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Sustainable Drainage Systems Assessment and Optimisation-A case study for Lussebäcken Catchment, Helsingborg

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Abstract

Increase of urbanization and climate change are one of the factors that have been highlighted to influence hydrography. This is resulting in an increase in peak flow within an urban watershed. Downstream flooding and increased pollution are the problems that arises. To solve this, the sustainable urban drainage system techniques are adopted for reducing peak flow and increase baseflow. This thesis studies the assessment and optimisation of multipurpose constructed structures used in drainage systems in the Lussebäcken catchment in Helsingborg to reduce the possibilities of flooding while serving an environmental task. The water is transported from an upstream high plain catchment with a good fall height through Ramlösa ravine and discharges in the Råå Brook near the sea. Downstream within the low-lying area are residential buildings, industries and roads that are sensitive to flooding. Using the computer software MIKE URBAN with MOUSE engine, rainfall runoff was simulated and flow in the network system was computed. Three scenarios have been simulated; before 2008, when no additional ponds were added, the current situation with new constructed ponds and wetlands added to the model and a proposed modification to improve the regulation of the already existing infrastructures. The results show an improvement of over 20% of reduction of the peak flow in the main stream around the most critical area for the current situation over the 2008 situation and 8% more of improvement is obtained by regulating the output of one of the ponds. From the obtained results the main improvement could be made when storing capacity to the network rahter than regulating the existing ponds.

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1 Introduction

1.1 General background

Urbanisation and climate change are typical challenges in stormwater management. The increase of population which has a significant impact on urbanization, weather variability and extreme precipitation event resulting from climate change, put many regions around the world to be more vulnerable to flooding by the increase of stormwater runoff (Huong and Pathirana, 2013). The water quality and environmental biodiversity are also threatened by pollutants from the surface water flow. In the last decades,new stormwater management technics have been implemented in different parts over the world for flood risk management and pollution control (Zhang, Li et al., 2018).

Sustainable Drainage Systems (SuDS) is a wide approach adopted for managing urban stormwater. It has great importance to attenuate flow and minimise peak flow. Moreover, SuDS is a cost-effective and practical method to combat floods and control pollutants from surface runoff. Woods Ballard, B. et al.(2007), Chunglim M. et al.(2016), Woods-Ballard, B. et al. (2015) and Kyle et al.(2017) have described numerous types of SuDS, design methods and their performance, including ponds and wetlands, swales, detention basins, green roofs, trees and grass, permeable paving, and so forth (Butler and Davies, 2011, Eckart, McPhee et al., 2017, Mak, Scholz et al., 2016, Woods Ballard, Wilson et al., 2015).

Sweden has used sustainable drainage systems as possible measures to control stormwater runoff, like in Helsingborg municipality. However, the ponds and wetlands are more dominating (Semadeni-Davies, Hernebring et al., 2008). Recent studies discussed on capacity and capability of ponds and wetlands in different catchments in Sweden as stormwater control measures for improving runoff water quality, quantity and peak flow reduction (Al-Rubaei, Engström et al., 2017, Persson, 2000). The results have revealed that the ponds and wetlands are a good measure to reduce pollutant, but further studies should be carried out to find long term solution for optimising the flow.

1.2 Problem description

Stormwater control measures such as ponds, ditches and wetlands are constructed with the purpose of increasing the residence time and flow detention in the upper part of the watercourse of Lussebäcken in Helsingborg. The water is transported from an upstream high plain catchment with a good fall height through the ravine (Ramlösa-ravine) and discharges in the Råå Brook (Råån) near the sea. In the lower part of the catchment, there are residential buildings, industries and roads that are sensitive to flooding. These control measures (i.e. ponds and ditches) store flow temporarily in order to attenuate flow and reduce peak flow. However, the flow is released too quickly. This results in flooding from the increase of water level in the downstream watercourse.

1.3 Aim and objectives

The main objective of this study is to optimize and assess the possibilities to drain constructed ponds and wetlands more slowly without any major impact on upstream properties in Lussebäcken catchment. The aim is to decrease the effect of heavy rainfall (peak flows) within the catchment boundary through modelling of Sustainable Drainage System. This aimed to reach on the target of reducing the maximum flow to $2m^3/s$ (2l/s ha) in the downstream part at Ramlösa ravine.
In order to achieve the aim and objectives, the following research questions were raised:

- How can the existing sustainable stormwater conservation measures be improved and optimized by decreasing the flow in a strategic manner?
- Where and how it is possible to regulate the flow?

1.4 Thesis structure

This thesis report comprises 7 chapters. The first chapter is an introduction which is mainly composed by the background, problem description, aim and objectives of the study. The second chapter is a review of literature which describes previous studies related to the topic such as theory about historical overview, techniques and benefits of sustainable drainage systems. The third chapter comprises modelling of sustainable drainage systems. In this section, the approaches of the used model (MIKE URBAN) are extensively explained. The fourth chapter is methodology, here, the used methods and materials during the study are discussed. The fifth chapter describes the findings and discussions. The last chapter is the conclusion and recommendation for the finished work.

2 Literature review

2.1 Sustainable Urban Drainage Systems

2.1.1 Historical overview

The term "urban drainage" is not new. Urban drainage systems are typically defined as the systems composed by connected sewers or drains which collect and remove unwanted water in the urban areas (Butler and Davies, 2011). With reference to past history, urban drainage systems have been adopted and executed from ancient civilisations. It had been designed and built to drain wastewater, collect stormwater and support flood mitigation. However, those unwanted waters were discharged into receiving water without any further treatment and taking care of the negative impacts on the environment as well as other adverse effect related to untreated wastewater and stormwater. Traditionally, the ancient drainage systems were proposed, designed and constructed in traditional ways.

Steven J. Burian and Findlay G. Edwards (2002) described the historical evolution of ancient urban drainage systems and discussed the traditional and modern techniques together with engineering skills that had been used in designing drainage systems in the early civilisation. Besides, most of all ancient infrastructures especially drainage systems, roads and houses were designed by using traditional methods such as estimating measurements (trial and error). Regardless of this, even they have been used estimated measurements in planning, designing and constructing infrastructures, the ancient water-related structures were good enough to serve their importance during that period. Currently, as a result of human development, urban drainage systems have evolved with modern technology and advanced tools (i.e. computer Models, mathematical equations and etc), precise and accurate measurements, and skilled engineers.

With return to back, several authors had been studied for the origin and evolution of cities and urbanisation development from the past-period specifically before industrial evolution (Davis, 1955, Sjoberg, 1965) showed that the most traditional cities were built to around 4000-3000 years before Christ. The thoughts of drainage systems were started at that time of cities development to drain (collect) sewerage and rainwater away from cities to the receiving water bodies. Nevertheless, urban drainage has been continued to be developed over time across different cities over the world. Most of the traditional urban drainage systems were combined, this means that the only single sewer was needed for stormwater and wastewater. But, at that time, some cities started to use separate systems for the purpose of collecting and treating stormwater runoff to provide drinking water. For instance, the Mesopotamian Empire cities (Babylon and Assyria) had been collecting rain water and wastewater separately (Burian and Edwards, 2002).

During the seventeenth century, urban drainage system had been widely spread and evolved over the Romans cities and worldwide(Burian and Edwards, 2002, Patouillard and Forest, 2011). Although the drainage system was used for the goal of collecting and conveying stormwater and waste water, Roman empire engineers invented the system to drain roofs rainwater and wastewater from the houses to the main drains. They then started to build underground sewers and storage facilities such as open ditches and ponds to reduce overland flow. The 18^{th} century, also called industrial revolution, is characterised by the increase of population, urban development and industrialisation, this led to the increase of water pollution and landscape modification. Currently, the management of urban stormwater has become a serious problem for urban planners and engineers. Thus, the natural surface and hydrological cycle had been highly modified by human activities. For instance, cutting down trees to provide construction space and materials accompanying with surface pavements impacted on the decrease of surface infiltration.

The huge amount of water drains from cities with short time concentration created the stress on the existing urban drainage system that is likely to cause flooding. Effectively, proper design of urban drainage system and suggestion of new solutions for stormwater are needed to solve those issues of urban stormwater for maintaining public health and protect environments. As time runs, cities and population have continued to grow worldwide. The 19th and 20th centuries have given the new change in the urban drainage system. Many cities include European and American cities started to treat urban runoff and wastewater before discharging into receiving water bodies. Even with the implementation of new solutions, illegal dumping of garbage into the streets, drains and receiving water is still being problematic on urban drainage system (Burian and Edwards, 2002).

In the middle of the 20th century, Guidelines and regulations for solid waste handling, wastewater management and their disposal were developed over the different cities to address the issues of improper waste and wastewater disposal on urban stormwater systems and environmental concern (UNEP). Despite of this, the drainage pipe systems have remained overloaded during the heavy rainfall events and this has been provoking floods. To tackle those problems and finding sustainable solutions on urban stormwater management, the new technology and design techniques were introduced. As the country cities remain growing, application of computer models has started to be applied in designing construction and planning urban drainage systems. It is not only this, to minimise the problem could be raised due to a dramatic increase of drainage system expansion in urban communities, the other alternative techniques such as an adaptation of sustainable drainages system, low impact developments and the best management practices are suggested and implemented for environmental concern and maintain public health.

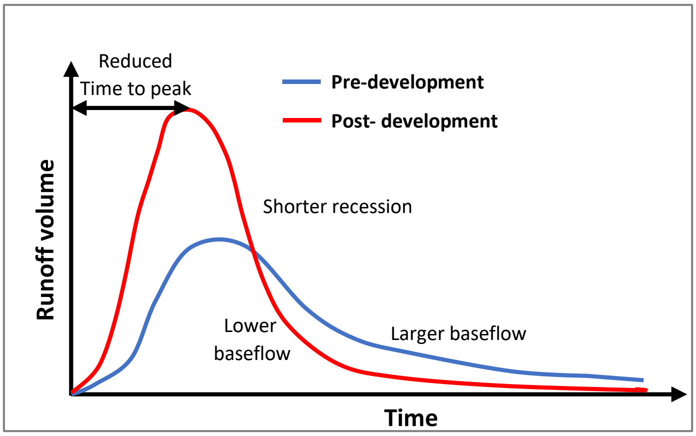
2.1.2 The approach of sustainable drainage systems

As natural watercourse patterns and hydrological components have been modified by an increase of human activities and urbanization, the generated stormwater runoff still remains a critical issue to cause flooding and water pollution discharge in natural environments (Buttler and Davis, 2011). Due to the increase of population of new areas, the natural landscape has been yet altered by the construction of infrastructures such as building, roads and industries. This followed by an increase of hard surface areas that have resulted in reducing of pervious areas on a natural surface. It explains that when it is raining, the overland flow will not be able to infiltrate into the soil and time of concentration will be reduced and will lead to a quick run off stormwater runoff over the impervious surface. This also provides the greatest negative consequences on the groundwater recharges and rivers (streams) baseflow (Fletcher, et al., 2013).

Figure 2-1 shows the effects of urban development on the urban runoff hydrograph, lower baseflow due to reducing of infiltration leads to the increase of water in the channel and more water accumulated on the surface. The peak flow and runoff volume are then increasing in traditional drainage system (Miller, Kim et al., 2014). However, those changes put stress on the existing drainage system of cities. For this case, the existing drainage will not be able to handle those additional stormwater volumes adequately.

Figure 2-1: schematic illustration of the effect of urban development on the peak flow, baseflow and time concentration of urban runoff in a catchment (Liu, J. et al., 2014)

The aim of sustainable urban drainage systems is to recreate the natural system and hydrological characteristic in development or in developed area to handle stormwater problems. Generally, the approach of sustainable urban drainage system is adopted and implemented by different states as the best conceivable solution to control stormwater runoff and mitigate flooding in urban development. However, as many countries implemented this strategy worldwide, urban drainage and stormwater management strategy has been termed according to the country, region (location) and the purpose of implementation. For instance, in Australia, the term Water Sensitive Urban Design (WSUD) was used to describe the stormwater management strategy and the most of European countries have then accepted sustainable (urban) drainage system as the official name defining the drainage of stormwater in sustainable ways (Fletcher, Shuster et al., 2014)



According to (Fletcher et al., 2014), Table 2-1 was created to summarize the term used in stormwater management strategy with selected countries.

Table 2-1: The common terminology used in stormwater management with different countries.

Countries	Stormwater management strategy
Scotland, Northern Ireland, England and Wales,	Sustainable (Urban) Drainage System (SUDS)
United States of America	Low impact development (LID)
United States of America, Canada and New Zealand	Best Management Practices (BMP)
Australia	Water Sensitive Urban Design (WSUD)
United States of America	Stormwater Control Measures (SCM)

Despite the different terminologies used to describe urban drainage and stormwater management philosophy, more clarification of the term is needed to facilitate readers and audience (Fletcher et al., 2014). Hence, although the stormwater management strategies have been found in various terms, they are commonly accepted to deliver an essential role in solving stormwater issues in urban development. In this report, the term sustainable (urban) drainage system is frequently used to describe stormwater management techniques and their benefits in a natural environment. This term "sustainable (urban) drainage system" typically used in the United Kingdom (UK) in various reports to explain UK stormwater management methods for the last three decades. Since it originated from the UK, the other European counties (Scotland, Wales and forth) have then begun to apply sustainable drainage system as the new term describing urban stormwater management concepts. The SuDS Manual report (2015) was released to provide guidance for implementing SuDS specifically in design, construction and maintenance of SuDS in the United Kingdom.

Several papers and reports describe different approaches for sustainable drainage system concepts to maximize the benefits of SuDS. The SuDS manual report (Woods Ballard, Wilson et al., 2015) and (Stahre, 2006) describe the four different fundamental elements that are crucial in the sustainable stormwater management process. Consequently, in order to design and implement effectively a sustainable stormwater management scheme, these elements should be cautiously respected.

Initially, the stormwater runoff is generated and transported from an upstream to a downstream of the catchment and it is found that the problem related to stormwater affects mostly downstream of the watershed. The best technique should be the most useful to maximise the results of the application of SUDS in handling of stormwater and control pollutants is that the stormwater runoff would have to be managed from the upstream to the downstream of the catchment. Source control (prevention), onsite control, slow transport and downstream control are the elements have been described Figure 2-2.

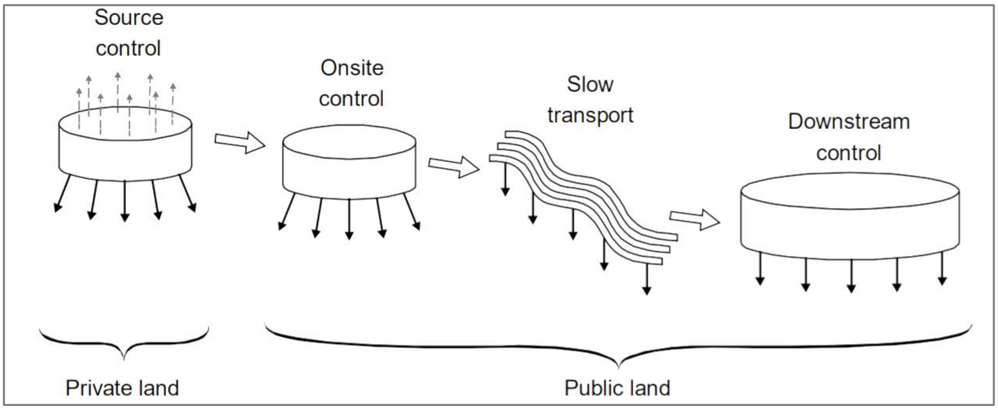


Figure 2-2: Four techniques of sustainable storm water management and pollution control (Sörensen, 2018)

As it is shown in figure 2, the four main elements are suggested to maximize the aim and benefits of the sustainable drainage system in an efficient manner. Thus, those categories of handling stormwater runoff have specific benefits to reduce overland peak flow, increase infiltration and runoff treatment train. The upstream control measures are usually done by private landowners, they typically involve the installation of source control facilities such as constructing local ponds, use of pervious pavements and green roofs. Other remain parts of stormwater handling categories are managed and taken care by a private landowner, onsite control, slow transport and downstream control are being involved. With regarding to (Woods Ballard, Wilson et al., 2015) CIRIA The SuDS manual report, Stahre (2006) and other researchers, those elements are briefly explained in the following sections.

Source control: This technique is aimed to control surface runoff at nearly and closely to the source as much as possible for the purpose of reducing the volume of water which enters in the main drainage system. Thus, it would likely be kept on its source and decrease significantly amount of water goes into drainage system through increase baseflow at the source (Perrine Hamel, et al., 2013)As it is mentioned in above paragraph, the common measures involved in runoff source control include maximize green area (green roofs), pervious parking pavements, rainwater harvesting and constructing local ponds. The construction of local ponds has benefits to reduce water flows by storing them and maximise infiltration in a private landowner (Stahre, P., 2006). Rainwater harvesting is also one of the source control facilities aimed to collect rainwater specifically from the roofs and it could stop it to enter into the drainage system (Woods-Ballard, B. et al.2015).

Onsite control: The runoff waters from different places such as agricultural land, municipal houses and roofs rainwater are needed to be detained and temporally stored (attenuated) in the open area (basins) to increase infiltration and maximize evaporation. Hence, the common measures like ponds and soakaways should be used(Sörensen, 2018) this method delivers benefits of reducing high peak flow and slow down water to the channels or drains.

Slow transport: Stormwater runoff from onsite control facilities flows quickly away to the collecting drains or channels where it will continue to move to the downstream. To avoid and reduce the impacts of a high speed of the water (quick runoff) on its way and reduce pressure on downstream parts, slow down runoff measures across on its transport structures should be taken. Increase of roughness like facilitating runoff water pass through vegetated ditches, strip, use of swales and construction of a dyke on the channels(drains) could be the most important measures to slow down runoff speed towards the downstream.

Downstream control: This stage is considered as the last part of stormwater runoff control. The water from the channels and drainage system is or may be stored in larger detention structures includes constructed wetland and ponds at a given period of time (permanently or temporarily) where it will be treated and then release to the environment.

2.1.3 The benefits of sustainable drainage systems

The sustainable drainage system can deliver numerous benefits in environmental surroundings. Climate change, land use and urbanisation are being crucial factors on the change of hydrological process and natural landscape modification of a catchment. These changes have resulted in the surface water runoff problems like an increase of high flow volume, transport of pollutants and overload of a natural drainage system. The SuDS are designed to solve these negative impacts through mimicking natural hydrological processes in development and developed area (Woods-Ballard, Kellagher et al., 2007, Woods Ballard, Wilson et al., 2015). The SuDS components have achieved their benefits based on four main categories include water quality, water quantity, amenity and biodiversity

Figure 2-3.

Reduction of water flows is one of the objectives of SuDS design, it can achieve through control of stormwater runoff. The runoff should be managed as much as possible close to its source, this would be beneficiating in reducing of a large volume of water into the drainage system and support in mitigating of flood at the downstream catchment as well as sustain and keep water balance in the natural environment. Thus, to maximize rainwater harvesting, infiltration and evaporation are the foremost activities to reduce surface runoff in efficiency manner. The SuDS components such as open constructed ponds and vegetated constructed wetlands, infiltration basins and devices, swales are the common SuDS to attenuate flow and slow-down of runoff, this results on increase of water infiltrate into the ground and increase of evaporation on surface.

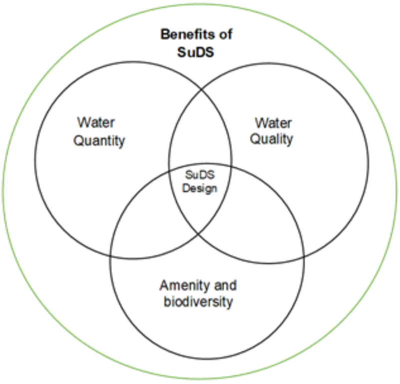


Figure 2-3: *Principle benefits of implementing Sustainable Drainage System (adopted from Woods-Ballard, B. et al., 2007)*

The pollution of runoff water is, therefore, one of the most critical issues on public health, aesthetics and environment point view and it should effectively be treated to reduce negative adverse impacts of poor treatment (Persson, 2000) The point and non-point source pollutants are likely to be carried by the surface runoff and then transported into the receiving environment. Due to the factor of that, the non-point source pollutants are generated from an unknown source, it would be complicated to control them. They might be generated by the land use activities such as agriculture while the point source pollutants are generated from a known place like industrial areas.

The organic and inorganic compounds are both the most common contaminants can be presented in stormwater runoff (Abdulkareem, Sulaiman et al., 2018). Once it rains, the rainwater flows over the land surface on the agriculture field and other different places which will wash pollutants into the drainage system where they are transported and discharged into the downstream receiving water bodies. This contributes to the contamination of stormwater runoff and the spread of unwanted waste in the natural environment (Abdulkareem, Sulaiman et al., 2018). The SuDS components can benefit on the treatment of these types of runoff pollutants and sediment control that generated from different sources.

