2/02	5 min	0.597731	0.480354	19.63709428
70/60	0.2/0.25 - 0.2/0.25	None	0.597731	0.495205	17.15253182
70/80	0.2/0.25 - 0.25/0.3	5 min	0.659031	0.596271	9.523072511
70/80	0.25/0.3 - 0.25/0.3	None	0.659031	0.58218	11.66121169
80/70	0.25/0.3 - 0.2/0.25	5 min	0.657288	0.518535	21.10992442
80/70	0.25/0.3 - 0.25/0.3	None	0.657288	0.576059	12.35820523

TABLE E.7: Results for Experiments with a Base Level in Conduit *C-6* ranging *0.4 m* (obtained with a Base Precipitation Intensity of *35 mm/h*).

Peak Intensity	Barriers Actuation (Start - End)	Barriers Actuation Change	C-6 Peak Level (No Control)	C- 6 Peak Level (Control)	% Level Reduction
65/75	0.2/0.2 - 0.2/0.25	5 min	0.627623	0.54749	12.76769653
65/75	0.2/0.25 - 0.2/0.25	None	0.627623	0.52683	16.05948157
75/65	0.2/0.25 - 0.2/0.2	5 min	0.627019	0.501072	20.08663214
75/65	0.2/0.25 - 0.2/0.25	None	0.627019	0.515069	17.85432339
75/85	0.2/0.25 - 0.25/0.3	5 min	0.692165	0.620595	10.34002008
75/85	0.25/0.3 - 0.25/0.3	None	0.692165	0.601653	13.0766508
85/75	0.25/0.3 - 0.2/0.25	5 min	0.690459	0.554921	19.6301301
85/75	0.25/0.3 - 0.25/0.3	None	0.690459	0.592537	14.18215998

Appendix F

Comparison Between Controller and Previous Scenarios

F.1 Constant Intensity

TABLE F.1: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.1 m*

Base Intensity	Peak Duration	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
2	5 min	12	0.219779	0.181525	0.181599	-7.4E-05
2	5 min	22	0.306149	0.212523	0.212579	-5.6E-05
2	5 min	32	0.37706	0.228579	0.228655	-7.6E-05
2	5 min	42	0.439385	0.263431	0.244412	0.019019
2	5 min	52	0.497976	0.352021	0.322601	0.02942
2	5 min	62	0.55466	0.369123	0.332521	0.036602
2	5 min	72	0.611518	0.46254	0.421896	0.040644
2	5 min	82	0.672457	0.475781	0.428254	0.047527
2	10 min	12	0.224617	0.203976	0.203994	-1.8E-05
2	10 min	22	0.306149	0.238834	0.23528	0.003554
2	10 min	32	0.37706	0.276504	0.269799	0.006705
2	10 min	42	0.439385	0.373569	0.370869	0.0027
2	10 min	52	0.497976	0.39269	0.383079	0.009611
2	10 min	62	0.55466	0.481901	0.487054	-0.005153
2	10 min	72	0.611518	0.570528	0.57523	-0.004702
2	10 min	82	0.672457	0.597466	0.610671	-0.013205
2	15 min	12	0.224957	0.223825	0.213327	0.010498
2	15 min	22	0.310077	0.265018	0.248819	0.016199
2	15 min	32	0.377116	0.353146	0.367489	-0.014343
2	15 min	42	0.439385	0.429797	0.467412	-0.037615
2	15 min	52	0.497976	0.466071	0.486501	-0.02043
2	15 min	62	0.55466	0.542283	0.587964	-0.045681
2	15 min	72	0.611518	0.581235	0.587822	-0.006587
2	15 min	82	0.672457	0.661399	0.723679	-0.06228

