CHALMERS

Assessment of flood mitigation measures

Further development of a proactive methodology applied in a suburban area in Gothenburg

Master of Science Thesis in the Master's Programme Geo and Water Engineering

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Department of Civil and Environmental Engineering
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CHALMERS UNIVERSITY OF TECHNOLOGY
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Cover:
Simulated flood from a rain event with 100 year return period with a flood mitigation measure (alternative 1 in the report) implemented into the model.

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ABSTRACT

Flooding due to extreme rain events in urban environments is a problem and a growing concern. When the stormwater systems design return period are greatly exceeded during extreme rain events, flooding is inevitable. However, the flood consequences can be mitigated by surface water management. A Swedish methodology, Plan B, is developed for investigation and planning of extreme rain events by means of flood simulation models. This thesis had its starting point in the Plan B methodology which was further developed through a case study by testing two assessment methods that provide information on the efficiency of surface water measures. The two methods were a key figure assessment and a more comprehensive cost-benefit analysis assessment.

It was found that the key figure assessment was easiest to use and that it would be a useful tool for decision makers, but also that it still need some improvements for being reliable. The cost-benefit analysis assessment demands much resource to set up and is tedious to manage and was consequently not recommended for smaller surface water investigations.

The case study also contributes with experience on relatively new methodologies and therefore also gives other insights. The most important was that a surface water planning strategy, overarching different departments within the municipalities, is required to mitigate flood consequences.

Key words: Urban flooding, surface water management, MIKE URBAN, 1D-2D simulation, flood mitigation measures, cost-benefit analysis

Bedömning av översvämningslindrande åtgärder
Vidareutveckling av en proaktiv metodik tillämpad på en förort i Göteborg

Examensarbete inom Geo and Water Engineering
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SAMMANFATTNING

Översvämningar till följd av extrema regnhändelser i urbana miljöer är ett problem och ett växande orosmoment. När återkomsttiden som dagvattensystemen dimensionerats för kraftigt överskrids är en översvämning oundviklig. Dock är det möjligt att med ytvattenplanering lindra översvämningskonsekvenserna. En svensk metodik, Plan B, är utvecklad för att undersöka och planera för extrema regnhändelser med hjälp av modeller för översvämningssimulering. Denna rapport hade sin utgångspunkt i Plan B metodiken som vidareutvecklades i en fallstudie genom att två bedömningsmetoder som ger information om effektiviteten hos ytvattenåtgärder testades. De två bedömningsmetoderna var en baserad på nyckeltal och en mer heltäckande bedömning baserad på en kostnads-nytto-analys.

Det konstaterades att nyckeltalsbedömningen var enklast att använda och att denna skulle vara ett användbart verktyg för beslutsfattare, men också att metoden fortfarande kräver utveckling för att vara tillförlitlig. Bedömningen baserad på kostnads-nytto-analys kräver mer resurser för att etablera och är omständig att hantera och följaktligen rekommenderades inte denna metod för mindre ytvattenutredningar.

Fallstudien ger också en ökad erfarenhet till relativt nya metodiker och därmed också andra lärdomar. Den viktigaste var att en planeringsstrategi för ytvatten som spänner över olika avdelningar inom kommuner är nödvändig för att lindra översvämnings-konsekvenser.

Nyckelord: Urban översvämning, ytvattenhantering, MIKE URBAN, 1D-2D simulering, översvämningslindrande åtgärder, kostnads–nytta analys

Contents

Preface

This thesis was the last part in my studies at the Master of Science programme Geo and Water Engineering at Chalmers University and Technology. It was performed during February to June 2012 in cooperation with Gothenburg Water and DHI.

Gothenburg city has through the years been subjected to flooding at several occasions due to heavy rain events. One of the recent was a rainstorm that struck in August 2011 and led to flooding of many basements and roadways. The municipal water company, Gothenburg Water, have through the years made many investigations regarding flooding and now wished to investigate the use of a proactive flood planning methodology. At the same time the consultant firm DHI had developed a flood planning methodology named Plan B and wished to further develop this.