They are also treating surface runoff by removal of contaminants to protect the release of them in the downstream environment during heavy rainfall or excess overland flow. Despite, the removal of runoff contaminants requires proper design of the SuDS to maximize their benefits. Constructed ponds and wetlands might reduce contaminants to greater extent compared with other retrofitted SuDS components. Nevertheless, the SuDS design (i.e. ponds) structures can be influenced by the hydraulic design (Persson, 2000, Persson and Wittgren, 2003) Thus, location, structure, geometry and topography are hence needed to be considered for better performance and get a complete benefit of retrofitting SuDS ponds and wetlands (Lawrence and Breen, 1998, Woods Ballard, Wilson et al., 2015). In addition to the benefit of sustainable drainage system on surface water quality and quantity, SuDS measures can deliver a greater benefit in providing amenity and enhance biodiversity in rural and urban areas. Amenity and biodiversity are both required in every pre-development and post-development communities.

Amenity typically refers to a place or services that can emphasize the attractiveness of a place in our local communities while biodiversity signifies an area that can be a habitat for plants and animals in the ecosystem (Woods-Ballard, B. et al, 2015). The SuDS design for amenity and biodiversity create and maintain a good place for human being and other animals. Once Sustainable drainage system is installed in a development area, they take opportunities to create green and open space that contribute to the increase of attractiveness of the area. The SuDS have been therefore benefiting to deliver a better place for habitats and wildlife in developing environment through improving amenity and biodiversity. The achievement of amenity and biodiversity are then providing the benefit for public health, aesthetics, recreation and tourism, this leads to the increase of economy and builds a better environment. Amenity and biodiversity should be maximized to produce the best benefits and their functions (Woods Ballard et al, 2015).

2.2 An overview of Sustainable Drainage System techniques

2.2.1 Ponds

Ponds are defined as a small open shallow basin with unbalanced water surface profile that are able to store temporarily or permanently stormwater runoff, they are generally classified by either natural or artificial (i.e. man-made). Artificial ponds are known as constructed ponds that are made by a human while the natural ponds can be generally described as ponds that already exist in the natural environment. Besides, natural ponds could occasionally but rarely be explained as manmade ponds. In comparison, natural ponds and artificial ponds both play essential functions to attenuate/store stormwater runoff, filter runoff pollutants and keep a balance between water and living organisms (plants and animals) in the ecosystem by means of providing a good environment for wildlife. Meanwhile, the natural ponds can or may be the best for maintaining ecological niche to plants, animals and wildlife and constructed ponds are a typical structure designed for multiple purposes. Recent years, ponds had been mainly constructed by farmers to provide adequate water demand for agricultural (i.e. irrigation), livestock and water supply as well as flood control. Nowadays, ponds are adopted by decision makers like one of the best structural measures and strategy for urban stormwater management practices.

Stormwater ponds are so called, wet ponds, dry ponds, retention ponds and detention ponds. These all types of stormwater ponds could be differentiated from each other by according to storage capacity, design and mode of construction, stormwater runoff characteristics and their functions. Obviously, Stormwater ponds are primarily designed to attenuate, treat stormwater runoff and serve as flood protection facilities in downstream of the catchment (Tom Schueler, David Hirschman et al., 2007). Due to urban development associated with construction activities by paving the top surface and land clearing, the percentage of impervious area increases, this contributes to reducing the amount of water penetrating into the soil. Higher volume of surface water from house roofs, gutters, streets and parking area will accumulate over the impermeable areas and runs to the downstream of an urban watershed. This will result in an increase of overland flow volume, peak flow and time of concentration. Stormwater ponds have been installed to serve the role of attenuating that volume and release it slowly to the downstream (Guo, 2001)

Furthermore, the surface water runoff may carry a large amount of unwanted pollutants and sediments washed or flushed from streets, agriculture areas, industrial areas and municipal houses including nitrogen and phosphorus that have an adverse impact on the environment and cause eutrophication. These pollutants are discharged into the stormwater ponds wherein they are treated through the biological and chemical process(Yang and Toor, 2018). As stormwater ponds installed to govern stormwater runoff and preserve biodiversity, better performance of the pond requires proper layout, construction and maintenance. According to the CIRIA report (2015), to increase effective performance, the possible details on designing, constructing and maintaining the ponds had been described. This report has continued to illustrate the principal components to be cautiously considered throughout the design and construction process. The sediment trapper (forebay), permanent pool (micro pool), storage volume and shallow water (aquatic bench) are the essential parts in designing stormwater ponds Figure 2-4. Consequently, inlets and outlets of the stormwater pond have substantial importance to control the deposit of the sediment and to ease the flow drains through the pond. They should be well located and properly designed to maximize the flow path in order to prevent the formation of dead zones (Woods Ballard et al, 2015).

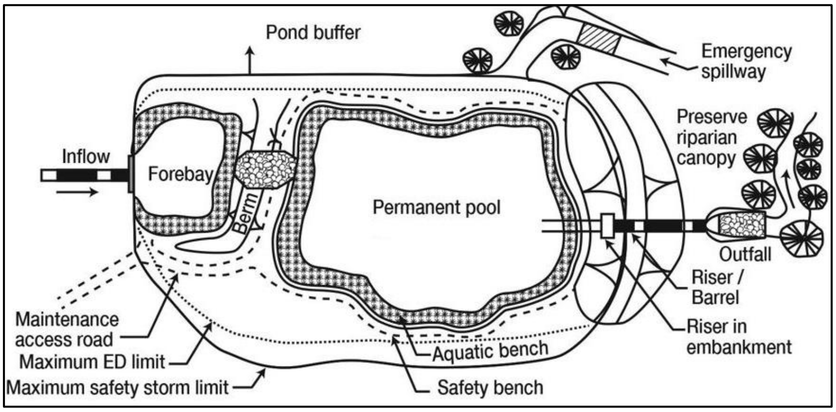


Figure 2-4: Typical illustration of a plan view of a stormwater detention pond and the sediment trapper (forebay), permanent pool (micro pool), storage volume and shallow water (i.e. Aquatic bench) (adapted from EPA, 2009).

From Figure 2-5, the plan view describes four different main parts of stormwater detention ponds to be carefully considered during designing and implementing stormwater detention ponds. The forebay is the storage part added to the stormwater detention pond which is usually located close to the inlet of the pond. The purpose of this additional storage facility is to help in pre-treatment process, the forebay traps and allows the coarse sediments to be settled down before leaving to the permanent pool(Blecken, Hunt III et al., 2017, USEPA, 2009). However, the stormwater detention pond may not usually have a forebay on its upstream and the installation of forebay could excessively be motivated by the characteristics of the inflow. For instance, when the stormwater detention pond located in the area of high sediment transport, it is highly recommended to separate the pond storage with including forebay for sediment deposit and sediment removal. Contrary, when the inflow contains a small amount of coarse sediments dissolved and debris, the forebay should be excluded and pre-treatment process might be then achieved by retrofitting Sustainable Drainage System components in the upstream and nearly of the stormwater pond inlet (Woods Ballard et al., 2015; USEPA, 2009).

Forebay should be placed for specific inlets and its size would be carefully selected to reduce the impact of inlet clogging. According to Field, R. et al. (2006), the depth of forebay should not be less or more than 1.22 m (4feet) and 1.83m(6 feet)

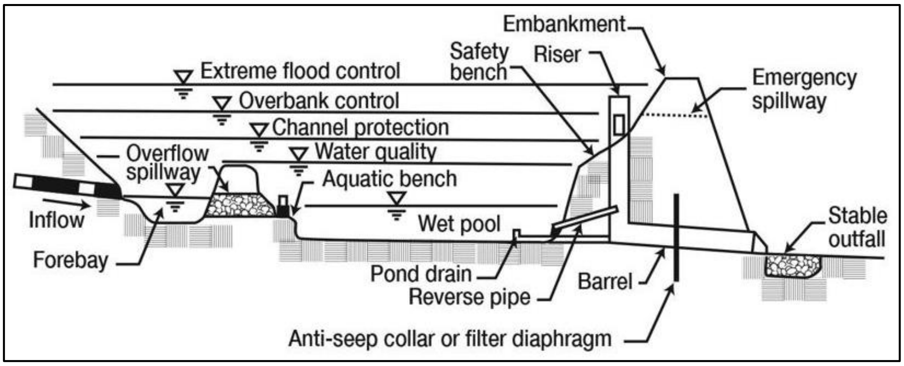


Figure 2-5: Typical illustration of a longitudinal cross-section of the constructed stormwater detention pond and the different water level design are showed (adopted from EPA, 2009)

respectively. In order to efficiently remove the inflow sediment, the flow rate at each inlet should be considered during constructing a forebay and it might be placed at every inlet. Nonetheless, forebay should mostly be installed when the inlet flow is greater than 10 % of designed incoming discharge (Field and Tafuri, 2006). After the pre-treatment stage, the stormwater is transported into the permanent filled pool that is also known as micro pool. In a permanent pool, the stormwater will store permanently or take a long period of time in the pool where stormwater would take occasion to evaporate and infiltrate prior to discharge into the next storage. The inflow fine sediments that have been not removed from the pre-treatment process; they are therefore deposited in the permanent pool. As it helps dissolved suspended fine sediments to be settled into the bottom of the permanent pool, this makes it to be considered as the main treatment of the contaminants (Woods Ballard et al., 2015). Efficiently, the volume of the permanent pool has been carefully designed to increase the efficiency of pollutants removal and it is accentuated that the storage volume of a permanent pool should be designed by comparing to its depth (J. Persson and H.B. Wittgren, 2003; Person, J., 2000).

The attenuation storage volume is then required during the high storm events and high peak flow to receive increased flowrate and it can be stored that overflow in a period short time before releasing it out through the outlet pipe (Woods Ballard et al., 2015). The latest main feature of the stormwater detention pond is the aquatic bench. This part is located alongside the permanent pool (figure 2-5) and it is always characterised by shallow water and emergent plants. The aquatic bench is the most important to provide amenity and biodiversity as well as a filter of water in the pool (Field, R. et al., 2006; Woods-Ballard, B. et al., 2015). Consequently, the evaporation and infiltration of water in the shallow zone should be controlled to maintain the growth of vegetation (Woods Ballard et al., 2015).

Several studies have been conducted to study and examine the efficient, hydraulic performance and characteristics of stormwater ponds to reduce flow and pollutant treatment. The number of authors had been pointed out that the layout and design parameters are the key factors affecting the hydraulic performance of the ponds. For instance,(Persson and Wittgren, 2003, Shih, Zeng et al., 2017, Su, Yang et al., 2009) revealed that the effective volume of the pond has been basically ascertained by length-to-width ratio and water depth. According to Persson and H.B. Wittgren (2003), albeit the water depth and length to width ratio are the most important aspect to be taken into consideration during analysis and evaluation of hydraulic performance of the pond, the other design parameters include number of outlets and inlets and their placements, topography (i.e. shape, bottom slope and side slope), flow control regulator (i.e baffles, spillway, weir) and pond vegetation could also highly influence the hydraulic function of the pond. In the study conducted on 13 Swedish ponds by Persson (2000) analysed the effect of designed length and width of the ponds, the two comparative ratios 2:1 and 5:1 with varying the number of inlets and outlets position were used and compared. The author has concluded that, even though the larger length of ponds with a small width yielded the higher percentage of the volume ratio, the position of the outlets and number of inlets are clearly one of the design factors that have been highlighted to contribute the hydraulic performance of the pond (Person, J., 2000).Commonly, the inlets and outlets of the sustainable stormwater ponds should consistently be designed in such way they ease the runoff to be able to flow into and out of the ponds for keeping a pond away from sediment deposit and clogging problems. For better performance and achieve design objectives, The Construction Industry Research and Information Association (CIRIA) the SuDS manual report (2015) can be a good guide to choose the best method of designing and constructing the inlets and outlets of the stormwater ponds. This has also been influenced by the size, types, location and functions of the ponds (Lawrence and Breen, 1998).

In order to reduce and to prevent downstream flood impacts that are caused by high runoff volume and overflow rate from the stormwater ponds, it can be necessary to design a control system to regulate the flow rate that would be released out and overtopped during heavy rainfall or high flow rate. Thus, proper design of the inlets and outlets structures are the main things to be kept in mind during designing the control system in the stormwater ponds. As it was mentioned by Persson and H.B. Wittgren (2003), the flow regulator could be introduced and well-designed to maximize the hydraulic performance of the pond. For the case of an increase of water level in the pond, it is clear that the water will be discharged over the top of the pond. To avoid and reduce the impacts of flooding in the downstream of the pond, regulator structures such as spillway, weir and baffles should be primarily needed and installed at the outlet of the pond

2.2.2 Wetlands

Wetlands are normally described like ponds or marshes. They are generally defined as very shallow area inundated with water and characterised by a growing of several small stemmed aquatic vegetations (USEPA, 2004). Despite wetland characterised with a flooded area which has almost covered by vegetations, it does not mean that every flooded area with vegetations is called wetland. Yet, the inundated area has been called wetland based on wetlands characteristics. Ellis, J.B. et al. (2003) revealed that wetlands have three fundamental characteristics, (i) wetlands are the area keeping regular growth of aquatic plants, (ii) wetlands substrates which are mostly underdrain of hydric soils and (iii) presence of permanent and seasonal soil moisture(Ellis, Shutes et al., 2003). Wetlands are essential for human being and play the most important for sustainable development worldwide. They deliver ecosystem services that are providing interest for people in the community include stormwater runoff control, mitigating flood, nutrients recycling and treatments, provision of clean water, recreation services and tourism attraction (Russi, ten Brink et al., 2013).

Wetlands exist in the environment as natural or artificial wetlands and they can store water either permanently or seasonally. Regardless of natural and artificial wetlands, several kinds of literature have been categorized wetlands in different categories. The United States Environmental Protection Agency (2001) has generally classified the most types of natural wetlands that could be found in the United States. Marshes, swamps, bogs, and fens are identified (USEPA, 2001). The Swedish wetland survey (VMI) report (2014) also stated that swamps forest, fen, bog and string mire are the common type of wetlands found in Sweden (Gunnarsson and Löfroth, 2014). However, the classification of wetlands may possibly be influenced by numerous factors including soil, climate variability, topography, human activities, river morphology (Junk, Piedade et al., 2011, USEPA, 2001).

Artificial wetlands are also called constructed wetlands, they are usually constructed with different purposes. According to the Ramsar Convention on Wetlands (2018), the most constructed wetlands are used to store water for agriculture, stormwater control and assist in flood protection and fishing. Hence, the constructed wetlands are useful in the treatment of nutrients from surface water pollutants to maintain water quality as well as improving amenity and maintain biodiversity (Ramsar, 2018). But, one of these types has been chosen during the design period and it would, of course, be influenced by design purpose.

Constructed wetlands are designed and implemented worldwide to mimic the function of a natural wetland in urban development. Ramsar Convention on Wetlands (2018) due to human activities, climate change and development, the large number of natural wetlands have been commencing to be disappeared since from last two centuries. In spite of this, artificial wetlands are obviously dominated (Davidson, 2018, Prigent, Papa et al., 2012, Ramsar, 2018). As the number of natural wetlands decreases, the benefit of wetlands to the human being and other ecosystem services have been affected. Actually, to restore natural wetlands have been providing an increase of wetlands benefit to people. Constructed wetland is one of the strategies implemented to reduce wetlands loss over the world. According to Davidson, N.C. (2017), the number of constructed wetlands has been increasing after half of 20th century (Davidson, 2018). With the time, the planners and decision makers have been considering the constructed wetlands like the most important structure measure for stormwater management and they had selected them as the Sustainable drainage system. The constructed stormwater wetlands are normally built for flood protection and pollution control in urban stormwater management, and wetlands SuDS also designed to serve benefit for habitat as well as improving the aesthetic view in the new development site. Based on several literatures, constructed stormwater wetlands are found under different types. (Blick, S.A. et al., 2004) have been discussed on three main types of constructed stormwater wetlands such as a pond, marsh and extended detention wetlands. Wetlands comprise mainly a pool that stores water for certain period either wet or dry season and water storage increases the time dissolved nutrient removal and increases infiltration in the vicinity of the pool. Even pond wetlands have a pool, marches are therefore required alongside the pond (figure). Marsh wetlands comprise only marshes, where water could be stored. The water depth in marsh wetlands is relatively very small and marsh wetlands should be designed in such way they will increase the time of concentration. For this case, design with zigzag paths should do be recommended to meet the time of pollutants removal.

Extended detention wetlands combine pond and marsh zones. They are needed to store water temporally in order to reduce the velocity of the flow and stored water releases slowly to the downstream of the wetlands. Apart from those three types, the constructed wetlands design is composed of four main parts. The description of those parts has been similarly defined as of the stormwater pond but there are a little different in the size and mode of operation (Wong, Breen et al., 1999). Figure 2-6 describes the main components of standard constructed wetlands, forebay, pond (pool), marsh and outlet pipe are shown.

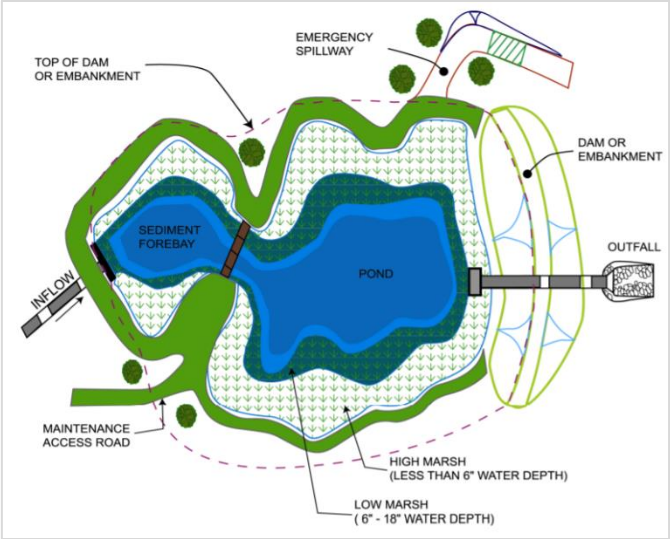


Figure 2-6: *Schematic illustration of a plan view of standard constructed stormwater wetlands components (adapted from Blick, S.A. et al., 2004)*

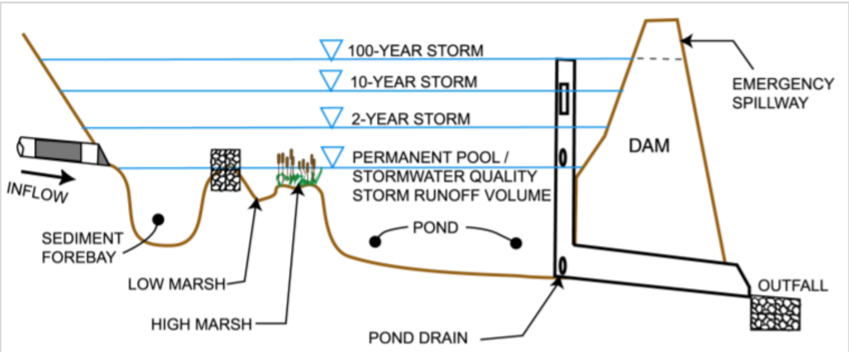


Figure 2-7: Schematic illustration of a longitudinal cross section of standard constructed stormwater wetlands components (adapted from Blick, S.A. et al., 2004)

Hydraulic performance of wetland design is based on the wetland parameters include water depth in the pond, extended length of the pool and its storage volume. Above all, the storage volume within its all storage parts should be designed with regarding to the total design volume of the incoming runoff. For the period of excessive runoff and flooding events, the control structure Figure 2-7 such as spillway or weir are needed to raise up the water level and increase the amount of water stored behind of the structure. The control structures (i.e. emergency structures) are also helping to release outflow slowly without any further downstream issues like flooding. The design of extended detention and mashes are crucial for pollutants removal in wetlands (USEPA, 2009, Woods Ballard, Wilson et al., 2015). The extended detention increases the time of settling and provide a high volume of storage, this influences the sedimentation and biological process within the wetlands. Marsh (i.e. high or shallow march) provides important for surface water treatment, it generally composed of vegetation. However, surface water runoff drains into those vegetations where it would be filtered before discharged into the main storage (pond).

Table 2-2: design parameters for different types of constructed stormwater wetlands (Blick, S.A. et al. (2004)

Wetland Design Features		**Type of Constructed Stormwater Wetland**		
		Pond	**Marsh**	**Extended Detention**
Minimum Drainage Area (Acres)		25	25	10
Minimum Length to Width Ratio		1:1	1:1	1:1
Allocation of Stormwater Quality Design Storm Runoff Volume	pool(%)	70	30	20
	marsh(%)	30	70	30
	wet(%)	0	0	50
Pool Volume	forebay(%)	10	10	10
	micro pond(%)	0	20	10
	pond(%)	60	0	0
Marsh Volume	low(%)	20	45	20
	high(%)	10	25	10
Sediment Removal Frequency (Years)		10	2 to 5	2 to 5
Outlet Configuration		Broad Crested Weir	Broad Crested Weir	Broad Crested Weir

2.2.3 Two-stage Ditches

A ditch is typically defined as a small narrow open channel which is built to convey excess water from an area and safely discharge to receiving water bodies (Hansen, Wilson et al., 2006). Despite draining excess water from an area, ditches are used for multiple purposes. They are normally used in irrigation system to transport water from an agricultural plot to another and remove excess surface water from crops (Kallio, 2010). Hence, non-point source pollutants from agriculture land (i.e. fertilizers) are likely to mix with stormwater runoff and they are discharged into the stream by overland flow. This increases the amount of organic and non-organic pollutants dissolved into surface runoff. The installation of ditches has a great benefit to reduce those nutrients transport (Davis, Tank et al., 2015). Two-stage ditches are one of the alternatives approaches of sustainable drainage system adopted to mimics natural channels in order to maximise its benefits as well as to improve water quality through nutrients removal and controlling contaminants discharge. Obviously, implementation of two-stage ditches as sustainable drainage system has been serving an essential role in protecting stream erosion, mitigate flooding and slow down the overland flow. In addition, construction of two-stage ditches enhances the bank stability of the channel which helps to resist on stream bank erosion (Davis, Tank et al., 2015, Mahl, Tank et al., 2015).