TABLE F.2: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.15 m*

Base Intensity	Peak Duration	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
6	5 min	16	0.259653	0.203956	0.203419	0.000537
6	5 min	26	0.33642	0.225377	0.225173	0.000204
6	5 min	36	0.402135	0.24028	0.238298	0.001982
6	5 min	46	0.462455	0.271853	0.250991	0.020862
6	5 min	56	0.519839	0.364718	0.329433	0.035285
6	5 min	66	0.576194	0.377212	0.335205	0.042007
6	5 min	76	0.633973	0.471245	0.42422	0.047025
6	5 min	86	0.697886	0.564201	0.437761	0.12644
6	10 min	16	0.260744	0.223193	0.223031	0.000162
6	10 min	26	0.33642	0.257427	0.246094	0.011333
6	10 min	36	0.402135	0.336809	0.330005	0.006804
6	10 min	46	0.462455	0.381225	0.380232	0.000993
6	10 min	56	0.519839	0.457066	0.453432	0.003634
6	10 min	66	0.576194	0.49369	0.502822	-0.009132
6	10 min	76	0.633973	0.578238	0.586955	-0.008717
6	10 min	86	0.697886	0.662937	0.679191	-0.016254
6	15 min	16	0.260995	0.232478	0.232306	0.000172
6	15 min	26	0.336447	0.278564	0.276545	0.002019
6	15 min	36	0.402135	0.359525	0.364477	-0.004952
6	15 min	46	0.462455	0.402095	0.407585	-0.00549
6	15 min	56	0.519839	0.47924817	0.479084	0.00016417
6	15 min	66	0.576194	0.560019	0.608008	-0.047989
6	15 min	76	0.633973	0.600678	0.608961	-0.008283
6	15 min	86	0.697886	0.679162	0.709039	-0.029877

TABLE F.3: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.2 m*

Base Intensity	Peak Duration	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
10	5 min	20	0.292339	0.222542	0.221768	0.000774
10	5 min	30	0.363579	0.248307	0.237179	0.011128
10	5 min	40	0.426485	0.268843	0.248309	0.020534
10	5 min	50	0.48527	0.355125	0.325381	0.029744
10	5 min	60	0.541865	0.3724	0.332641	0.039759
10	5 min	70	0.598337	0.463135	0.420252	0.042883
10	5 min	80	0.657765	0.475641	0.429162	0.046479
10	10 min	20	0.293011	0.237872	0.237499	0.000373
10	10 min	30	0.363579	0.277013	0.268999	0.008014
10	10 min	40	0.426485	0.370387	0.369894	0.000493
10	10 min	50	0.48527	0.43125	0.430295	0.000955
10	10 min	60	0.541865	0.478882	0.483372	-0.00449
10	10 min	70	0.598337	0.558464	0.557345	0.001119
10	10 min	80	0.657765	0.591305	0.603865	-0.01256
10	15 min	20	0.293235	0.24776	0.247424	0.000336
10	15 min	30	0.363602	0.335731	0.364807	-0.029076
10	15 min	40	0.426485	0.383625	0.385577	-0.001952
10	15 min	50	0.48527	0.460145	0.459761	0.000384
10	15 min	60	0.541865	0.499918	0.507194	-0.007276
10	15 min	70	0.598337	0.57934	0.573581	0.005759
10	15 min	80	0.657765	0.652841	0.715091	-0.06225

TABLE F.4: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.25 m*

Base Intensity	Peak Duration	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
15	5 min	25	0.329416	0.271145	0.254759	0.016386
15	5 min	35	0.395625	0.339005	0.257347	0.081658
15	5 min	45	0.456047	0.328539	0.292671	0.035868
15	5 min	55	0.513431	0.368852	0.330304	0.038548
15	5 min	65	0.569656	0.451326	0.416911	0.034415
15	5 min	75	0.626922	0.472266	0.424052	0.048214
15	5 min	85	0.690254	0.565703	0.511288	0.054415
15	10 min	25	0.329877	0.2748	0.259209	0.015591
15	10 min	35	0.395625	0.340816	0.329491	0.011325
15	10 min	45	0.456047	0.381359	0.375192	0.006167
15	10 min	55	0.513431	0.456236	0.452703	0.003533
15	10 min	65	0.569656	0.531587	0.533008	-0.001421
15	10 min	75	0.626922	0.577713	0.584815	-0.007102
15	10 min	85	0.690254	0.660238	0.659181	0.001057
15	15 min	25	0.330079	0.317672	0.335653	-0.017981
15	15 min	35	0.395658	0.357206	0.361626	-0.00442
15	15 min	45	0.456047	0.436193	0.478345	-0.042152
15	15 min	55	0.513431	0.480644	0.493751	-0.013107
15	15 min	65	0.569656	0.556629	0.556179	0.00045
15	15 min	75	0.626953	0.596268	0.604549	-0.008281
15	15 min	85	0.690392	0.675742	0.691303	-0.015561