Two-stage ditches involve the modification of existing streambanks channels through expanding the wetted channel width, this transformation of natural channels has been resulted in reducing of water velocity and strengthening of banks of the channels. These outcomes have a crucial benefit to protect bank sliding and erosion at the base of the channel (Powell, Ward et al., 2007). Two-stage ditches have been implemented and benefited in the area which is flooded several times during the high peak flow events. Two-stage ditches have been constructed by changing the shape of the natural channel and the bench (flood plain) was created. The flood plain was created on both sides through reduction of the slope in existing natural channel, they are therefore built with creating buffer strips of vegetations or selected grasses and allowing them to grow into the floodplain as well as on the sloped bank of the channel or stream (Davis, Tank et al., 2015, Powell, Ward et al., 2007) Figure 2-8.

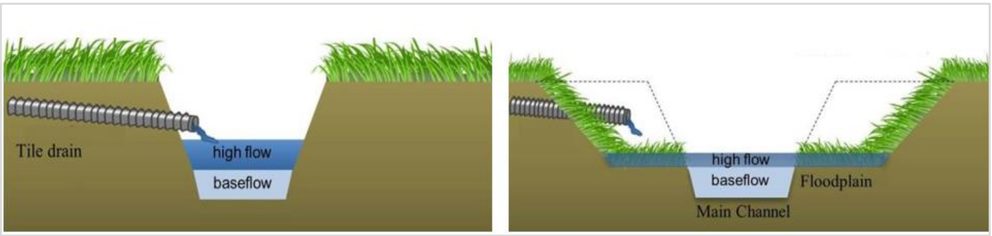


Figure 2-8: Representative cross-section of conventional channelized single stage ditch (left) and Representative cross-section of two-stage ditches (right)

The effective performance design of two stages ditches is based on the sizing of floodplain and sloping sides. During the period of high rainfall events, more water runs and accumulates into the stream. Apart from this, the water depth is totally increasing in the channel and this is likely to cause flooding. The size of the flood plain and slopping should be able to hold the much water for given return period(Davis, Tank et al., 2015).

2.2.4 Swales

Swales typically described as an open shallow and wide channel that are mainly designed to transport runoff, remove contaminants and used to store water (Wilson, Bray et al., 2004). Globally, swales have been used to manage and treat stormwater runoff since from the end of the last 20th century (F.C. Boogaard, et al., 2014). The swale channels are designed and constructed with covering planted or native vegetation (i.e. grasses) on its sides and the bottom surface. This provides the opportunities to slow down runoff before discharging to the downstream receiving waterbodies, filter pollutants and store water within a certain period. The storage of water in vegetations increases the residence time of flows in the swale, based on soil condition and groundwater characteristics, the stored water may infiltrate in the soil and recharge groundwater. In addition, swales are used as a stormwater drainage system to convey water from one location to another and this is also helping in the treatment of stormwater runoff. Once the swales are implemented and constructed in an area, they can take the chance to replace kerbs and gullies as well as traditional drainage systems (Wilson, Bray et al., 2004).

In the construction of swales are found into different types, this is influenced by the purposes and benefits of the implementation. According to Wilson, S. et al. (2004) CIRIA report " *sustainable drainage systems: hydraulic, structural and water quality advice*", based on stormwater quality design, swales have been classified swales into 3 types including swale, dry swale and wet swale

Figure *2-9*.

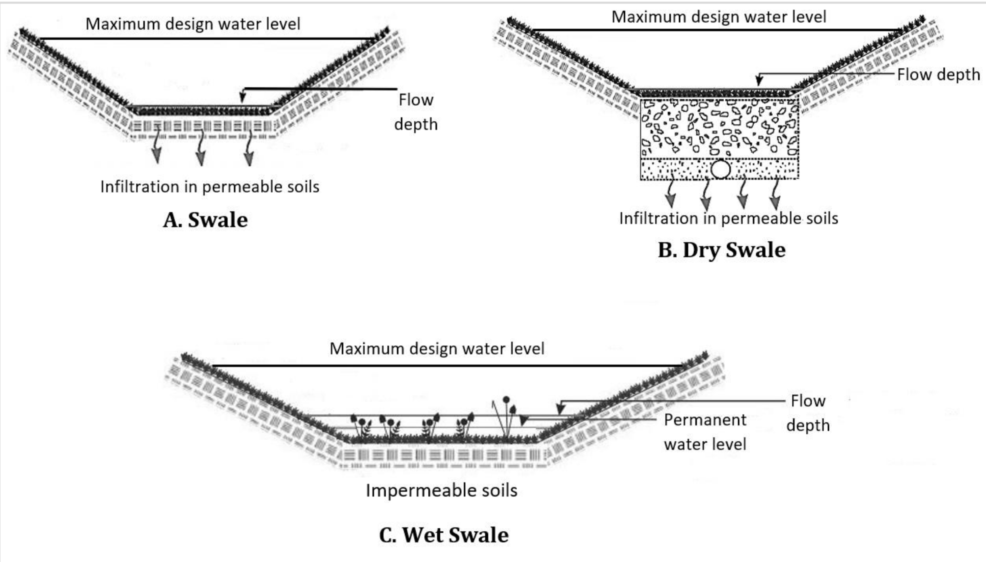


Figure 2-9: a Typical cross-section of three different types of swale: a) swale, b) enhanced dry swale c) wet swale (modified from Wilson, S. et al. ,2004)

Swale type is also known as grassed channel, it is a simple channel that is characterised with the presence of vegetation alongside and bottom of the channel and bottom allows water to infiltrate into the ground. Dry swales consist of underdrain pipe and permeable soil layer above of the pipe, the stored water is infiltrating into the permeable layer and drained by the underlaid pipe. The permeable soil layer is likely to make the swale dry due to facilitating quick downward of water movement into the subsoil layers. Swale is said to be a wet swale when it keeps water in through all the time, either in dry or in the wet season. This would be achieved from that the bottom surface or soils (i.e. clay) which difficult water passing through them and that soil material is acting as the impervious layer. Swales could be applied in all type of development. Even if they may be constructed everywhere, factors such as topography, soil type and urbanisation influence the implementation of them. For instance, it might be not easy to implement swales in a steep slope, coarse soil material and dense residential building (Wilson, Bray et al., 2004). With regarding to this, the implementation of swales has disadvantages over the above factors. Swales have often been constructed with roads where they are usually located alongside of the road or highway to receive water from the roads. Not only could swales construct with roads, but also, they can have found them in the car parking place. Due to the limited capacity of storage, the swales are not able to hold much water for a given long return period.

In order to maximize the benefit of swales as a sustainable drainage system, it could be emphasized to combine swales with other sustainable drainage system techniques, for instance, ponds. Possibly, the swales could be constructed in the upstream and the downstream of the pond, but to optimise the water flows into the ponds, it is better to implement swales in the upstream of the pond. However, the swales in downstream of the pond are crucial to reduce downstream flooding. The hydraulic performance of the swales can be influenced by the design of the underdrain gravel size, density and type of vegetation cover and slope. Several studies have been conducted to assess the hydraulic performance of the swales for peak reduction and pollutants removal. For example, (Deletic and Fletcher, 2006, Li, Li et al., 2016) have revealed that the percentage of longitudinal and transverse slope of the swales have a great impact on the efficiency of pollutants removal and flow conveyance. In addition, the minimum diameter of underdrain soil materials affects the rate of infiltration.

2.2.5 Detention Basins

Detention basins are described as surface depression (storage) that can be able to control stormwater runoff through attenuation of the flow and they are also used to treat surface water runoff (Wilson, Bray et al., 2004). Detention basins are typically dry during a dry season and wet during or after a period of rainfall. According to these characteristics of the detention basin, it is likely to describe detention basins like a dry pond and detention basin can be implemented in every proposed development area, but it would be limited by the availability of the space (Wilson, Bray et al., 2004, Woodard, 2006).

Detention basins are generally designed to detain, and store runoff generated by rainfall for a given period, for instance, 24 hours of rainfall has been mentioned by Woodard, F. in 2006. The construction of the detention basins can be done by lining the ground with earth or reinforced concrete. They also designed with a storage pool to store water which provides the time for settling of suspended sediment. This is therefore enhanced the pollutant removal to maintain runoff water quality. For the excessive or long period of rainfall that results to the increase of water level in the storage basin, the excess water runoff could be discharged by control structures such as embankments, weir or spillway Figure 2-10.

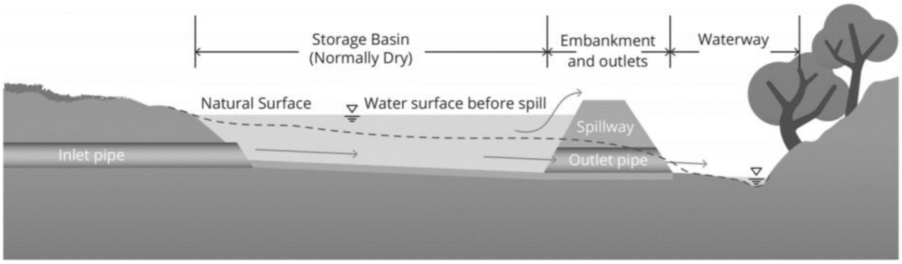


Figure 2-10: Typical illustration of detention basin cross section (Ball, Babister et al., 2016)

According to Ball, J. et al.(2016), in case of heavy rainfall intensity it will contribute to the increase of water in the stream or river and leads to a large amount of flow to the downstream, the detention basins could be constructed directly across the stream or river to reduce flow by diverting one part of the flow and another part store in detention storage. In order to maximize the benefits of detention basin as stormwater treatment train, the extended detention basins can be designed and constructed within a catchment to attenuate flow and pollutant removal (Ball, Babister et al., 2016, Wilson, Bray et al., 2004, Woods Ballard, Wilson et al., 2015).

The extended detention basins are typically designed to increases the detention time for facilitating pollutants removal. Generally, the design of the detention basin should meet design criteria for better performance of the ponds. Wilson, S. et al. (2004) has described crucial parameters which are needed to be considered during the design of detention basins such as slope (bottom or side), depth, length to width, incoming flow velocity storage area and its volume and forebay. These parameters are mostly affecting the hydraulic perfomence of the detention basin. For instance, according to (Wilson, Bray et al., 2004) the depth available for water storage should be exceded by 3m and slope should justly small slope from upstream to the downstream part.

2.3 Impacts of urbanisation on urban drainage systems

In last decade, growing populations and urbanisation has remained as one of the most important challenges on drainage system across the world especially in Sweden (Miller, Kim et al., 2014, Semadeni-Davies, Hernebring et al., 2008). Urbanisation has been recognized to have a significant effect on the runoff, water quality, aquatic habitat and change of river or channel morphology in the urban catchment(McGrane, 2016, Woods-Ballard, Kellagher et al., 2007). Development of the area associated with human activities has been greatly contributing to the change of landscape properties and runoff. Human activities such as clearing of vegetation, trees and natural soil disturbances for agriculture and finding construction sites may likely to reduce evapotranspiration and infiltration which results to the change of hydrological pattern (Naef, Scherrer et al., 2002).

In addition, these human activities contribute to the reducing of land surface permeability by paving the ground with impermeable layer and this leads to the decrease of water that can infiltrate in the soil. When it rains, water runs and accumulates over the impermeable surface, the amount of water which is needed to go into the soil(i.e. baseflow) will decrease with reducing of infiltration. The time of concentration can be also shortened that is resulting to the increase of peak flow in urban catchment. Apart from this, high volume of the water will have to be transported to the downstream of the catchment. The increase of water is an issue to the existing drainage system because it put much stress on drainage pipes and can cause downstream flooding.

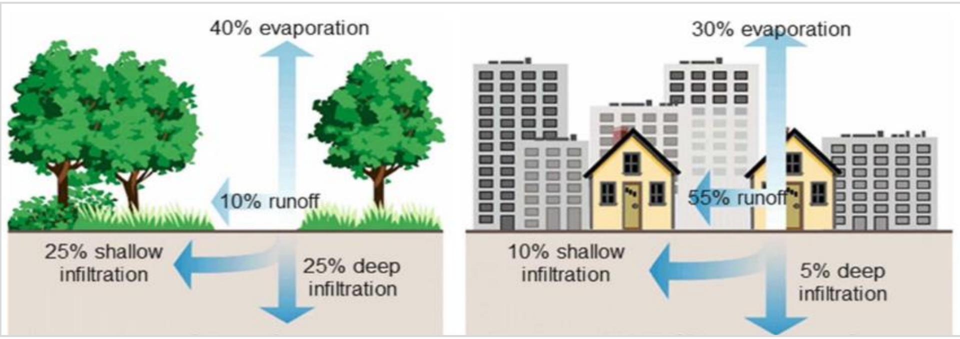


Figure 2-11: Impacts of urbanization on urban hydrology (adopted from Ligtenberg, J., 2017)

Figure 2-11 shows the comparison between rural and urban hydrological cycle. Due to urban development, runoff hydrograph of an area has been altered by the modification of catchment. As it can be seen on *Figure 2-11,* The rural catchment has been characterized by a decrease of runoff, high evaporation and increase of infiltration. However, during urban development, the land surface has been modified by human activities include covering surface by pavement and impermeable roofs result in reducing of infiltration and increase of runoff. This makes the change on runoff hydrograph by an increase of peak flow. To reduce the impacts of urbanization on the urban drainage system, the sustainable urban drainage system has been adopted and constructed worldwide. However, in order to achieve the objective of retrofitting sustainable drainage system for solving urban runoff problem, the optimization is the most crucial during the implementation of them.

3 Modelling of Sustainable Drainage Systems

3.1 Mike Urban

MIKE URBAN is a powerful kind of water modelling software developed by the Danish Institute of Hydraulics for modeling urban water distribution and collection systems. The software simulates the system in two steps. The first step consists on simulating the rainfall-runoff process to generate the inflows of the pipe network of pipes. This simulation gives the discharge for every catchment through time.

The second part of the simulation consists of a hydraulic simulation of the network. In order to model a collection system and simulates rainfall runoff, the Mouse engine can be used. Through the hydraulic modeling, The program calculates the water level, flow discharge, velocities and other parameters (DHI, 2017c)The following Figure 3-1 describes the process of generated flow into Mouse hydrological and hydraulic modelling.

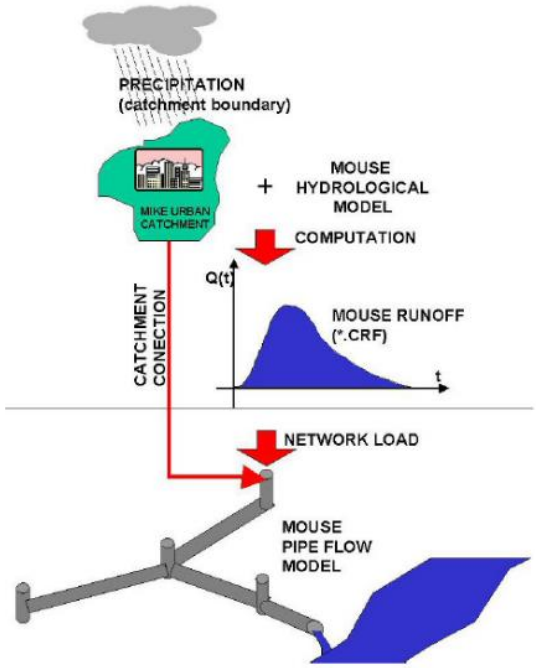


Figure 3-1: Schematic illustration of flow modelling in a collection system MOUSE models (hydrological and hydraulic model) (DHI, 2017c)

3.1.1 Rain Runoff Modelling

Rainfall-runoff modelling is necessary to understand, simulate and analyse the hydrological process in the catchment with a connection of the incoming precipitation. Rainfall-runoff models are typically built based on the available data and parameters from catchments. Hence, rainfall models are available in different types (Beven and Beven, 2001). In this study, Mike urban with mouse engine used for modelling runoff, it provides and simulates rainfall runoff through 4 different surface runoff models such as Time- area Method, Non-linear Reservoir method, Linear Reservoir Method and Unit Hydrograph Model(DHI, 2017c). With referring to those types of surface runoff models, the modelling of the runoff is done using the "Time-Area" method. This method implies that for each time the runoff from the corresponding area will be added to the hydrograph of the outflow of the catchment. The relation between both time and area is done through a time-area curve. This method is generally considered the runoff capacity and geometry of the catchment (DHI, 2017c). The general input information needed to construct the rainfall-runoff model with the Time-Area Method includes the following:

Catchment data: The study area is modelled by dividing catchments into polygons and each subdivision is assigned for specific ID and corresponding horizontal coordinates (x and y).
Catchment connections: The split sub-catchments are connected for transferring outflow of the catchment into the pipe network system.
Hydrological model: The process used to define the runoff of every catchment
Precipitation time series: The profile of the rainfall.

The other parameters that are relevant for the model of the surface runoff are:

Imperviousness of the area (%): This is a fraction of the catchment areas expressed in percentage that contributes to runoff. It is determined the degree of infiltration within the catchment boundary. This parameter is assigned for each sub-catchment based on the property of land cover.

Initial loss: This describes a depth of water before the water starts to run over the surface. It represents a threshold before the water starts to contribute to the runoff.

Hydrological reduction: This a model parameter generally represents the fraction of water lost due to evapotranspiration, perviousness of the soil and so forth.

Time of Concentration: The time of concentration refers to the required time that water takes from the most distant place of the catchment to start flowing water to the outflow of the catchment.

Time area curve: During modelling and computation of the runoff by MIKE URBAN, the geometry of the catchment such as size and shape should be considered, and the Time area curve parameter used for the model. With regarding to the mouse runoff reference manual, MIKE URBAN has an available different pre-defined Time Area Curves for runoff computations includes:

- TACurve1 – Rectangular Catchment
- TACurve2 – Divergent Catchment
- TACurve3 – Convergent Catchment

Initially, TACurve1 is set as default in the model and the users can set the curves accordingly. But instead of defining Time area curves, they can be replaced by setting Time area coefficient, that is corresponding to shape catchments. This coefficient results in a linear relationship between time and area Figure 3-2. In this project, time area curves was set as a rectangular catchment.

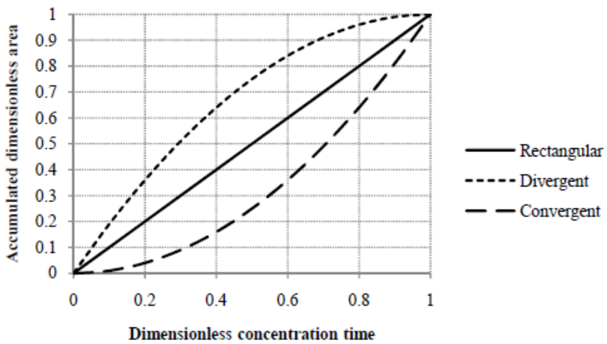


Figure 3-2: Relationship between time area coefficients with corresponding Time area curves.

Before every simulation, an RDI hot start file will be created. RDI stands for Rainfall Dependent Infiltration. This model takes into account the behaviour of the moisture in the soil and will separate the reaction of the outflow of the catchment into 2 separate components (DHI, 2017a):

- **Fast Response Component:** this response is not influenced by the previous condition of the soil like moisture levels.
- **Slow Response Component:** the infiltration response considers the previous state of the soil and will change the rate of infiltration depending on the moisture.

3.1.2 Modelling of hydraulic network

The hydraulic model is described and considered the geometric properties and the material from which are made of each element within the network. When the generated runoff from each catchment is produced through runoff modelling, it runs into the network system which made with the pipes, channel and basins (i.e. ponds and wetlands). Here, the water level, velocity, discharges are needed as the output results for a catchment. For this need, the hydraulic network helps to simulate flow, velocity and depths in the networks from the input catchment runoff. With regarding (DHI, 2017a), In order to achieve the simulation of the runoff in the network system, each sub-catchment is connected to every element of the network. The model is based mainly on the following components:

- **Links:** During the drawing the network in MIKE URBAN, a link is describing the pipes, canals and open channels and It is connected from each other by the nodes. However, the way of designing link depends on the shape of pipes or channels is being to be modelled. It can be drawn as straight line or polyline. In mike urban, the pipes cross section can be either circular, egg-shaped, O-shaped or rectangular. The computation of the flow in the link is based on the Hazen William or mining equations.

- **Nodes:** These elements can refer to manholes, outlets, basins or storage basins. Nodes are open structures connecting every pipe and found between two pipes as well as at the end of the pipe and it is always to have outlets at the end of the pipe.

- **Basins:** Ponds and storage basins are modelled as basins in the MIKE URBAN and they connect two pipes. In order to model this, the geometry of the basins should be determined and other parameters.

- **Weir:** This is a control structure connecting two nodes and they don't let the flow run until the crest level is reached. The weir can be found in different shapes, rectangular, V-notch, trapezoidal, irregular and long weirs are available for selection in the MIKE URBAN. Afterwards, the weir equation controls the flow. For the case of selecting another type of the weir more than those available, The Q-H relationship must be the second option (DHI, 2017a).

Once the model is properly set up with the network elements and adjusted parameters, the simulation can be done.

3.1.3 Computation of flow in the network system.

The flow is considered as an open channel flow, which means it has a free surface in open channel and partially full in pipes and pressurized flow for completely filled pipes. The model of the 1-D flow in an open channel network system is based on the principles of Saint Venant one dimensional equations, that describes the unsteady, gradually varied flow with pipe networks and assumptions. This equation is used by MIKEMIKE URBAN to compute water flow (DHI, 2017b). The used Saint Venant's equation in 1-D is based on equation of mass conservation which is described as continuity equation and momentum equation (French, 1985).

- **Open channel flow equations**

(i) The continuity equation is expressed as:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{3.1.3}$$

Where $\frac{\partial Q}{\partial x}$ is the change of flow in the x-direction and $\frac{\partial A}{\partial t}$ is the change cross-section of the channel with time?

(ii) The second of the equations is the conservation of momentum equation and it expressed as:

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

Where Q is the flow rate (m3/s)

A is the flow area (m2)

x is the length in the direction of flow (m)

y is a depth of the flow (m)

t is the time (s)

g is an acceleration due to gravity (9.81m/s^2)

S_o is a bottom slope of the channel

S_f is a friction slope

α is velocity distribution coefficient

Due to the complexity of solving these equations in an analytical form, numerical methods are used to find a numerical solution.

The friction slope coefficient of the channel is obtained from the Manning formula (French, 1985, Yen, 2002), it is expressed as

$$S_f = \frac{u|u|}{M^2 R^{4/3}}$$

Where, u is the average velocity of flow (m/s)

R is the hydraulic radius

M is the Manning roughness coefficient

The Manning coefficient is selected based on the channel bed material. Since no experimental data from the onsite roughness is available, already available roughness coefficients will be used:

Table 3-1: Typical roughness coefficient based on the bed channel materials (Arcement and Schneider, 1989).

Material	Manning roughness coefficient
Concrete	0,012 - 0,018
Firm soil	0,025 - 0,032
Gravel	0,028 - 0,035
Cobble	0,030 - 0,050

- **Pressurized flow equations**

To model flow in pressurized flow, 1-D requires the flow to change from non-pressurized to pressurized states. This is given the factor that the assumptions and equations for governing flow in non-pressurized and pressurized could be separately This is problematic for modelling the flow by MIKE URBAN and other similar software's. This issue has been reported by (Leandro, Chen et al., 2009). Apart from this, since the assumptions change, the equations to compute for pressurized flow is given as:

(i) The continuity equation is expressed as:

$$\frac{\partial H}{\partial t} + \frac{C^2}{gA}\frac{\partial Q}{\partial x} = 0$$

Where $\frac{\partial Q}{\partial x}$ is the change of flow in the x-direction and $\frac{\partial A}{\partial t}$ is the change cross-section of the channel?

(ii) The second of the equations is the conservation of momentum equation and it expressed as:

$$\frac{1}{A}\left[\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right)\right]+g\frac{\partial H}{\partial x}=g\left(S_0-S_f\right)$$

Where, H is piezometric head in the pipe and C is a force due to the pressure waves called celerity.

4 Methodology

4.1 Materials and Methods

A general review of the literature to the sustainable drainage system, field investigation and data collection were used to gain further information about the thesis topic and physical characteristics of the catchment. A literature review was done throughout finding the relevant materials (scientific papers, journals, report and previous studies) related to Sustainable Urban Drainage System, Stormwater Control Measures, Best Management Practices, Low Impact Development, SuDS Design and Construction, Ponds/Wetlands, Residence Time and Two-Stage Ditches. Web search engine such as Google scholar and Lub search were used and MIKE Urban software to simulate the network.

4.2 Description of case study

4.2.1 Lussebäcken catchment

This study was carried out in the Lussebäcken catchment, which is situated south-east of the Helsingborg downtown. The Lussebäcken catchment drains an area of 20 km^2, it starts at the rural part and then extends into the urban neighbourhoods of the cityb. About 2,474 km^2 of total catchment area delimited by the Highway 111 and the road 109. It is divided into a different type of land covers (i.e. agriculture land, residential and industrial area). The upper part of the catchment mainly consists of farmland and an increasing industrial area that will contribute to an increase of impervious land (Ättekulla and Långeberga) and small urban areas of Påarp). It is expected a change of land use concerning the conversion of rural land into industrial uses.

The catchment is composed of 3 small creeks, each one of the creeks forms a branch that joins and forms the main Lussebäcken creek at the downstream. The lower part of the catchment starts at Ramlösa, where the creek enters a culverted section (Figure *4-1*). It then goes through the Ramlösa brunnspark in an open-air section at the end of where there are the second culvert starts. After this point, the creek runs mostly underground until the last part, close to the Råån river, where it discharges its waters. The maximum elevation of the channel is 43 m. a. s. l. and the minimum altitude is just a few centimetres above sea level (Semadeni-Davies, Hernebring et al., 2008).

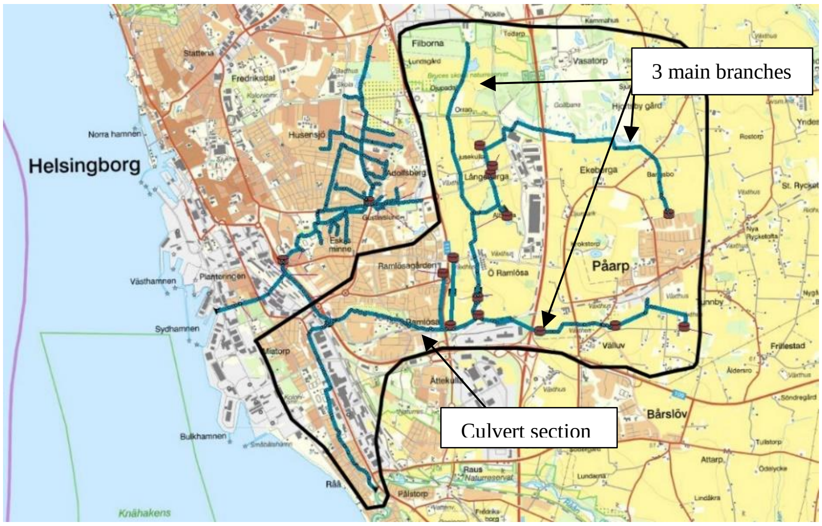


Figure 4-1: Map shows the boundary outlined in black of Lussebäcken Catchment

4.2.2 Climate and hydrology conditions

Helsingborg is characterised by the variation of seasonal temperature throughout the year. Based on the SMHI data (SMHI, 2018), the annual mean temperature was 9.9^{0}C in 2018. It is also considered a warm and temperate climate. The highest temperature occurs in July when the average temperature is about 16.1^{0}C and the lowest temperature occurs in January when the average temperature is about -0.6. From this, the climate of Helsingborg is generally classified as the mild climate zone. This seasonal change of temperature influences the precipitation of the area. The precipitation is slightly increasing with the change of temperature especially in the dry season and the annual precipitation is about 660 mm per year. The high extreme precipitation occurs in July and the lowest occurs in February. Therefore, this weather variation has a great effect on precipitation that account for the high inflow runoff in the urban catchment.

4.2.3 Data collection

The identification of data is a necessary part to evaluate the effectiveness and optimization of the sustainable drainage system in the watershed. In this study, existing and current data were collected. For instance, the existing data such as rainfall data series for previous years (provided by SMHI) was used to construct the hyetograph in order to study the hydrological characteristics of the catchment (i.e. change of runoff) and analyses the peak flow to study the impact of urbanization linked with the sustainable drainage systems. The brook water level data (provided by the NSVA) was also used to study the river characteristic for a given period, the combination of existing and current river flow measurements was used to analyse the variation of river flow. GIS data such as Digital elevation, vegetation cover and so forth was used to analyze the green cover and topography of the area.

4.2.4 SMHI data

Observation Rainfall data at Helsingborg station (A) which is located at Latitude: 56.0304, Longitude: 12.7653, between Påarpsvägen and Österleden, were obtained from Swedish Meteorological and Hydrological Institute (SMHI). Rainfall data with 15-minute precision from 1995 until 2019. Also accumulated rainfall data was obtained with 1-hour precision period.

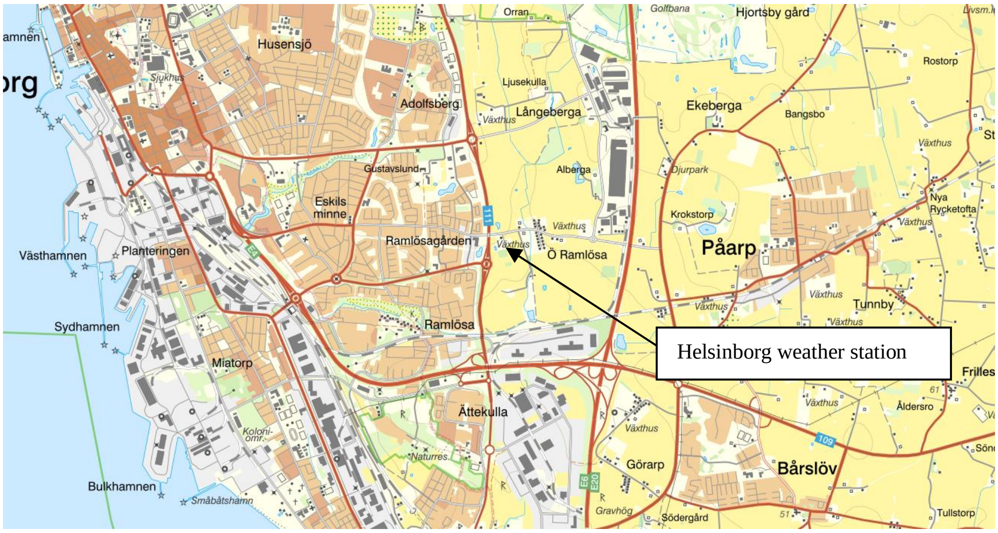


Figure 4-2: Helsingborg A weather station location

4.2.5 Field observation

Field observation was carried out around different parts of the catchment. The purpose of the visit was to gather information regarding the current situation of the study area , such as observing the different existing type of stormwater control measures (ponds, wetlands, two-stage ditches), water level measurement points, and to help to assess sustainable drainage system for finding possibilities to reduce peak flow and increase base flow as well as optimization where possible.

4.3 An assessment of Sustainable Drainage Systems in Helsingborg

Different environmentally friendly measures have been taken to prevent flooding in the catchment, like ponds, two-stage ditches and wetlands. In Figure 4-3, a map of the locations visited during the field observation can be found.

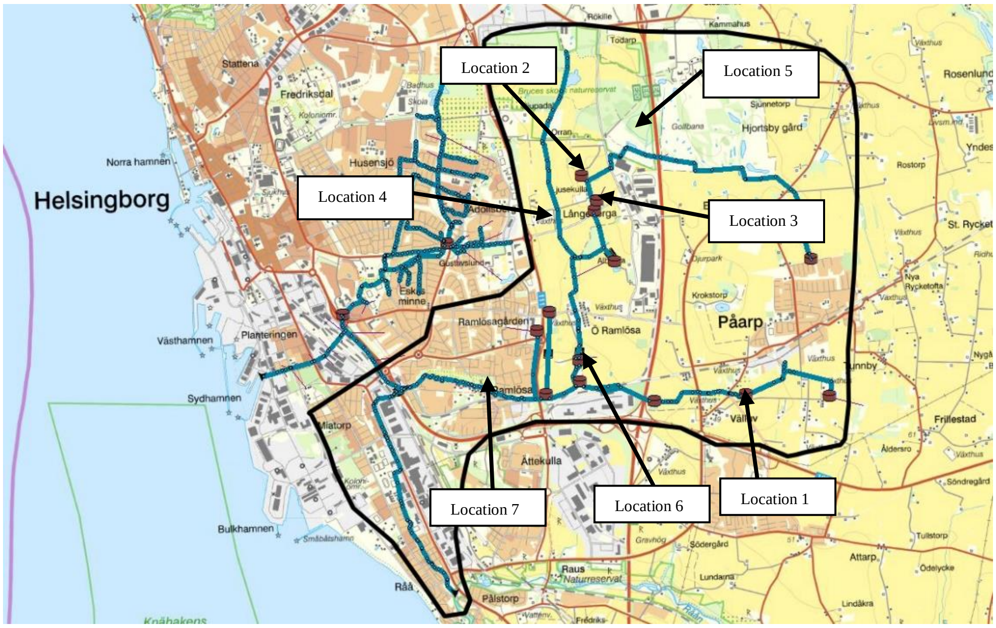


Figure 4-3: Map shows the locations of implemented stormwater control measures. Blackline delimits the extension of the Lussebäcken catchment.

In location 1, there is an artificial built pond close to the Påarp town . The output flow is regulated using a fixed metal structure. If the water level raises during an episode, it can overflow the weir Figure 4-5.

Figure 4-4: Location 1. Påarp pond

Figure 4-5: Outlet of the pond with a control structure

Around location 2, two-stage ditches have been implemented, but unlike the traditional design, the lower stage it is permanently flooded to improve nutrient elimination. Other parts of the catchment have undergone a reconstruction give them a more natural shape.

Figure 4-6: Location 2, two-stage ditch

Figure 4-7: Location 3, Reshaped ditches

This is the case of the location 2, Figure 4-7, in which the traditional shallow ditch was replaced with a wider-angle ditch.

Figure 4-8: Location 4, classic ditch

Figure 4-9: Location 2, two-stage ditch

The traditional shape of a draining ditch can be seen in **Error! Reference source not found.** where the narrow ditch with a steep wall angle is presented.
Currently, measurements of water level are being recorded through different positions to be available to compute the water flow (Figure 4-9).

An artificial build wetland is depicted in Figure 4-10. This wetland has a regulated inflow but when there is a rain episode, it acts as a flood plain. In the beginning of the first culverted section under the Ramlösa, there is a grid to prevent gross materials entering the section as seen in Figure 4-11.

Figure 4-10: Artificial wetlands

Figure 4-11:Beginning of the culvert section in the Ramlösa

4.3.1 Flow measurements

The river flow level is being measured by the municipality of Helsinborg for the purpose of determining how the water level is changing in the river. The water level (i.e. stage) and flow data is being collected across the stream to construct a rating curve (i.e. plotting river discharge and stage). Thus, the river flow can be either measured direct or indirect. For the case of indirect measurement, the water level is measured across the river and then use stage-discharge relationship in order to determine the river flow, it is also involving the use of hydraulic structures such as installing weir or other obstructions in river and the river discharge determined by using energy equation relationship. However, for the case of direct measurement, the water level and its corresponding discharge are recorded automatically at any location across the river or stream by installing automatic instrument in the river (i.e. stream gauge). Recent researchers (Fenton and Keller, 2001, Herschy, 2008, Rantz, 1982), through various studies on streamflow measurement , have been described the commons different technics for flow measurements such as a velocity area method, Acoustic Doppler Current Profiler (ADCP), Discharge measuring structures, Dilution, current meter, staff gage and so forth. The existing water level data and flow discharge for different locations were provided by NSVA. The water levels compared with the rainfall episodes show how fast the system reacts.

4.3.2 Literature review

A general review of the literature to the sustainable drainage system, field investigation and data collection were used to gain further information about the thesis topic and physical characteristics of the catchment. A literature review was done throughout finding the relevant materials (scientific papers, journals, report and previous studies) related to Sustainable Urban Drainage System, Stormwater Control

Measures, Best Management Practices, Low Impact Development, SuDS Design and Construction, Ponds/Wetlands, Residence Time and Two-Stage Ditches. Web search engine such as Google scholar and Lub search were used and MIKE Urban software to simulate the network.

4.4 Data analysis and processing

4.4.1 Rainfall data

Rainfall analysis was done through observed rainfall data series which were collected every 15 minutes during the period from 1995 to 2018 at Helsingborg A station which is located at latitude: 56.0304, longitude: 12.7653, between Påarpsvägen and Österleden, provided by SMHI. The analysis was performed in order to identify extreme precipitation events for rainfall time series, that later could be used as the input for the model to simulate runoff. During analysis, the accumulation of precipitation for different periods was determined and analysed (

Table *4-2*). Based on information from the previous model of the Lussebäcken provided by the NSVA, the most significant rainfall episodes that were used during simulation of runoff before 2008 were taken.

Table 4-1: Accumulated extreme precipitation events used for previous model (NSVA)

Date of the episode	Total precipitation (mm)	Maximum intensity (mm/5 min)
15/9/1994	75,8	12
5/7/2007	46	5
11/8/2007	39,5	1,5

Table 4-2: The highest Accumulated rainfall depth for different periods 2, 6, 12 hours, 1 day and 5 days),

2 hours	**Depth (mm)**	**Daily**	**Depth (mm)**
2013-08-14 08:00	25.9	2008-08-04	79.7
2013-08-14 06:00	24.8	2013-08-14	75.4
2008-08-04 08:00	24.7	2007-07-05	54.4
6 hours	**Depth (mm)**	**12 hours**	**Depth (mm)**
2013-08-14 06:00	55.7	2013-08-14 00:00	72.1
2007-07-05 12:00	37.4	2008-08-04 00:00	49.3
1999-08-09 12:00	31.7	2007-07-05 12:00	37.7

Apart from this, the other extreme rainfall events were obtained through rainfall data by analysis of accumulated rainfall from different periods. In order to find the highest

5 days	Depth (mm)
1999-08-15	102
2013-08-11	97.6
2008-08-02	95.3

episodes. The cumulative precipitation for 2, 6, 12 hours, 1 day and 5 days were plotted against time and they can be found in the Appendix A.1

4.4.2 The water level in the Lussebäcken

The water level was analysed from previous hourly water level data collected from October 2018 to February 2019 at different 4 water level measurement stations (i.e. naturcentrum 1, 2, 3, and 4) located in the Lussebäcken stream (Figure 4-12. *Water level measurement sites.*) around the Långeberga area (location 2) and data for analysis were provided and given by the municipality of Helsingborg.

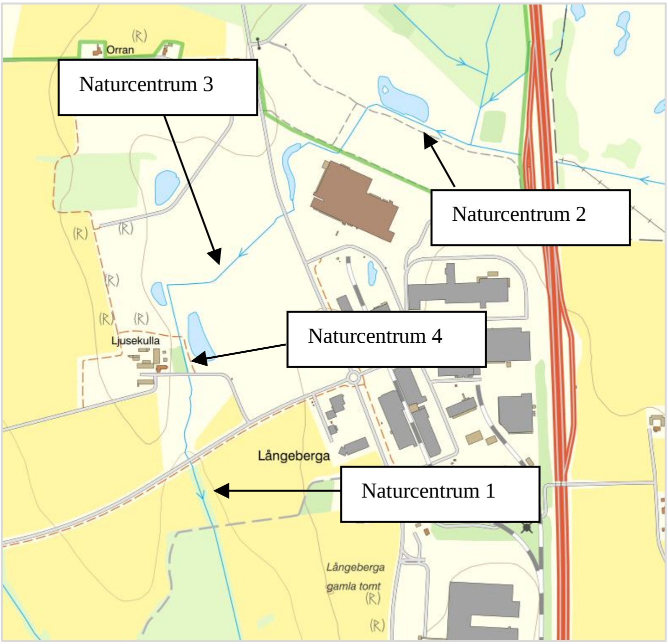
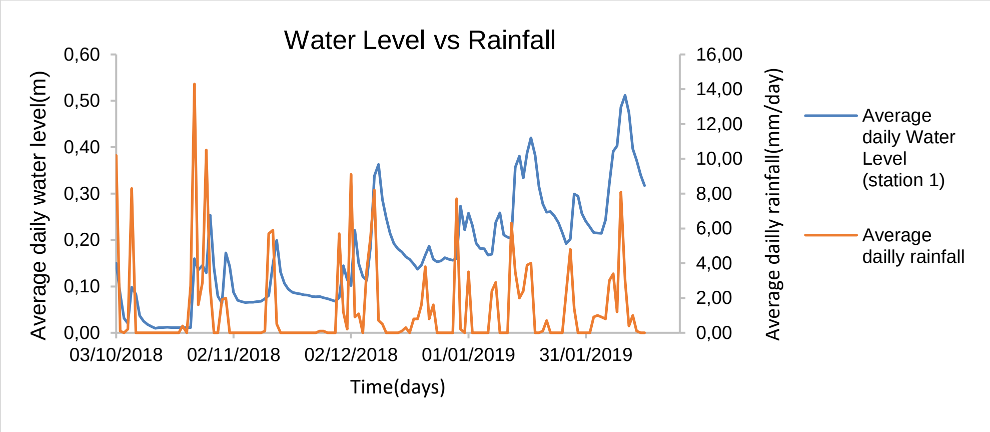
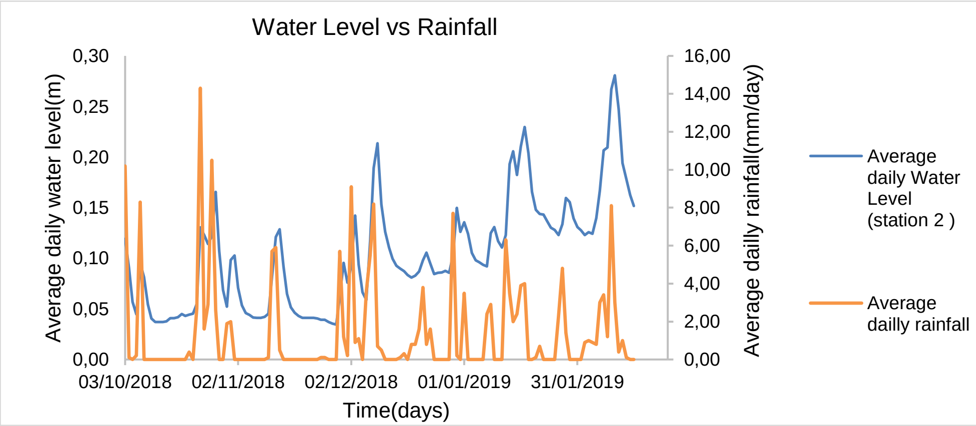


Figure 4-12. Water level measurement sites.