TABLE F.5: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.3 m*

Base Intensity	Peak Duration	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
20	5 min	30	0.363655	0.316866	0.307689	0.009177
20	5 min	40	0.426302	0.332656	0.327138	0.005518
20	5 min	50	0.484865	0.365689	0.341037	0.024652
20	5 min	60	0.541303	0.378134	0.356153	0.021981
20	5 min	70	0.597683	0.467951	0.420729	0.047222
20	5 min	80	0.657196	0.476856	0.435898	0.040958
20	10 min	30	0.364073	0.335841	0.3259	0.009941
20	10 min	40	0.426302	0.374328	0.370063	0.004265
20	10 min	50	0.484865	0.392736	0.384596	0.00814
20	10 min	60	0.541303	0.480467	0.485445	-0.004978
20	10 min	70	0.598661	0.559157	0.557781	0.001376
20	10 min	80	0.657196	0.592741	0.605885	-0.013144
20	15 min	30	0.364259	0.339326	0.33882	0.000506
20	15 min	40	0.426907	0.383918	0.384662	-0.000744
20	15 min	50	0.485415	0.469985	0.484206	-0.014221
20	15 min	60	0.541819	0.506452	0.508937	-0.002485
20	15 min	70	0.597704	0.580923	0.573391	0.007532
20	15 min	80	0.657262	0.652959	0.717877	-0.064918

TABLE F.6: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.35 m*

Base Intensity	Peak Duration	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
30	5 min	40	0.426403	0.419555	0.369064	0.050491
30	5 min	50	0.48485	0.407099	0.377232	0.029867
30	5 min	60	0.541215	0.426344	0.39407	0.032274
30	5 min	70	0.597617	0.472437	0.425152	0.047285
30	5 min	80	0.657242	0.558915	0.508354	0.050561
30	10 min	40	0.426741	0.406772	0.412091	-0.005319
30	10 min	50	0.48485	0.439334	0.421685	0.017649
30	10 min	60	0.541215	0.481141	0.481071	7E-05
30	10 min	70	0.597617	0.559402	0.557987	0.001415
30	10 min	80	0.657242	0.594322	0.588431	0.005891
30	15 min	40	0.426909	0.414153	0.42056	-0.006407
30	15 min	50	0.484934	0.462719	0.460116	0.002603
30	15 min	60	0.541335	0.502286	0.505849	-0.003563
30	15 min	70	0.597763	0.576988	0.589794	-0.012806
30	15 min	80	0.657307	0.653073	0.69626	-0.043187

TABLE F.7: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.4 m*

Base Intensity	Peak Duration	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
35	5 min	45	0.456145	0.430663	0.405981	0.024682
35	5 min	55	0.513328	0.432117	0.422617	0.0095
35	5 min	65	0.569464	0.467372	0.440355	0.027017
35	5 min	75	0.626982	0.47783	0.462841	0.014989
35	5 min	85	0.690393	0.571308	0.521964	0.049344
35	10 min	45	0.456466	0.432622	0.4239	0.008722
35	10 min	55	0.513328	0.463246	0.46286	0.000386
35	10 min	65	0.569464	0.537247	0.555182	-0.017935
35	10 min	75	0.626982	0.578571	0.588342	-0.009771
35	10 min	85	0.690393	0.661665	0.660738	0.000927
35	15 min	45	0.456632	0.456632	0.438955	0.017677
35	15 min	55	0.513406	0.482245	0.493372	-0.011127
35	15 min	65	0.569572	0.557458	0.562095	-0.004637
35	15 min	75	0.62703	0.597926	0.606699	-0.008773
35	15 min	85	0.690495	0.67619	0.69183	-0.01564

F.2 Variable Intensity

TABLE F.8: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.1 m*

Base Intensity	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
2	32/42	0.438469	0.287884	0.27437	0.013514
2	42/52	0.496493	0.383961	0.380446	0.003515
2	62/72	0.609507	0.50196	0.503763	-0.001803
2	42/32	0.439749	0.271427	0.2652	0.006227
2	52/42	0.498455	0.322241	0.322601	-0.00036
2	72/62	0.612276	0.452025	0.437043	0.014982