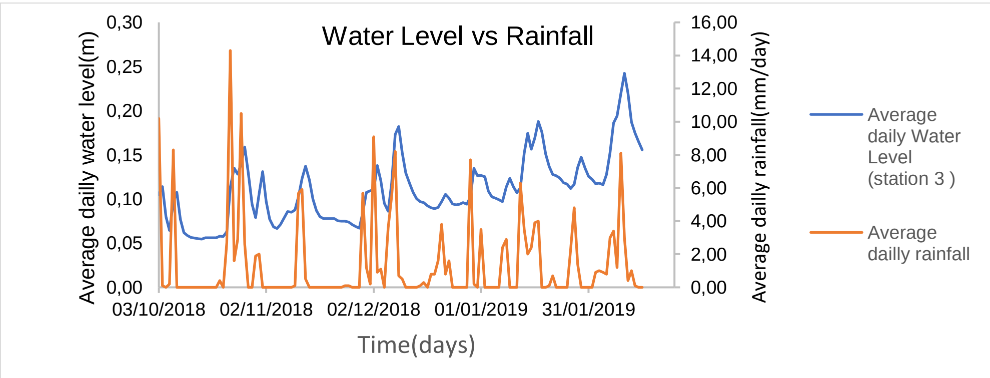
All stations were installed for the purpose of tracking water level in the stream and one of them (Station 4) is installed in the vicinity of the modified channel with two-stage ditches to measure water level in ditches. The analysis of water level provided information about the change of water level in relation to rainfall. To do this, water level data from 4 stations and average daily rainfall were plotted against time Figure 4-13.



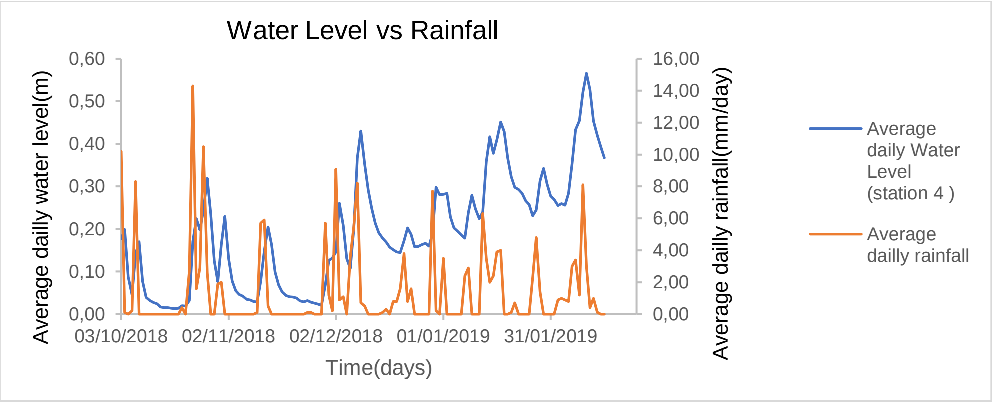
A. Variation of water level in Naturcentrum 1



B. Variation of water level in Naturcentrum 2



C. Variation of water level on Naturcentrum 3



D. Variation of water level on Naturcentrum 4

Figure 4-13: variation of annual water level for four different locations in Lussebäcken against time

Based on the variation of water level in the Lussebäcken from on 4 stations an increase of the water level along time.

4.5 Model flow on constructed sustainable drainage system

4.5.1 Mike Urban model

This section describes the modelling of flow with constructed sustainable drainage system ponds and wetlands in the Lussebächen catchment. In order to simulate the runoff and hydraulic of the flow in the pipe network, the previous MIKE URBAN model was used, and it was originally provided by NSVA. It was vin created in 2008 so the model was not up to date in terms of the ponds represented in it.

The model also included a part of a different catchment that was included in the simulation because a part of the waters of the Lussebäcken can overflow into this catchment. Since the analysis of the catchment was done inside the Lussebäcken catchment no analysis was done in the other parts of the modelled network.

4.5.2 Verification of the model

First of all, validation of the model was needed to ensure that the water levels and flows would approximate to the real values. For that purpose, a set of data from 23-01-2008 to 4-06-2008 was provided by the NSVA which consists of different flow measurement points. The two measurement points are the discharge of the Valluvs pond and the flow in the main Lussebäcken stream up the Österleden highway.

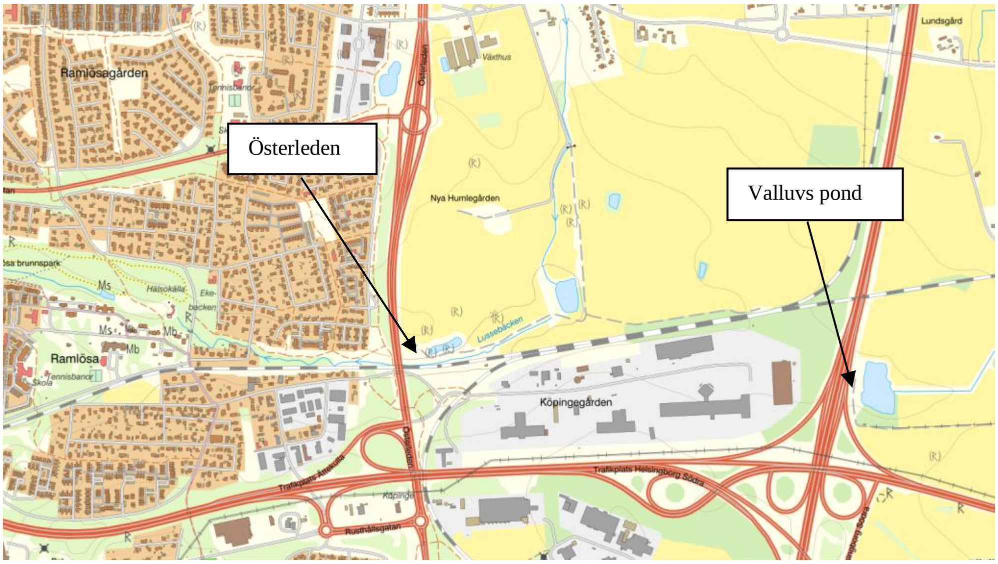


Figure 4-14. Water flow measurement points for validation of the model.

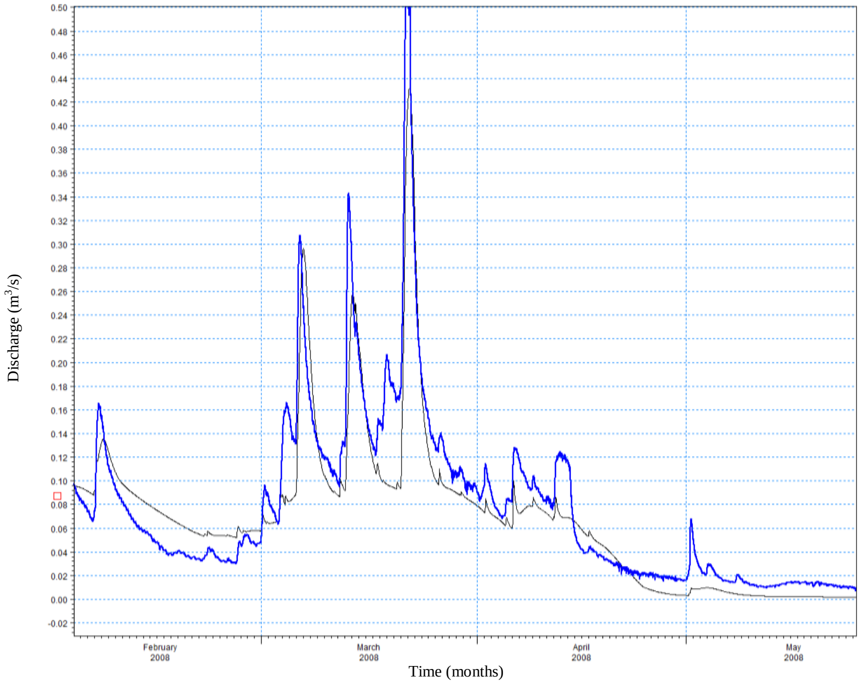


Figure 4-15: Valluvs pond discharge. The blue trace corresponds to the real data and the black trace corresponds to the simulated data.

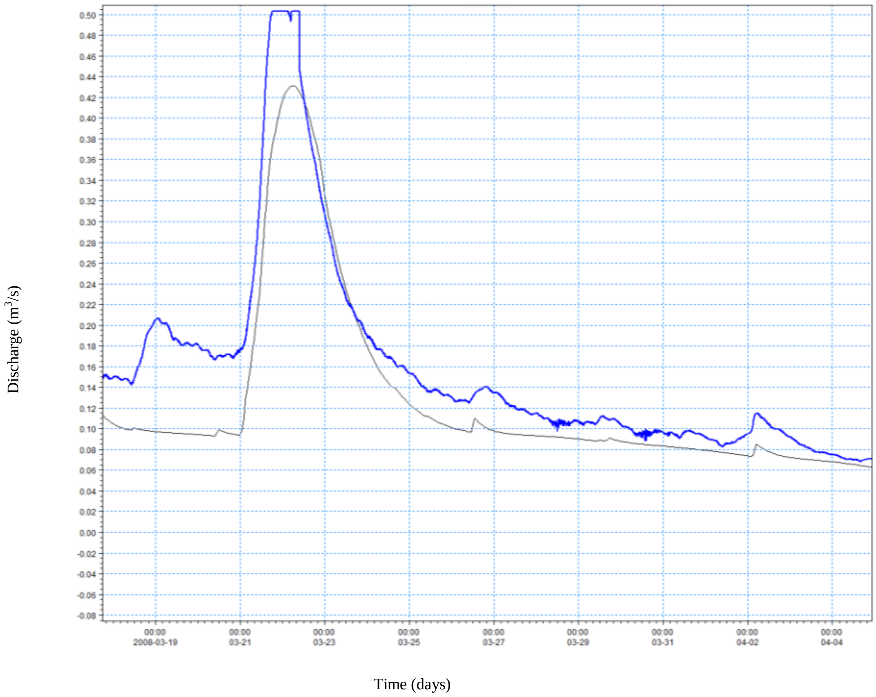


Figure 4-16. Detail of the Valluvs pond discharge around 23-03-2008. The blue trace corresponds to the real data and the black trace corresponds to the simulated data.

The model tends to underestimate the discharge rate. The error percentage between the maximum flow of both time series was determined. In the case of the Valluvs pond the maximum error is located around 23-3-2008 with a 13,6% of underestimation from the modelled data. In the case of the Österleden highway, the underestimation is 32,6% of less flow than the real one. It is worth to mention that the measure in Österleden is very noisy and that factor could contribute to the imprecision of the data. Although the deviation of the modelled data, the reaction times after rainfall episodes follow closely the real data. The following Figure 4-15 and Figure 4-17 describes the comparison between modelled flow with the measured flow at Valluvs pond and Österleden. The blue trace corresponds to the real data while the black trace is the simulated data for the same period of time

Detail graph of the evolution of the discharge in which the maximum discharge divergence values are depicted in the Figure 4-17 for the Vallüvs pond and in Figure 4-18 for the Österleden pond.

Österleden

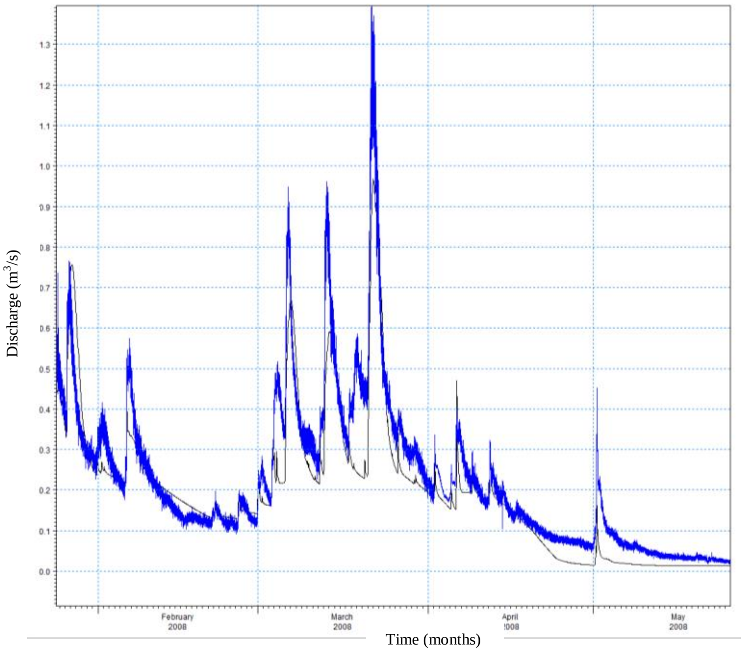


Figure 4-17. Österleden pond discharge. The blue trace corresponds to the real data and the black trace corresponds to the simulated data.

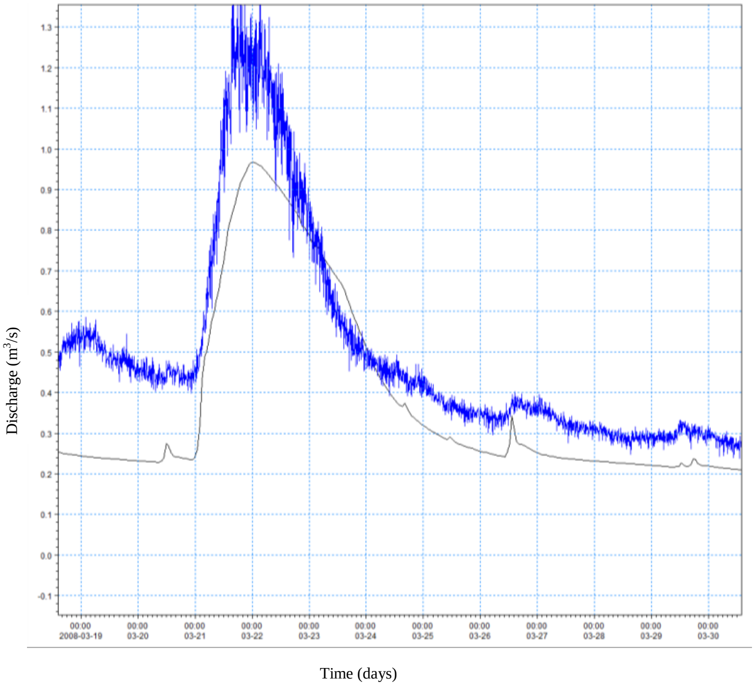


Figure 4-18. Detail of the Österleden pond discharge around 23-03-2008. The blue trace corresponds to the real data and the black trace corresponds to the simulated data.

4.5.3 Update of the model to 2019 status

From the year 2008 onwards a various set of measures were built in the Lussebäcken catchment with environmental goals to improve water quality. These structures can also be used to store water in the case of heavy rainfall. The result of combining uses is a multipurpose structure that serves both as an environmental water reserve for fauna and vegetation and to prevent sudden surges in the water flow during storms. The main locations are found on Figure 4-19.

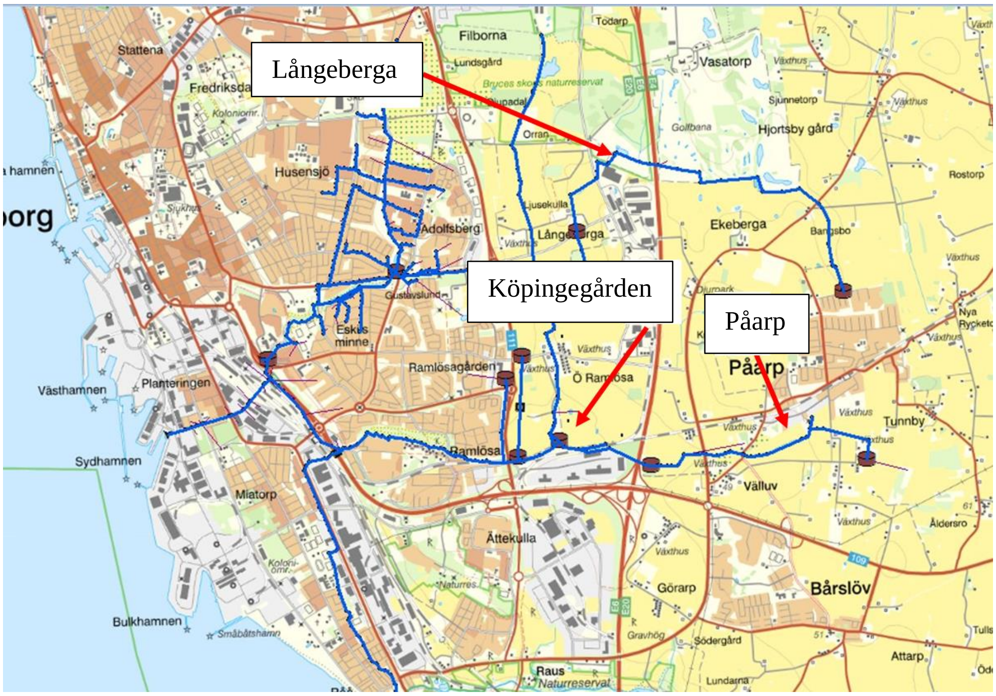


Figure 4-19: Map of locations of the new ponds.

Around Långeberga a set of ponds have been dug in order to control the runoff water flowing into the main stream. The aim of those is to compensate the loss of pervious areas around the Långeberga industrial area. The intention is to store the additional runoff water in a pond

There are also two stage ditches that have been constructed with the intention of mimicking the floodplain of a natural water stream. Vegetation has been planted in the middle of the lower plain to reduce water flowing speed and for environmental reasons. In the model, the location 1 is modelled as two additional ponds.

The pond north of Ljusekulla has a capacity of 6.500 m3 and drains an area of 15.24 ha. The other pond drains the Långeberga industrial area and has a volume of 12,000 m3 and receives the rain runoff of a 50.6-ha area.

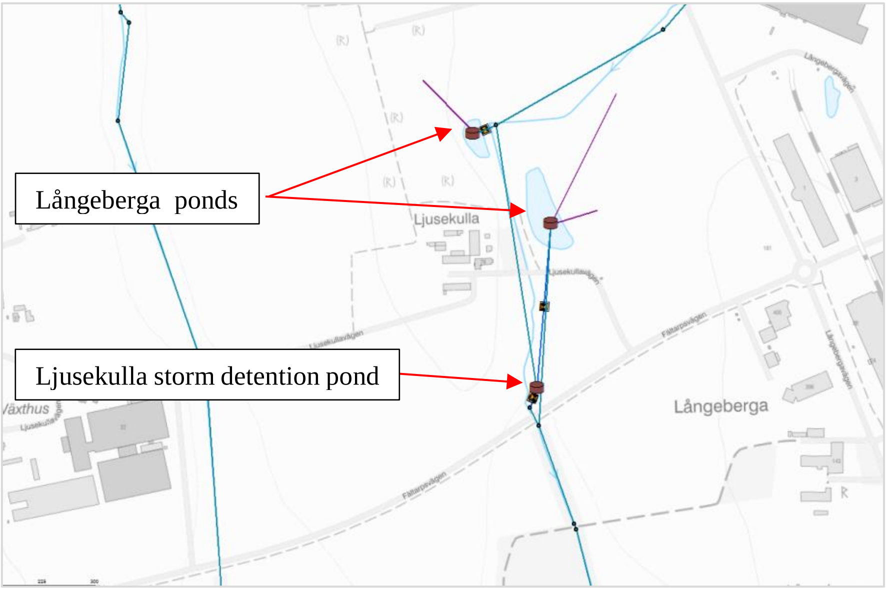


Figure 4-20. Detail of the modelling of the ponds in the Långeberga area. Marked with red arrows are the the modelled ponds.

In Påarp, an artificial pond of 9000 m3 of capacity has been built in line of the water stream with a weir in the end of it to control its outflow. The weir has a height of 30 centimetres above the bottom level of the pond, as checked in the field observations.

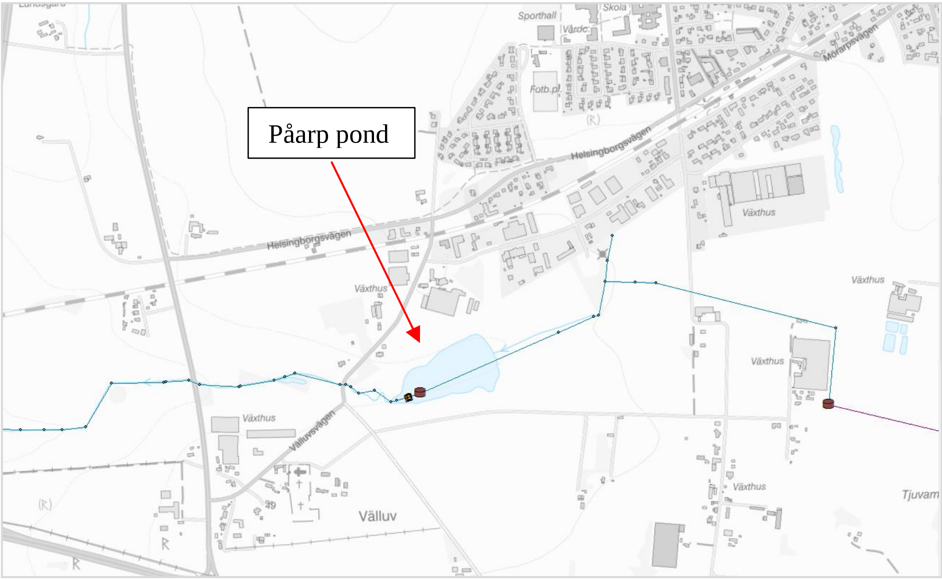


Figure 4-21. Detail of the modelling of the pond in the Påarp area. Marked with red arrows are the the modelled ponds.

Lastly, around the Köpingegården area an artificial wetland has been dug in the ground with two different ponds at different heights. The estimated total volume for these two structures is 1390 m3.
The structure has an inflow in the form of a weir which takes a part of the flow of the main stream and deviates it through the wetland.
In case of high water, the surrounding area serves as a flooding plain.

For the purpose of modelling this wetland, the storage volumes have been joined into a one single pond with a total capacity of 1390 m3.

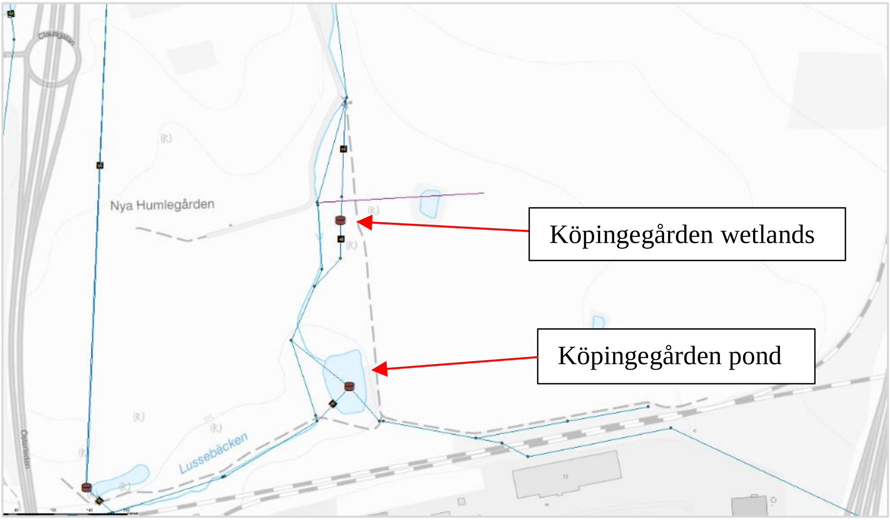


Figure 4-22. Detail of the modelling of the ponds in the Köpingegården area.

One of the aims of this project is to optimize the use of the already constructed SuDS to prevent further flooding. After consulting with the local authorities, the available locations where there was possibility to modify, were the Påarp and Långeberga ponds, discarding the Köpingegården pond and its surroundings, including the wetlands, because of the difficulties to modify it since it is a protected area for natural wildlife. Regarding Långeberga area, the ponds are located before a storm detention structure, Ljusekulla pond which has plenty of capacity, so no big impacts on reducing the peak flows downstream were to be expected.

The main proposal is to optimize the Påarp pond since it implies regulating the flow downstream of the southern branch of the Lussebäcken. A raised weir 70 centimetres over the existing one and with an orifice of 50 centimetre located 20 centimetre above the bottom level were added in the model at the outflow control structure of the pond.

4.6 Theoretical calculation of the Påarp pond

After evaluating the possibilities for modification of the current ponds, the Påarp pond was chosen. The incoming flow generated by rainfall from the upstream catchment to the pond should be determined and this was done by determining the runoff for the design rainfall event of 5 of July of 2007. The continuity equation and a conceptual flow equation were applied. Based on the catchment area calculated using MIKE URBAN, the area of upstream sub-catchment was obtained.

For the Påarp sub catchment area, the continuity equation is defined as:

$$\frac{dS}{dt} = Q_{in}(t) - Q_R(t) \quad (4.6.1)$$

Where, dS is the change in the amount of water stored in a given catchment(m3/s),
$Q_{in}(t)$ is the incoming flow generated by the rainfall (m3/s) at given time period
Q_R(t) is the runoff from the catchment (m3/s) at a given time period
dt is the change of time; time step (second).

The evaporation and the infiltration terms are no taken into account because since this calculations focus on extreme rainfall episodes, the mentioned terms are negligible.

Based on a known rainfall intensity, the amount of incoming flow generated from rainfall which is likely to contribute to the surface runoff in a determined sub-catchment of Påarp was calculated using a rational method. The application of rational method is acceptable for design of water related structures. But, the application of this method is limited and highly recommended for estimating runoff within a small catchment (Thompson, 2006). It was applied during designing Påarp pond because the upstream catchment is comparably small, it is about 3.5 km^2.The rational method is grounded on the simple mathematical equation formed by the combination of catchment area, rainfall intensity and runoff coefficient. It is expressed as:

$$Q_{in}(t) = I(t) \times \varphi \times A \quad (4.6.2)$$

Where, $I(t)$ is the rainfall intensity, mm/h
φ is the runoff coefficient, dimensionless
A *is* the catchment area(ha).

From the above equation, the amount of incoming flow generated by rainfall into the sub-catchment at upstream of a Påarp pond was obtained. After this was calculated, getting the flow from an upstream to the pond was problematic. For this case, linear reservoir theory was applied. This method is normally based on assumption and concept that the catchment describes as reservoir. Then, the linear relationship equation between storage (S) and outflow from a catchment was produced. This produced linear equation is mathematically obtained as:

$$Q_R(t) = k_R \times S(t) \quad (4.6.3)$$

Where k_R is called a storage coefficient, it describes the influence of smoothness of the catchment on the rainfall-runoff process and $S(t)$ is the storage at any time t.

By substituting equations (2) and (3) into (1), the continuity equation is thus expressed as:

$$\frac{dS}{dt} = I(t) \times \varphi \times A) - k_R \times S(t) \qquad (4.6.4)$$

It is clearly known that water stored in a pond varies according to the incoming flow. However, the amount of water that has been stored and how the outflow from a pond storage varies, was obtained through combining continuity equation and flow equation.

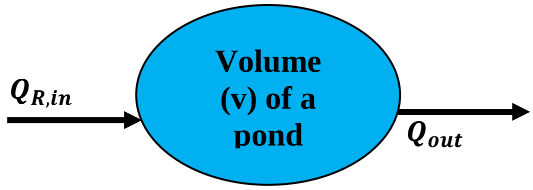


Figure 4-23: typical sketch of pond at steady state conditions

As it can be shown on the above figure, the water storage volume is defined with relationship between incoming flow and outgoing flow. Therefore, the continuity equation was expressed as:

$$\frac{dV}{dt} = Q_R(t) - Q_{out}(t) \qquad (4.6.5)$$

Where, V is the volume of water in the pond (m^3)

$Q_R(t)$ is the runoff from the catchment to the pond at any given time period

$Q_{out}(t)$ is the outflow from a pond,

dt is the change of time; time step.

To control and optimise the amount of water that flows out from the pond storage , as well as hydraulic load downstream of the pond and flooding, it would be emphasised to discharge water from a pond through a circular orifice. Currently, that Påarp pond discharges its waters through a metallic structure that regulates the outflow. An orifice was added the to the designed structure to be able to maintain a constant environmental flow. In order to design orifice, quasi-steady condition was assumed and the released water from the storage will be depended and controlled by on upstream water depth *(h)*. Thus, the outflow from the pond depends on h and the outflow from orifice (Q_{out}) can be expressed as:

$$Q_{out} = C_D \times A_0 \times \sqrt{2gh} \tag{4.6.6}$$

Where, C_D is a discharge coefficient,

A_0 is the cross-sectional area of the orifice, calculated based on the diameter of the opening (D_o)

Thus,

$$A_0 = \frac{\pi D_0^2}{4} \tag{4.6.7}$$

Based on collected rainfall data from SMHI , the rainfall intensity is recorded every 15 minutes. As the rainfall intensity changes with time , the water level in the pond will also change. The governing equations to determine runoff in a catchment and storage volume could be solved numerically and simultaneously. This is implied that the solution is referred from calculated solution of the previous time steps.
Therefore, the discretization of the runoff from a catchment is expressed as:

$$S_{k+1} = S_k + \Delta_t\{Q_{in,k}(t) - k_R S_k(t)\} \tag{4.6.8}$$

Thus, $Q_{in,k}(t) = I(t)_{,k} \times \varphi \times A$

$Q_{R,k+1} = k_R \times S_{k+1}(\mathrm{t})$

where the subscript k is the time step number and Δt is the time step.
By setting conditions, where k = 0, so that $t = 0$ and $S_o = 0$. The catchment runoff can be obtained iteratively at each time step. Thus, the numerical and governing equation describing the water flow through the pond storage has been computed.

$$h_{k+1}(t) = h_k(t) + \frac{\Delta t}{A_B}\{Q_{R,k}(t) - C_D A_O \sqrt{2gh_k(t)}\} \tag{4.6.9}$$

Where $h_k(t)$ could be h_0 at t = 0, and A_B is the surface area of the pond.
This above equation is only defining the water flow from pond through the orifice. In the case that water level exceeds the maximum design level in the pond, a weir is needed to control the overflow. To discharge the overflow in Påarp pond, a rectangular weir was designed and sized. The general mathematical expression of the rectangular weir is the following:

$$Q_w = c_e \frac{2}{3}\sqrt{2g}B_w(h - H)^{3/2} \tag{4.6.10}$$

Where, Ce is a discharge coefficient,

B_w is the length of the weir,

H is the distance from the bottom of the pond storage to the weir crest.

The overflow is changing with rainfall intensity, the governing equation to determine the relationship between water level and outgoing discharges was altered accordingly:

$$h_{k+1} = h_k + \frac{\Delta t}{A_B}\left(Q_{R,k} - C_D A_O \sqrt{2gh_k} - c_e \frac{2}{3}\sqrt{2g} B_w (h_k - H)^{3/2}\right) \qquad (4.6.11)$$

Where, $h > H$

Based on collected information for the pond Påarp sub-catchment as well as using above formulas, height and width of the weir were obtained and then the size of orifice was also determined. Rainfall intensity for the 5th of July 2007 episode with an observation period of every 15 minutes within 24 hours was used as input rainfall time series in a catchment. The selection of this rainfall time series was motivated because it this episode has a long duration before the peak rainfall is produced, leading to a high runoff due to the moisture of the soil. The total area of an upstream sub-catchment was 3,491,700 m^2 (3,491ha), this area was obtained from delineated sub-catchment areas by MIKE URBAN. In addition, the runoff coefficient of the area was then chosen based on the general runoff coefficient table suggested by Thompson, D.B. (2006)(see appendix). The calculations are summarized in the following Table 4-3*: Table shows the size of Påarp pond, outlet and rectangular weir*

Table 4-3: Table shows the size of Påarp pond, outlet and rectangular weir

Pond Size		**Outlet (Orifice)**		**Rectangular Weir**	
Pond Total Volume(V)	9000 m^3	Diameter (Do)	0.5 m	Water Level up to weir Crest(H)	0.32 m
Surface Area (As)	28125 m^2	Discharge Coefficient (Cd)	0.61	Height of The Weir(H-H)	1 m
Length of The Pond	162 m	Gravitational force	9.81m/s^2	Adjusted Width of the Weir(Bw)	10 m
		Area(A0)	0.196 m^2	Weir Discharge Coefficient (Ce)	0.65
		Outlet Position	0.2 m		

5 Results and discussions

5.1 Results of simulated scenarios with rainfall episodes

This section describes the different scenarios simulated with rainfall episodes during modelling of constructed ponds/wetlands and display the outcomes from them. The three scenarios were split and simulated for 2 rainfall extreme episodes 05/07/2007 and 15/09/1994 respectively. The purpose of these scenarios is to compare the effect of extreme rainfall on the current storm water control measures and after implementation of new structural sustainable drainage system ponds and wetlands. The following scenarios are simulated in MIKE URBAN.

- **First scenario: simulation of the model without additional measures**
  The previous model of sustainable drainage systems which was constructed in 2018 and provided by NSVA was simulated with the above-mentioned rainfall episodes. This provides the status of the current situation of the catchment with existing control measures.
- **Second scenario: simulation of the model with additional ponds**

  In this scenario, the model was modified by adding new storm water measures. The ponds are added around the upper part of the catchment. After this, the new model was obtained and simulated for the rainfall episodes

- **Third scenario: Proposed optimization of the ponds**.

  Modified model with the additional pond. Using the theoretical calculations, the Påarp pond is in section 5.5 The pond located at Påarp was modified by increasing the height of the weir and adding an orifice to maintain the water flow even during the dry season.

As it has been mentioned before, there were more severe rainfall events after 2008, like the ones analysed in the chapter 5.3.1, but when trying to simulate those episodes MIKE URBAN could not handle such a large amount of water inflows.
Because of that, the simulated episodes correspond to the event that were possible to simulate in its whole extension.

To evaluate the performance of the Lussebäcken the discharge at three representative points which are the two main branches of the catchment and the outflow of the Köpingegården pond in which the two branches merge into the main stream. A map of the locations is displayed in Figure 5-1.

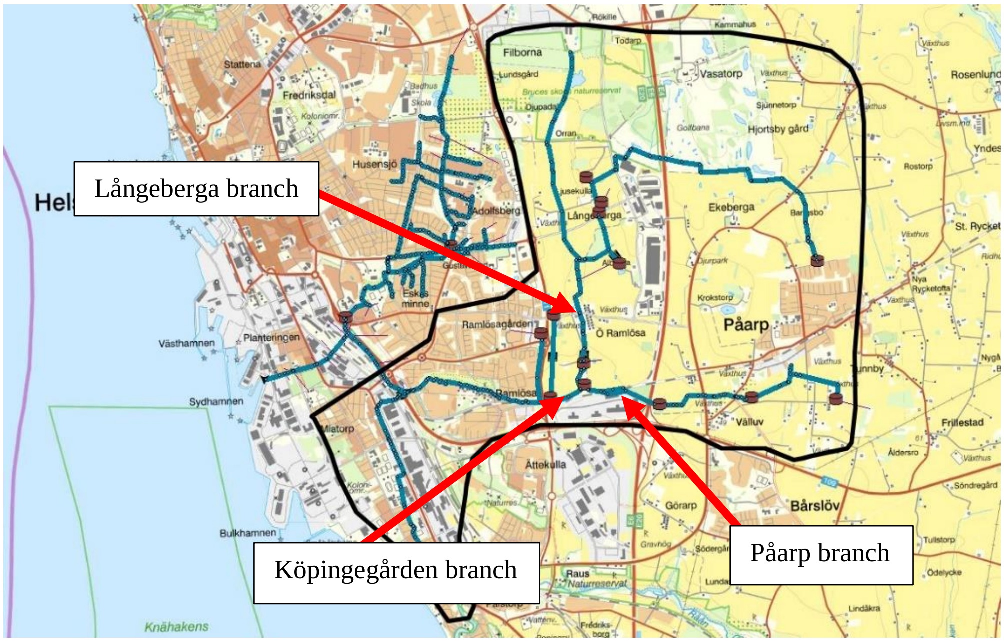


Figure 5-1: Map of the location for the selected flow measurements

The variations of the peak flow of Köpingegården pond outflow with the different simulation can be observed in the Table 5-1.

Table 5-1: Simulated peak flow at the Köpingegården pond outflow. Change compared with the scenario 1.

Scenarios	Peak flow (m3/s)		Peak flow (m3/s)	
	15/09/1994	Change	05/07/2007	Change
Scenario 1 (2008 Layout)	3,56		4,76	
Scenario 2 (2019 Layout)	2,78	-21,9%	3,39	-28,8%
Proposed modifications	2,57	-27,8%	3,19	-32,9%

The detailed evolution of the discharge for each episode and each scenario can be found in the Figure 5-2, Figure 5-3 (2008 situation),

Figure 5-4, Figure 5-5 (2019 situation), Figure 5-6 and Figure 5-7 (Proposed modifications situation). The green trace corresponds to Långeberga incoming branch, the blue trace corresponds to the Påarp incoming branch. The black trace corresponds to the outflow of the Köpingegården pond.

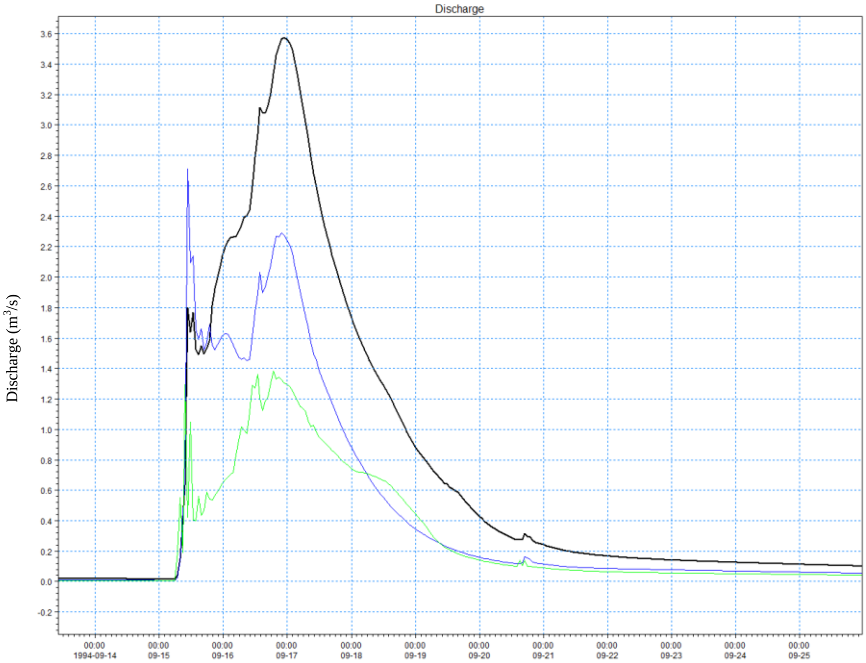


Figure 5-2: Simulated flows for 15/09/1994 in the 2008 scenario. Black trace corresponds the outflow of the Köpingegården pond. The green trace corresponds to Långeberga incoming branch, the blue trace corresponds to the Påarp incoming branch.

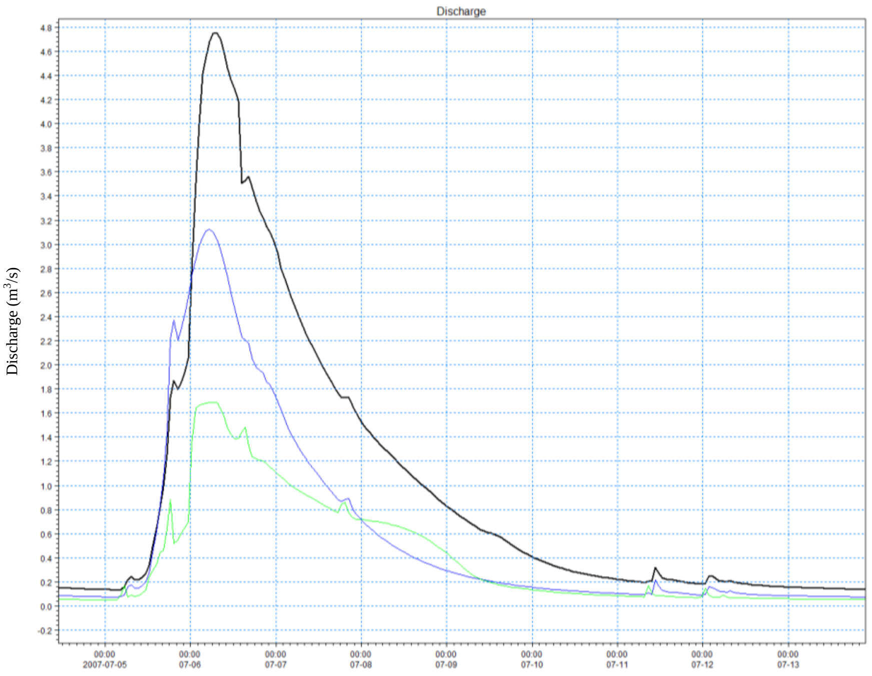


Figure 5-3:Simulated flows for 5/07/2007 in the 2008 scenario. The green trace corresponds to Långeberga incoming branch, the blue trace corresponds to the Påarp incoming branch .