TABLE F.9: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.15 m*

Base Intensity	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
6	36/46	0.462232	0.298651	0.282949	0.015702
6	46/56	0.519211	0.394026	0.38922	0.004806
6	66/76	0.633798	0.51964	0.538926	-0.019286
6	46/36	0.462803	0.283461	0.275769	0.007692
6	56/46	0.520295	0.32928	0.329433	-0.000153
6	76/66	0.634604	0.469796	0.463405	0.006391

TABLE F.10: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.2 m*

Base Intensity	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
10	30/40	0.426634	0.289879	0.273262	0.016617
10	40/50	0.485158	0.382995	0.379875	0.00312
10	60/70	0.598031	0.49648	0.49842	-0.00194
10	40/30	0.426707	0.276895	0.266462	0.010433
10	50/40	0.485596	0.325188	0.325381	-0.000193
10	70/60	0.598878	0.444331	0.420252	0.024079

TABLE F.11: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.25 m*

Base Intensity	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
15	35/45	0.456364	0.355577	0.337427	0.01815
15	45/55	0.513613	0.39441	0.383122	0.011288
15	55/65	0.569796	0.482241	0.482832	-0.000591
15	75/85	0.692164	0.598791	0.606305	-0.007514
15	45/35	0.45626	0.295129	0.292671	0.002458
15	55/45	0.513737	0.379354	0.366849	0.012505
15	65/55	0.57006	0.416884	0.416911	-2.7E-05
15	85/75	0.690374	0.55116	0.54105	0.01011

TABLE F.12: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.3 m*

Base Intensity	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
20	40/50	0.485164	0.385174	0.379374	0.0058
20	50/60	0.541586	0.413193	0.398032	0.015161
20	60/70	0.598035	0.498604	0.501516	-0.002912
20	40/60	0.541512	0.397948	0.3886487	0.009299
20	40/70	0.597916	0.478626	0.473229	0.005397
20	50/40	0.485056	0.368269	0.364868	0.003401
20	60/50	0.541576	0.409587	0.400538	0.009049
20	70/60	0.598057	0.447477	0.431223	0.016254
20	60/40	0.541576	0.333716	0.377738	-0.044022
20	70/40	0.598057	0.420659	0.420729	-7E-05

TABLE F.13: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.35 m*

Base Intensity	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
30	40/50	0.485172	0.4271	0.415634	0.011466
30	60/70	0.598039	0.50093	0.495075	0.005855
30	70/80	0.659031	0.58218	0.587771	-0.005591
30	50/40	0.484899	0.391564	0.381378	0.010186
30	70/60	0.597731	0.480354	0.470244	0.01011
30	80/70	0.657288	0.518535	0.508354	0.010181

TABLE F.14: Comparison between Experiments with the developed Controller and the set of previously defined rules with a Base Level in Conduit *C-6* ranging *0.4 m*

Base Intensity	Peak Intensity	C-6 Peak Level (No Control)	C-6 Peak Level (Hard-Coded)	C-6 Peak Level (Controller)	Difference
35	65/75	0.627623	0.52683	0.5273	-0.00047
35	75/85	0.692165	0.601653	0.610737	-0.009084
35	75/65	0.627019	0.501072	0.499506	0.001566
35	85/75	0.690459	0.554921	0.548241	0.00668

Appendix G

Overload in Downstream Conduits

G.1 Constant Intensity

G.1.1 5-minute Peaks

TABLE G.1: Results of the experiments conducted with a constant rainfall intensity during peaks promoting an overload in downstream conduits of the drainage system, for water levels ranging from *0.25* to *0.4 m*.

Base Water Level	Peak Intensity	Constant Inflow Intensity	Max Outlet Flow (No Control)	Max Outlet Flow (Control)	Max Surface Level (No Control)	Max Surface Level (Control)
0.4	85	1	0	0	0	0
0.4	85	1.1	0.05607	0	0.020551	0
0.4	85	1.2	0.157959	0	0.046258	0
0.4	85	1.3	0.260341	0.033981	0.066955	0.0019
0.4	85	1.4	0.347973	0.085412	0.0907	0.00806
0.4	85	1.5	0.42806	0.169323	0.112235	0.0278
0.35	80	1	0	0	0	0
0.35	80	1.1	0.026099	0	0.003538	0
0.35	80	1.2	0.100346	0	0.032572	0
0.35	80	1.3	0.203552	0.002523	0.05577	0.0011
0.35	80	1.4	0.301479	0.083019	0.076921	0.00289
0.35	80	1.5	0.383225	0.128211	0.100679	0.0204
0.3	80	1	0	0	0	0
0.3	80	1.1	0.026099	0	0.003538	0
0.3	80	1.2	0.100346	0	0.032572	0
0.3	80	1.3	0.203552	0.002523	0.05577	0.0011
0.3	80	1.4	0.301479	0.083019	0.0769	0.00289
0.3	80	1.5	0.383225	0.128211	0.122739	0.0204
0.25	85	1	0	0	0	0
0.25	85	1.1	0.054758	0	0.019124	0
0.25	85	1.2	0.156443	0	0.045149	0
0.25	85	1.3	0.259427	0.043102	0.066119	0.00113
0.25	85	1.4	0.348177	0.098302	0.089313	0.00396
0.25	85	1.5	0.428499	0.139182	0.111238	0.0212

TABLE G.2: Results of the experiments conducted with a constant rainfall intensity during peaks promoting an overload in downstream conduits of the drainage system, for water levels ranging from *0.1* to *0.2 m*.

Base Water Level	Peak Intensity	Constant Inflow Intensity	Max Outlet Flow (No Control)	Max Outlet Flow (Control)	Max Surface Level (No Control)	Max Surface Level (Control)
0.2	80	1	0	0	0	0
0.2	80	1.1	0.024475	0	0.003187	0
0.2	80	1.2	0.09884	0	0.031273	0
0.2	80	1.3	0.20211	0.002529	0.054676	0.0011
0.2	80	1.4	0.301505	0.083816	0.07555	0.00136
0.2	80	1.5	0.384028	0.125316	0.099283	0.0149
0.15	86	1	0	0	0	0
0.15	86	1.1	0.065036	0	0.021542	0
0.15	86	1.2	0.167034	0	0.047056	0
0.15	86	1.3	0.270076	0.042129	0.067768	0.001
0.15	86	1.4	0.357311	0.064261	0.091157	0.00344
0.15	86	1.5	0.437586	0.130982	0.112906	0.0194
0.1	82	1	0	0	0	0
0.1	82	1.1	0.036884	0	0.00822	0
0.1	82	1.2	0.120274	0	0.036099	0
0.1	82	1.3	0.223869	0.002089	0.058597	0.000848
0.1	82	1.4	0.320269	0.060036	0.079937	0.00155
0.1	82	1.5	0.401956	0.128201	0.103159	0.0144

G.1.2 10-minute Peaks

TABLE G.3: Results of the experiments conducted with a constant rainfall intensity during peaks promoting an overload in downstream conduits of the drainage system.

Base Water Level	Peak Intensity	Constant Inflow Intensity	Max Outlet Flow (No Control)	Max Outlet Flow (Control)	Max Surface Level (No Control)	Max Surface Level (Control)
0.4	85	1	0	0	0	0
0.4	85	1.1	0.059934	0.050434	0.026161	0.00104
0.4	85	1.2	0.162987	0.100236	0.050395	0.0296
0.35	80	1	0	0	0	0
0.35	80	1.1	0.026099	0.003621	0.007245	0.000807
0.35	80	1.2	0.105641	0.094825	0.037846	0.0197
0.3	80	1	0	0	0	0
0.3	80	1.1	0.025159	0	0.003334	0
0.3	80	1.2	0.099591	0.090097	0.031927	0.0187
0.25	85	1	0	0	0	0
0.25	85	1.1	0.059884	0.056763	0.02608	0.00134
0.25	85	1.2	0.162967	0.099332	0.050378	0.0279
0.2	80	1	0	0	0	0
0.2	80	1.1	0.024475	0	0.00715	0
0.2	80	1.2	0.105606	0.071403	0.037808	0.0175
0.15	86	1	0	0	0	0
0.15	86	1.1	0	0	0	0
0.15	86	1.2	0.069676	0.046075	0.026036	0.00638
0.1	82	1	0	0	0	0
0.1	82	1.1	0.036884	0	0.015561	0
0.1	82	1.2	0.128427	0.070916	0.043003	0.0208

G.1.3 15-minute Peaks

TABLE G.4: Results of the experiments conducted with a constant rainfall intensity during peaks promoting an overload in downstream conduits of the drainage system.