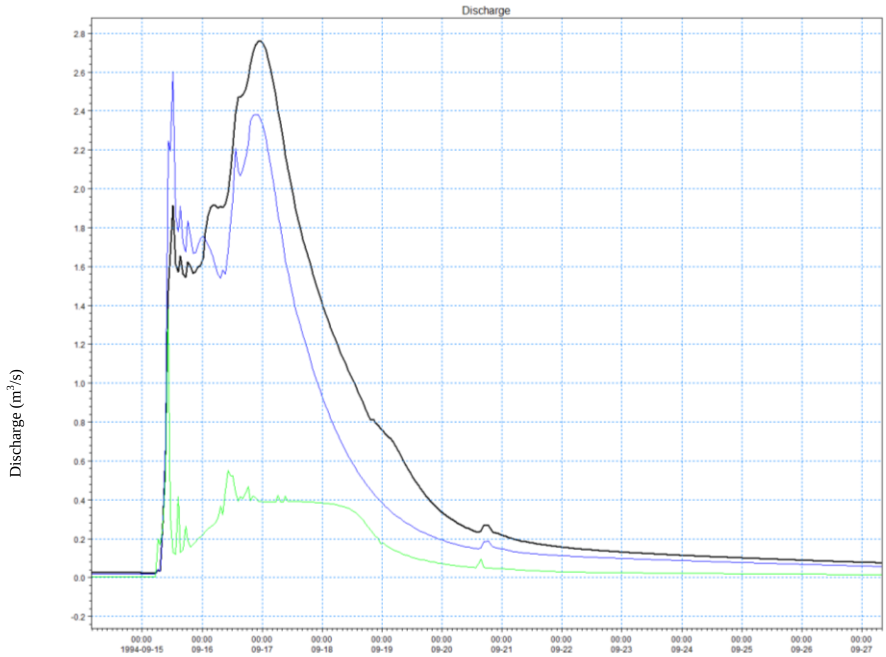


Figure 5-4: :Simulated flows for 15/09/1994 in the 2019 scenario. The green trace corresponds to Långeberga incoming branch, the blue trace corresponds to the Påarp incoming branch.

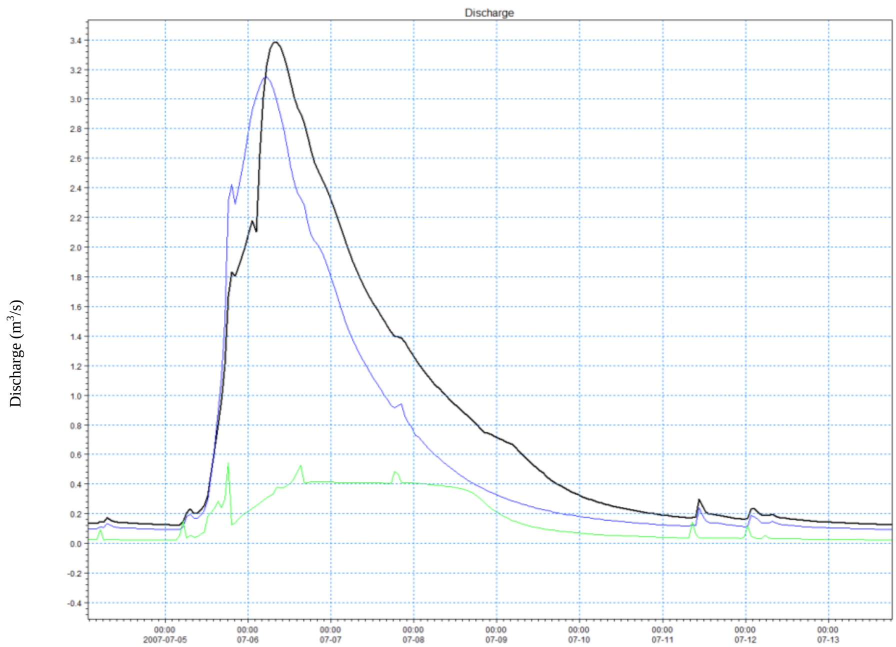


Figure 5-5: Simulated flows for 5/07/2007 in the 2019 scenario. The green trace corresponds to Långeberga incoming branch, the blue trace corresponds to the Påarp incoming branch.

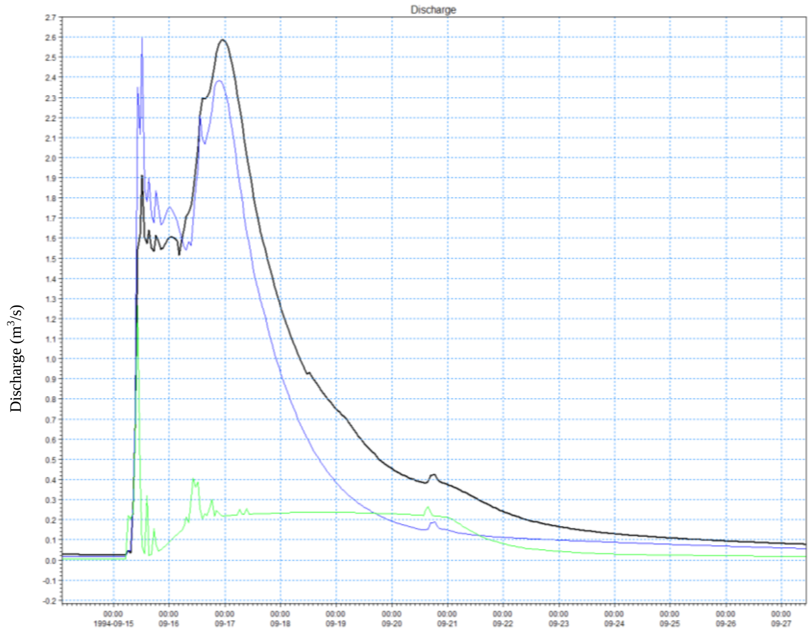


Figure 5-6: Simulated flows for 15/09/1994 in the proposed scenario. The green trace corresponds to Långeberga incoming branch, the blue trace corresponds to the Påarp incoming branch.

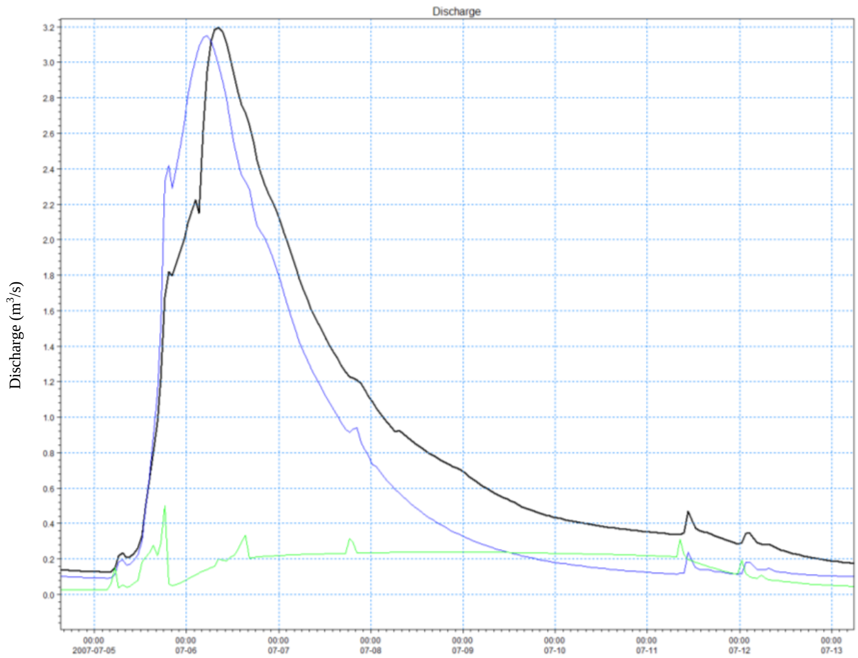


Figure 5-7: Simulated flows for 5/07/2007 in the proposed scenario The green trace corresponds to Långeberga incoming branch, the blue trace corresponds to the Påarp incoming branch.

5.2 Köpingegården pond water levels

The analysis of the water level at the pond where the main branches of the catchment are merged was considered important to study as this pond would be prone to flooding. The maximum water level at the Köpingegården has been reduced below the ground level (26,2 m) for the 2007 event for the simulation of the proposed modifications scenario.

In the 2008 scenario flooding could be observed, but after the addition of flow control measures the flood is completely eliminated for the 5/7/2007 episode.

Table 5-2. Maximum water levels simulated at Köpingegården pond

Scenarios	Maximum water level at the pond
First scenario (2008 Layout)	26,3
Second scenario (2019 Layout)	26,08
Proposed modifications	26,06

The red line in the graphs represents the ground level of the pond. Water levels above it represent flooding.

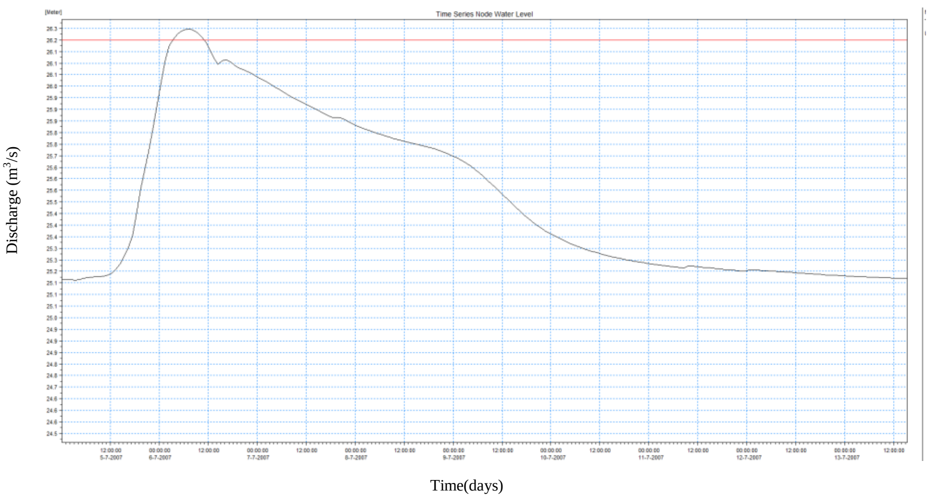


Figure 5-8. Water level at the Köpingegården pond for the 2008 layout

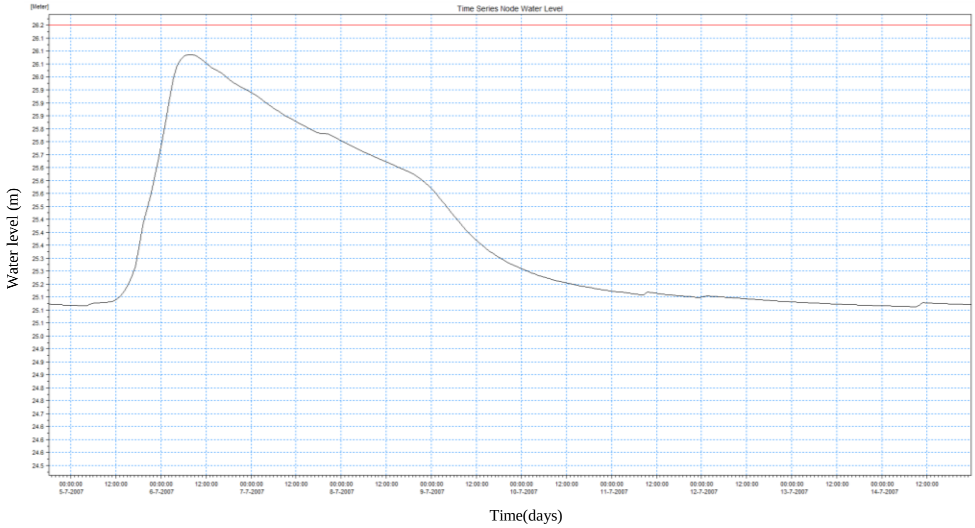


Figure 5-9. Water level at the Köpingegården pond for 2019 layout

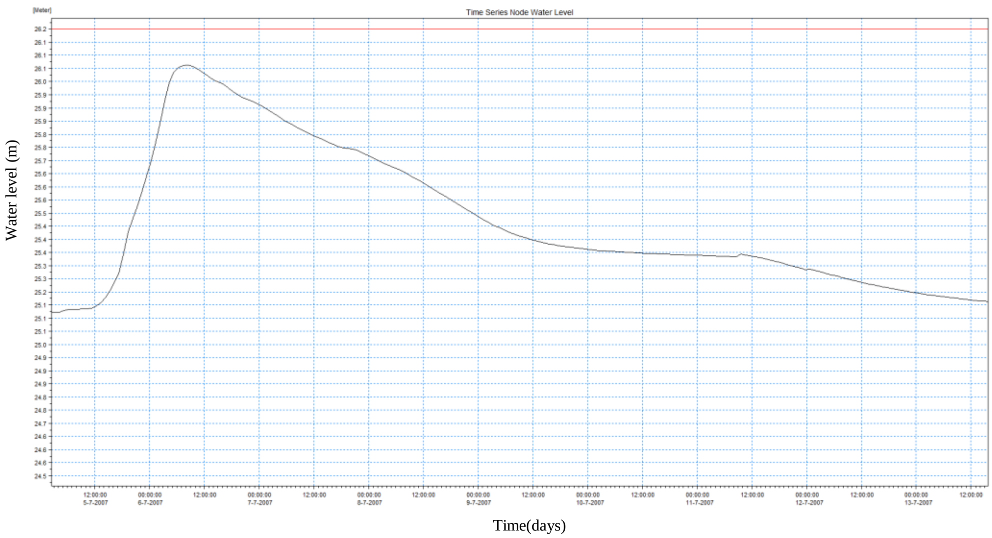


Figure.5-10. Water level at the Köpingegården pond for the proposed modifications scenario layout

5.3 Ramlösa Ravine water levels

One of the most critical sections of the catchment is the Ramlösa ravine, since it consists of a culvert section through a pipe and the slope get as high as 13,70 cm/m which implies high speed flows and consequently risk of flooding.

This section is situated after the Köpingegården pond and it is the beginnig of the low part of the section. Problems of flooding were reported by the local authorities during heavy rainfalls. In the Ramlösa Brunnspark there are historical buildings that make this section of the network even more critical when it comes to flooding prevention. By adding storage capacity and source control in the upper catchment the water levels are reduced.

Table 5-3. Maximum water depth and time for the Ramlösa culvert

Scenarios	Time of the maximum level	Water depth (m)
First scenario (2008 Layout)	5:47 h	1,17
Second scenario (2019 Layout)	6:19 h	0,86
Proposed modifications	7:19 h	0,83

The effect of the water retention in the ponds delays the peak flow. For the 5-7-2007 episode in the table can be seen the maximum depth of water in the Ramlösa culvert. In the Figure 5-12, Figure 5-13, Figure 5-14 the water levels through the culvert are represented at their highest level.

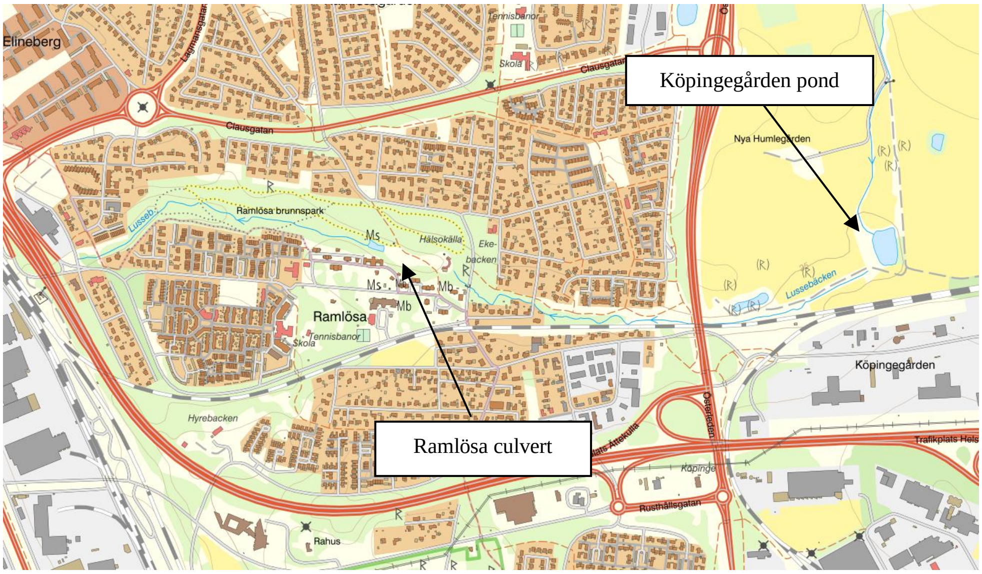


Figure 5-11. Map of the location of the Ramlösa culvert

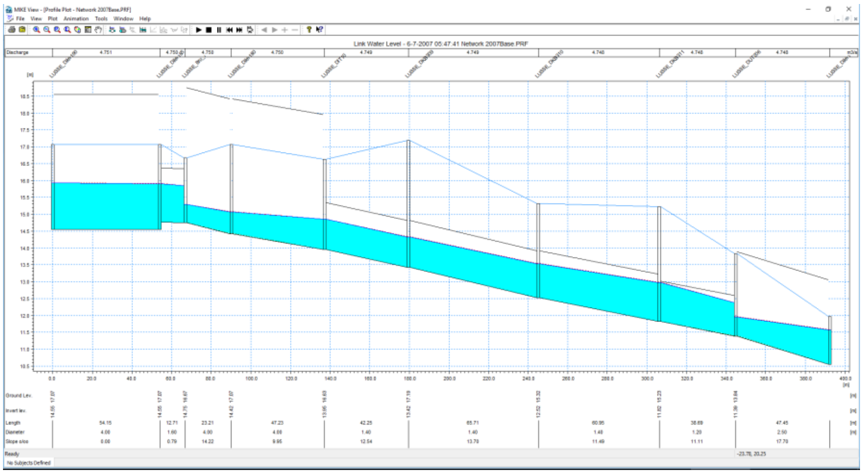


Figure 5-12. Water profile of the Ramlösa culvert for the 5/07/2007 in the 2008 scenario

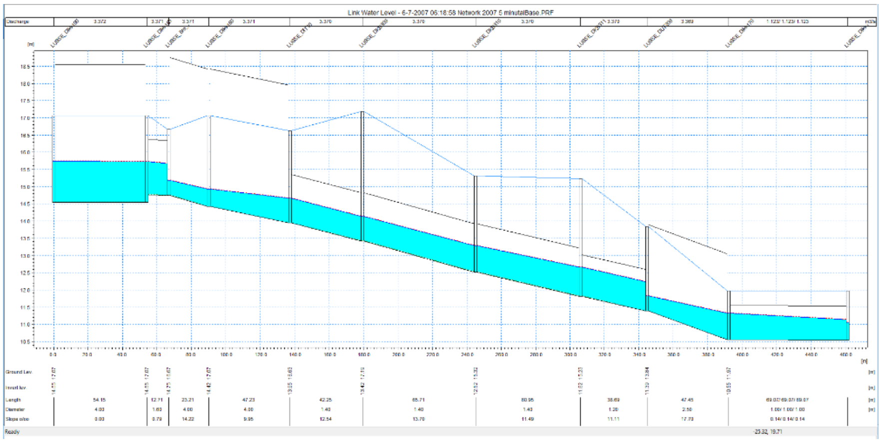


Figure 5-13. Water profile of the Ramlösa culvert for the 5/07/2007 in the 2019 scenario

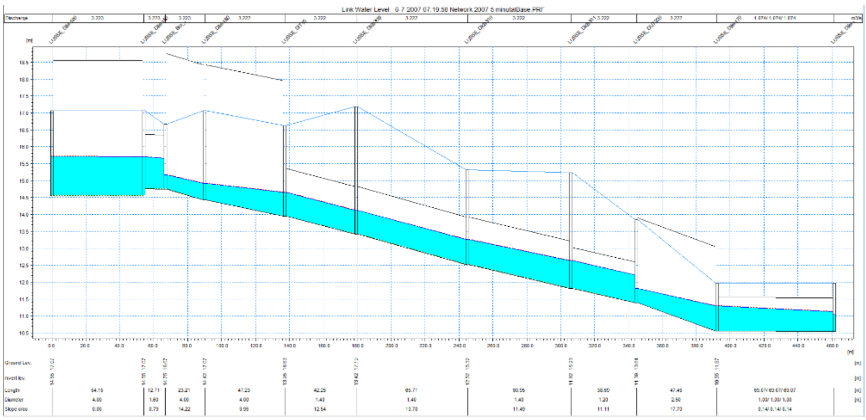


Figure 5-14. Water profile of the Ramlösa culvert for the 5/07/2007 in the proposed scenario

5.3.1 Simulated Scenarios

Comparing the three scenarios a great improvement in the decreasing of the peak flows can be seen. At the same time, this translates in lower water levels and in a reduced risk of flooding. The main reduction of the flow is in the Påarp branch due to the addition of the regulation pond. This has the biggest impact in the output flow of the pond into the stream.

The biggest reduction, as it was expected, was reached with the addition of the ponds in scenario 2. Additional improvements are produced with the modification of the output infrastructure of the Påarp pond, but not as pronounced as with the adding of storing capacity.

There are specifics tools in the software MIKE URBAN to compute the overflow but because of working with an open channel model for the vast majority of the channels are open and the manholes are fictitious, so this tool is not the optimal in this case for the simulation of the upper section of the catchment, which is a mostly rural area.

It is also worth to mention that the absence of regulation in one of the Långeberga branch sections, which covers a big drainage area produces a big impact in the downstream water levels.

The water levels in the Köpingegården pond for extreme levels have been reduced below the overflow level provided by the NSVA. This improvement would reduce the damage to the nearby fields of the area.

The most significant reduction is produced with the change from scenario 1 to scenario 2. Scenario 3 does not produce a significant change in the water levels of the pond. This is related to the fact that from the scenario 1 to the scenario 2 the increase of retention volume is bigger because 3 ponds are addded compared to the modification of one of the ponds in the scenario 3.

The water levels around the critical part of the catchment, in the Ramlösa culvert where the highest slope of the network is located, there is a significant reduction of the water depth.

In the 2008 scenario with the rainfall episode of 5/7/2007 there is one critical point where the water level reaches the top of the pipe. The addition of the regulation ponds on the scenario 2 and scenario 3 minimizes the problem of the limited capacity of the culverted section that could lead to the excess of water running over the Ramlösa gardens and the historical buildings.

5.4 Discussion

This chapter includes the discussion about the model, its validation and the results obtained for the different simulated scenarios.

5.4.1 Model Setup and validation

Geometric data: the availability of detailed cartography was not available for all the situations, so some approximations were made to simplify the model and be able to add the ponds to the model. All the ponds were assumed to be rectangular and the depth and length of each one was adapted to match the total volume of the pond.

Rainfall data: one of the limitations found in the model was the inability to handle extreme rainfall episodes. This was due to the fact that MIKE URBAN cannot handle water levels that surpass the channel level. This limitation does not happen when simulating pipe systems since the water cannot go out of the system at any other point than at the manholes. This limitation could be overridden if a 2D elevation model was obtained and the channel cross-section profiles were extended by the sides to allow a flood plain to be developed.

Catchment connections: the model takes only one point of connection between each catchment and the network. This simplification does not describe the reality precisely because in real life the water enters the channels along the whole length of them. For catchments which size is small this is acceptable, but for some of the catchments that were simulated that implied a big inflow of water at a single point that would not represent the reality accurately.

Validation: the model validation was done in the 2008 scenario, but the current scenario was not possible to calibrate because there is no available flow data for the latest stage of the network.

6 Conclusions and recommendations

6.1 Conclusions

In this project, the performance of big scale sustainable drainage systems has been studied for the Lussebäcken catchment in the city of Helsingborg. A MIKE URBAN model has been used to study the effects of rainfall on extreme events and study the peak discharges in the water stream.

The aim of this project was to study de viability of using ponds constructed with environmental goals in mind to reduce peak flow to prevent flooding. The main aims of the project have been accomplished regarding the improvement of the already existing ponds to further improve their function as storm detention ponds. Between 4% and 6% reduction of peak flow for the simulated episodes has been obtained from the current situation. This result is considered as satisfying because the proposed modification for the increment of storing volume is relatively simple in terms of impact and its cost is lower than the one of building a new pond.

The main improvements are produced when adding storing capacity to the network rather than regulating the existent ponds. Although both improvements should be considered as equally important to maximize the efficiency of sustainable drainage measures. The limitations of the places where it was possible to modify the structures without having an impact on the environment have been restrictive.

7.2 Recommendations

Future work could go in the direction of obtaining real data for the discharge to be able to calibrate the modified model.

Also, the change to a model capable of handling bigger rain episodes should be considered. One of the solutions could be to add fictitious walls to the open channels, although this solution would imply distorting the reality because the water would overflow the nearby soil and not raising its water level.

Regarding the fact that a big part of the Långeberga branch is not regulated, it would be interesting to study the possibility of adding sustainable drainage infrastructures to regulate the flow to discover if they would have a big impact downstream.

Further investigations could be to investigate the viability of an active control system with an active weir to regulate the outflow of the ponds depending on the water level.

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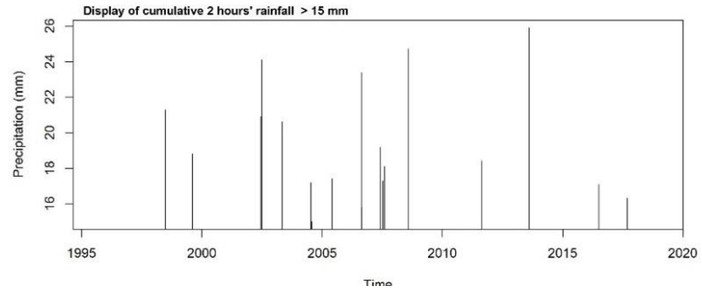
Appendix

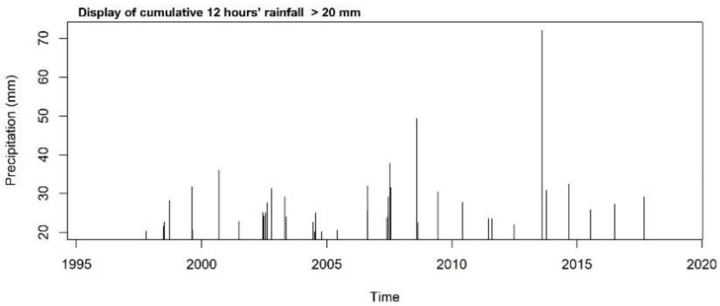
Appendix A

Figure A.1: Cumulative rainfall for different period of time between 1995 and 2018

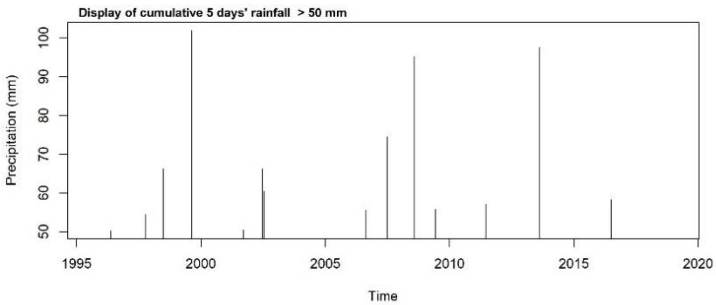
I.



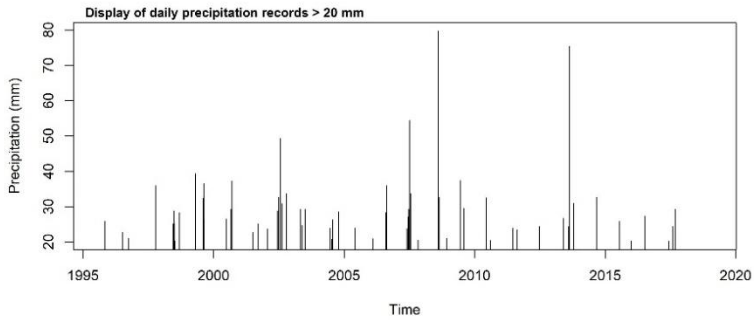
II.



III.

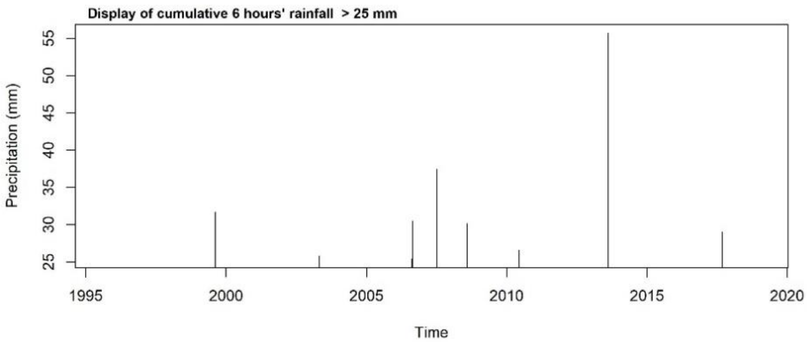


IV.



Display of daily precipitation records > 20 mm
Precipitation (mm)
80
70
60
50
40
30
20
1995
2000
2005
2010
2015
2020
Time

V.



Display of cumulative 6 hours' rainfall > 25 mm
Precipitation (mm)
55
50
45
40
35
30
25
1995
2000
2005
2010
2015
2020
Time

Appendix B

Figure B.1: Runoff coefficients used within the rational method for different land uses

Description	Runoff Coefficient
Business	
Downtown Areas	0.70–0.95
Neighborhood Areas	0.50–0.70
Residential	
Single-family	0.30–0.50
Multi-family detached	0.40–0.60
Multi-family attached	0.60–0.75
Residential suburban	0.25–0.40
Apartments	0.50–0.70
Parks, cemetaries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yards	0.20–0.40
Unimproved areas	0.10–0.30
Drives and walks	0.75–0.85
Roofs	0.75–0.95
Streets	
Asphalt	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Lawns; sandy soils	
Flat, 2% slopes	0.05–0.10
Average, 2%–7% slopes	0.10–0.15
Steep, 7% slopes	0.15–0.20
Lawns; heavy soils	
Flat, 2% slopes	0.13–0.17
Average, 2%–7% slopes	0.18–0.22
Steep, 7% slopes	0.25–0.35

Appendix C

In order to design structures for stormwater management such as sustainable drainage system components, it is necessary to construct Intensity-Duration-Frequency (IDF) curve and determine return period of rainfall that used for rainfall frequency analysis, selecting the maximum design discharge and estimating maximum flood events. IDF curve typically used during design of water resources hydraulic and hydrologic structures, it is constructed through the relationship of between rainfall intensity, rainfall duration, and return period. IDF curve shows the change of rainfall intensity against rainfall duration and it is generated through the rainfall observation data record for a specific location. In this study, Intensity-Duration-Frequency (IDF) was constructed and return period of rainfall intensity was also determined. Return period of rainfall also known as recurrence interval is used to describes the amount of time of rainfall can be expected to happen again between of a certain rainfall event. The procedure of calculating and statistical frequency analysis of rainfall return period of can be done through different methods.

Several studies for instance (Al-anazi and El-Sebaie, 2013, Ewea, Elfeki et al., 2017, Yue, Ouarda et al., 1999) have been highlighted the Gumbel theory of distribution as the one of method frequently used to determine return period and IDF curves for flood frequency analysis. Based on this, the Determination of Return Period and construct IDF Curve of Lussebäcken catchment were done and the observation hourly rainfall intensity data collected on Helsingborg station A which is located in vicinity of the catchment were used The determination of return period and constructing IDF curve were performed in the following summarised steps:

(1) After collecting observation rainfall depth, the maximum annual hourly rainfall was selected, and it was ranked in descending order. According to Al-anazi, K.K. and El-Sebaie, I.(2013), the exceding probability with each maximum rainfall was computed.

$$P(X \geq d) = \frac{1}{T_r} = \frac{m}{n+1} \quad (1)$$

where, $P(X \geq d)$ is plot with the exceedance probability of rainfall depth each year, m is the number of the rank, n is the number of the years, T_r recurrence interval and x is the rainfall depth. Thus, From the above equation recurrence interval were determined,

$$T_r = \frac{1}{P(X \geq x)} = \frac{n+1}{m} \quad (2)$$

(2) By plotting annual maximum hourly rainfall depth with recurrence interval, the Gumbel Extreme Value (Type I) is fitted. Then, The Gumbel extreme value cumulative distribution was computed as:

$$P(X \leq x) = e^{-e^{-\alpha(x-\beta)}} \quad (3)$$

where, $P(X \leq x)$is the probability of non-exceedance, e is the Euler's number (i.e. the Napier's constant), α and β are distribution parameters. With regarding to (Kite, 1978) , the value of α and β parameter were estimated as:

$$\alpha = \frac{1.2825}{S} \text{ and } \beta = \bar{x} - 0.5772\alpha \quad (4)$$

where, S is the standard deviation and $\bar{x}$ is the mean of maximum hourly rainfall data. Thus, the mean and standard deviation were obtained from the following equation:

$$\bar{x} = \frac{\sum x_i}{n} \text{ and } s = \left[\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}\right]^{0.5} \quad (5)$$

(3) The random value was then calculated with relation of return period, it is given as:

$$K_T = \bar{x} + K_T S \quad (6)$$

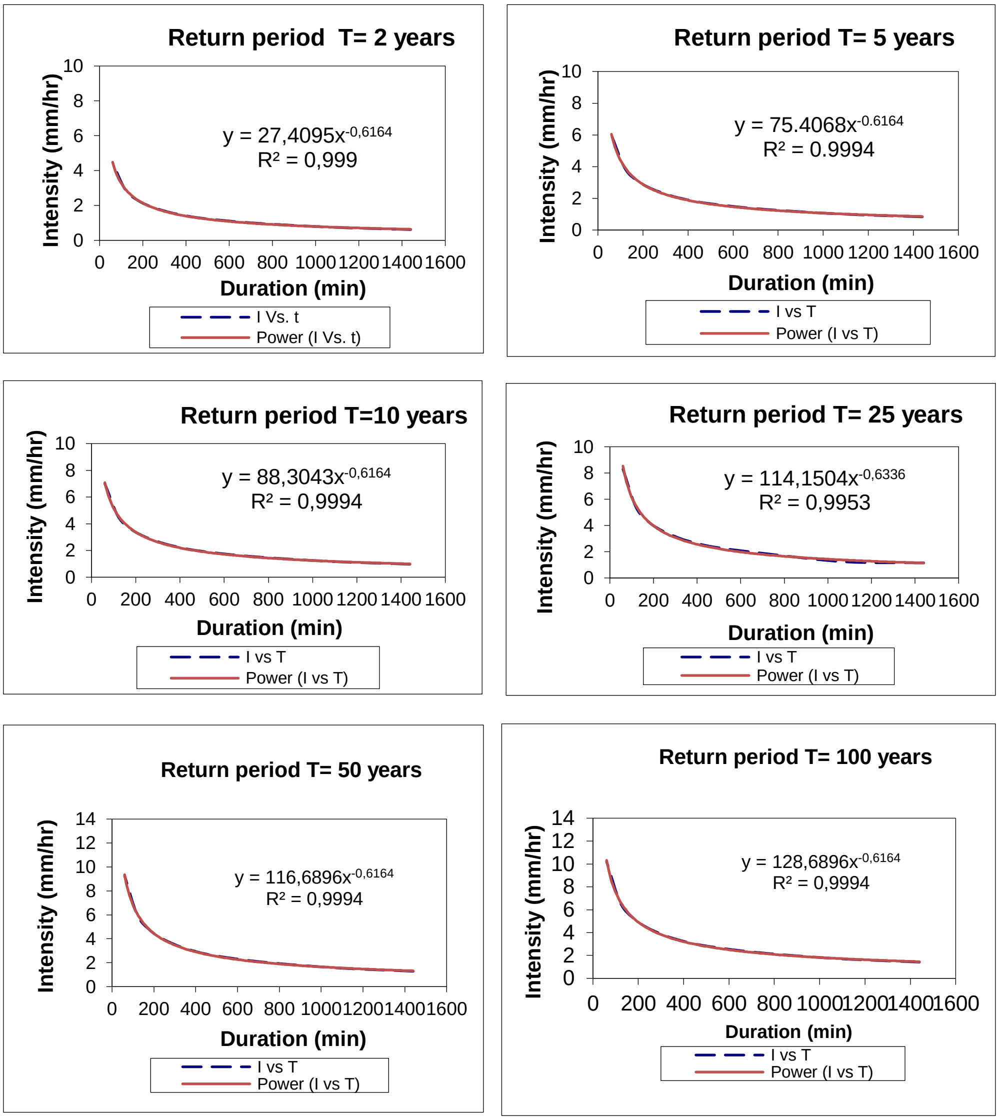
where K_T is the frequency factor related with return period and it is estimated as:

$$K_T = -\frac{\sqrt{6}}{\pi}\left\{0.5772 + ln\left[ln\left(\frac{T}{T-1}\right)\right]\right\} \quad (7)$$

(4) Based on the equation(4.4.8) the rainfall intensity(I_T) for each duration (i.e. 60, 120, 180, 240, 300, 360, 480, 720, 1080, 1440mim) for each selected return period(i.e. 2, 5, 10, 25, 50, 100 years) was determined and it was computed as:

$$I_T = \frac{P_{(t)}}{T_d} \quad (8)$$

where, $P_{(t)}$ is rainfall depth(mm) and T_d is duration in hrs. From this equation (8), the computed rainfall intensity was plotted against duration for return periods, Figure



0-1.*Figure 0-1:Relationship between intensity and return period with the calculated power for Helsinborg*

(5) The last step was to construct Lussbäcken IDF curve, Gumbel empirical formula was used. It is expressed as:

$$I_T = \frac{K * T^m}{T_d^n} \tag{9}$$

Where, I_T Intensity (mm/hr), T_d is Rainfall duration (min), T is Return period (Years) and K, m, n are adjusted parameters. The computed rainfall intensity was potted against duration and the following IDF curves was produced.

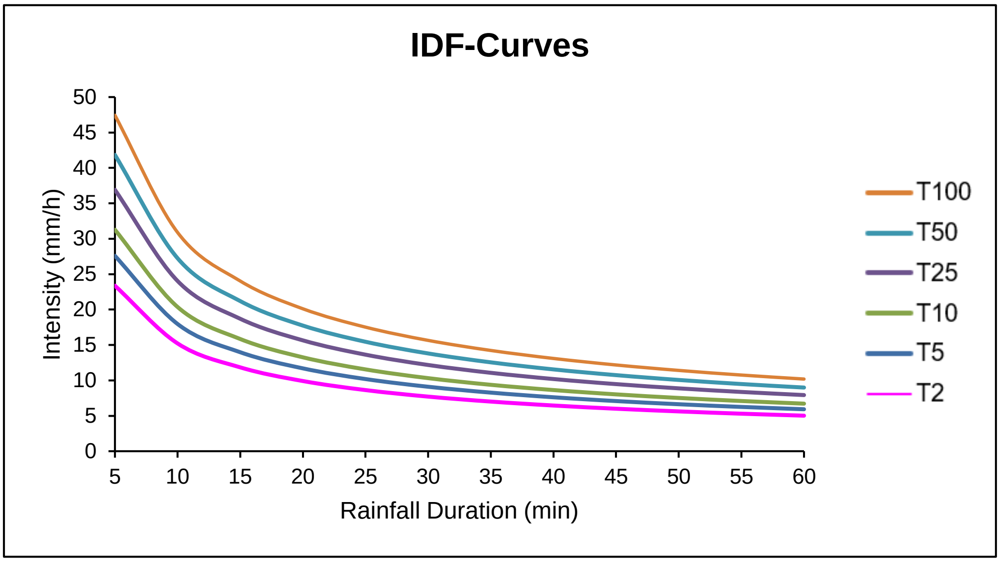


Figure 0-2: Intensity-Duration-Curves for Lussebäcken: Variation of intensity against rainfall duration for each return period.

Appendix D

Sustainable Drainage Systems Assessment and Optimisation
A case study for Lussebäcken Catchment, Helsingborg

Drastic increase of population, urbanization, climate variability and extreme precipitation are the main challenges on urban storm water management worldwide. These put many regions over the world to be more vulnerable for flooding. Due to human activities within the watersheds that have resulted in changing of river morphology and urban hydrological cycle, are still dominating. As the results, the urban runoff hydrograph is affected and result in increasing of the urban runoff peak flow. This is a problematic for the existing urban drainage pipes. To handle the addition flow would be a tough problem to the decision makers and planners. The storm water control measures techniques such as the sustainable drainage system has been adopted around the world cities.

Sweden has the one of the Europe countries adopted these techniques. Based on the problem of flooding and increase of urbanization in Helsingborg city, the sustainable drainage system techniques have been also implemented and adopted by municipality planners in Helsingborg to tackle the impacts of runoff flooding on infrastructures and property of the people. The constructed ponds and wetlands have been prioritized, even these measures are implemented there, the performance of those constructed ponds and wetland has still remain the issues to maximize its benefits. This thesis studies the use and performance of combined use structures in urban drainage systems in the Lussebäcken catchment, Helsingborg to reduce the happening of flooding while serving an environmental task. The main control structures are artificial constructed ponds and wetlands in which water is stored, regulating the flow of the water stream and preventing a fast raise in the water level. This ponds also serve an environmental task because fauna and flora can develop in them, giving added natural value. A MIKE URBAN model was used to simulate the rainfall-runoff process. First of all, the model was validated with real data.

This model was modified because ponds and wetlands have been added since the setup of the original model. Afterward it was further modified with the proposed measures to improve and optimize the performance of the already built measures. Three different scenarios were performed to simulate the process. First scenario is a simulation of the model without addition measures, second scenario is a simulation of the model with addition ponds and Third scenario is Proposed optimization of the ponds. The results were displayed and the change of peak flow with added measures were identified. A reduction in the peak flows in the water stream for big rainfall episodes is accomplished.

The decrease of peak flow in the main stream is close to the 20% for the added ponds and a further 8% is gained after optimizing one of the ponds. According to this result, the reduction of peak flow is achieved, and the base flow is also increased in the ponds by storing water long time which is increased the infiltration the ponds. Even this were achieved, the further study is recommended for simulating runoff with connection of ground water flow. Mike she, or Mike 21 software's and other similar software's is recommended in order to simulated 2-Dimensional flow is recommend