Base Water Level	Peak Intensity	Constant Inflow Intensity	Max Outlet Flow (No Control)	Max Outlet Flow (Control)	Max Surface Level (No Control)	Max Surface Level (Control)
0.4	85	1	0	0	0	0
0.4	85	1.1	0.060449	0.053835	0.02657	0.0215
0.35	80	1	0	0	0	0
0.35	80	1.1	0.034281	0.034281	0.008281	0.00603
0.3	80	1	0	0	0	0
0.3	80	1.1	0.030505	0.030505	0.008272	0.0058
0.25	85	1	0	0	0	0
0.25	85	1.1	0.060446	0.053756	0.026565	0.0204
0.2	80	1	0	0	0	0
0.2	80	1.1	0.033854	0.033854	0.008261	0.00572
0.15	86	1	0	0	0	0
0.15	86	1.1	0.071791	0.058701	0.029621	0.0231
0.1	82	1	0	0	0	0
0.1	82	1.1	0.036884	0.030308	0.016332	0.0109

G.2 Variable Intensity

TABLE G.5: Results of the experiments conducted with a variable rainfall intensity during precipitation peaks promoting an overload in downstream conduits of the drainage system, for base water levels of *0.4* and *0.35 m*.

Base Water Level	Peak Intensity	Constant Inflow Intensity	Max Outlet Flow (No Control)	Max Outlet Flow (Control)	Max Surface Level (No Control)	Max Surface Level (Control)
0.4	75/85	1	0	0	0	0
0.4	75/85	1.1	0.058354	0	0.02285	0
0.4	75/85	1.2	0.161456	0.048153	0.049154	0.00472
0.4	75/85	1.3	0.263181	0.138634	0.069546	0.0325
0.4	75/85	1.4	0.346617	0.242353	0.09437	0.0579
0.4	75/85	1.5	0.427719	0.336958	0.113702	0.0792
0.4	85/75	1	0	0	0	0
0.4	85/75	1.1	0.056641	0	0.021545	0
0.4	85/75	1.2	0.158661	0	0.047203	0
0.4	85/75	1.3	0.260892	0.033982	0.067674	0.00134
0.4	85/75	1.4	0.347973	0.09387	0.091995	0.0205
0.4	85/75	1.5	0.42806	0.199535	0.112851	0.0497
0.35	70/80	1	0	0	0	0
0.35	70/80	1.1	0.025202	0	0.004943	0
0.35	70/80	1.2	0.103284	0	0.035097	0
0.35	70/80	1.3	0.20741	0.06257	0.058582	0.0168
0.35	70/80	1.4	0.301345	0.16515	0.08128	0.0454
0.35	70/80	1.5	0.38206	0.266716	0.103455	0.0666
0.35	80/70	1	0	0	0	0
0.35	80/70	1.1	0.026099	0	0.003881	0
0.35	80/70	1.2	0.101173	0	0.033689	0
0.35	80/70	1.3	0.204194	0.002523	0.056672	0.0011
0.35	80/70	1.4	0.301479	0.083019	0.078265	0.00518
0.35	80/70	1.5	0.383225	0.140846	0.101746	0.036

TABLE G.6: Results of the experiments conducted with a variable rainfall intensity during precipitation peaks promoting an overload in downstream conduits of the drainage system, for base water levels of *0.3* and *0.25 m*.

Base Water Level	Peak Intensity	Constant Inflow Intensity	Max Outlet Flow (No Control)	Max Outlet Flow (Control)	Max Surface Level (No Control)	Max Surface Level (Control)
0.3	70/80	1	0	0	0	0
0.3	70/80	1.1	0	0	0	0
0.3	70/80	1.2	0.012424	0	0.000959	0
0.3	70/80	1.3	0.090224	0	0.030317	0
0.3	70/80	1.4	0.19323	0.032053	0.054079	0.000933
0.3	70/80	1.5	0.291763	0.058943	0.07508	0.00767
0.3	80/70	1	0	0	0	0
0.3	80/70	1.1	0	0	0	0
0.3	80/70	1.2	0.01029	0	0.001031	0
0.3	80/70	1.3	0.088812	0	0.029589	0
0.3	80/70	1.4	0.191897	0	0.053428	0
0.3	80/70	1.5	0.291379	0.066114	0.074193	0.0014
0.25	75/85	1	0	0	0	0
0.25	75/85	1.1	0.058306	0	0.022672	0
0.25	75/85	1.2	0.161368	0.032686	0.049052	0.00177
0.25	75/85	1.3	0.263184	0.123859	0.069489	0.0274
0.25	75/85	1.4	0.346654	0.229614	0.094332	0.054
0.25	75/85	1.5	0.427718	0.328647	0.113701	0.0748
0.25	85/75	1	0	0	0	0
0.25	85/75	1.1	0.055631	0	0.020674	0
0.25	85/75	1.2	0.157758	0	0.046595	0
0.25	85/75	1.3	0.260247	0.043092	0.067246	0.00136
0.25	85/75	1.4	0.348177	0.098302	0.091302	0.018
0.25	85/75	1.5	0.428499	0.192518	0.112	0.0474

TABLE G.7: Results of the experiments conducted with a variable rainfall intensity during precipitation peaks promoting an overload in downstream conduits of the drainage system, for base water levels of *0.2* and *0.15 m*.

Base Water Level	Peak Intensity	Constant Inflow Intensity	Max Outlet Flow (No Control)	Max Outlet Flow (Control)	Max Surface Level (No Control)	Max Surface Level (Control)
0.2	70/80	1	0	0	0	0
0.2	70/80	1.1	0	0	0	0
0.2	70/80	1.2	0.012745	0	0.00107	0
0.2	70/80	1.3	0.091667	0	0.031829	0
0.2	70/80	1.4	0.19575	0.038384	0.056152	0.000859
0.2	70/80	1.5	0.290387	0.08073	0.078142	0.0261
0.2	80/70	1	0	0	0	0
0.2	80/70	1.1	0	0	0	0
0.2	80/70	1.2	0.013161	0	0.001149	0
0.2	80/70	1.3	0.088809	0	0.030026	0
0.2	80/70	1.4	0.191932	0.045596	0.053811	0.00132
0.2	80/70	1.5	0.29146	0.061956	0.074691	0.0151
0.15	76/86	1	0	0	0	0
0.15	76/86	1.1	0	0	0	0
0.15	76/86	1.2	0.05824	0	0.022557	0
0.15	76/86	1.3	0.161241	0.07109	0.048948	0.0063
0.15	76/86	1.4	0.263185	0.12341	0.069396	0.0363
0.15	76/86	1.5	0.344985	0.225764	0.094274	0.0593
0.15	86/76	1	0	0	0	0
0.15	86/76	1.1	0	0	0	0
0.15	86/76	1.2	0.069676	0	0.020241	0
0.15	86/76	1.3	0.157151	0	0.046281	0
0.15	86/76	1.4	0.259782	0.061988	0.067	0.00132
0.15	86/76	1.5	0.348063	0.117453	0.090956	0.0251

TABLE G.8: Results of the experiments conducted with a variable rainfall intensity during precipitation peaks promoting an overload in downstream conduits of the drainage system, for a base water level of *0.1 m*.

Base Water Level	Peak Intensity	Constant Inflow Intensity	Max Outlet Flow (No Control)	Max Outlet Flow (Control)	Max Surface Level (No Control)	Max Surface Level (Control)
0.1	72/82	1	0	0	0	0
0.1	72/82	1.1	0	0	0	0
0.1	72/82	1.2	0.034661	0	0.008394	0
0.1	72/82	1.3	0.114576	0	0.037859	0
0.1	72/82	1.4	0.218481	0.033652	0.060619	0.00114
0.1	72/82	1.5	0.308374	0.094559	0.083818	0.0291
0.1	82/72	1	0	0	0	0
0.1	82/72	1.1	0	0	0	0
0.1	82/72	1.2	0.029131	0	0.006056	0
0.1	82/72	1.3	0.110329	0	0.035208	0
0.1	82/72	1.4	0.213785	0.009148	0.057916	0.0012
0.1	82/72	1.5	0.310502	0.072531	0.079699	0.0181

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