First I want to thank my supervisors Annika Malm and David Jacobsson, Gothenburg Water, and Håkan Strandner, DHI, for their support and guidance during this semester. Also want to thank Olle Ljunggren who with great experience and local knowledge has contributed with much useful information regarding the case area. Special thanks to Christian Brinkeberg for the help with the digital elevation models and to my opponents Tania Sande Beiro and Ángela Serrano Manso for feedback and exchanged experiences.

Last I want to thank all the people working at the Gothenburg Waters office in Hjällbo for many fun and interesting talks over lunches and coffee breaks.

Gothenburg July 2012,

Jonatan Larsson

List of abbreviations

1D	One Dimensional
2D	Two Dimensional
AAD	Annual Average Damage
CBA	Cost-Benefit Analysis
CDS	Chicago Design Storm
CSIR	The Council for Scientific and Industrial Research
DCC	Dublin City Council
DEFRA	Department for Environment, Food and Rural Affairs (UK government department)
DEM	Digital Elevation Model
DHI	Danish Hydraulic Institute (consultant firm)
FRC	FloodResilienCity
GIS	Geographical Information System
IPCC	Intergovernmental Panel on Climate Change
MOUSE	Model Of Urban SEwers (software)
NBCR	Net Benefit Cost Ratio
SEK	Swedish Krona
SMHI	Sveriges Meteorologiska och Hydrologiska Institut (Swedish meteorological and hydrological institute)
SUDS	Sustainable Urban Drainage Systems
SWMP	Surface Water Management Plans
SvD	Svenska Dagbladet (Swedish daily newspaper)
UK	United Kingdom

1 Introduction

Extreme rain events have always been a problem and a challenge in the urban society. When they occur in urban areas the consequences can be striking with severe flooding and damage to properties and infrastructure. In June 2011 a heavy rain event stroke Copenhagen and in two hours as much precipitation fall as normally falls during three summer months. Several big approach roads become blocked, the subway was closed for several hours and the train traffic did not function as normal for days (SvD 2011). Afterwards the insurance costs have been calculated to about six billions SEK and thereto comes the costs for the society and the individuals (for instance insecurity and loss of affective values) (Moberg 2012).

The last decade the climate change has been an increasing issue. Most studies agree on the climate change being a reality, but the opinions about the severity and the prognosis on the future climate differs. Some studies states that extreme events like heavy precipitation will occur more frequently in the future (IPCC 2007, Christensen & Christensen, 2002) whereas other studies states that measured precipitation series does not indicates an increased frequency of heavy rain events (Hernebring, 2006). A British study agrees on extreme rain events occurring more frequently, but also makes an important statement that the vulnerability for extreme rain event exists already today (DEFRA 2012). Events like those in Copenhagen testify on this vulnerability. Consequently, already today, there exists a need of preparation actions against floods.

To design stormwater pipe systems for extreme rain events are in most cases unrealistic due to economical and practical reasons. To increase the capacity in existing network in developed areas to cope with extreme rain events are in most cases impossible. There exists examples on large underground sewer constructions that can handle very heavy events but there will however always be a possibility for a more severe scenario than the scenario design for, meaning one can never protect against all rain events.

Urban storm water systems in Sweden are normally designed so that the pressure level in the pipe network not will exceed the ground or basement level for a rain event with a return period of ten years (Svenskt Vatten 2004). For rain events with a return period that greatly exceeds the design return period, whatever it might be, the focus needs to be shifted into mitigation of flood consequences. This could be done by a proactive approach where the surface water flow patterns and storage areas are known and controlled, if possible. This has in a Swedish study been addressed as “Plan B”, and in the same study a methodology for flood control investigations are described (Ahlman, 2011).

To design and assess flood mitigation measures demands knowledge on how an area would react to a heavy rain event under current situation and under changed circumstances. It is not an obvious task to obtain this knowledge but there are tools that can simplify this process and that can provide estimates on flood scenarios under different conditions. Still, with the flood extents estimated, there is still a challenge in assess and compare different flood mitigation measures.

1.1 Purpose

The purpose with this thesis was to evaluate and develop available flood control planning methodologies, with starting point in the Plan B methodology and with main focus on how to assess and rank surface water control measures. The overall long term purpose was to provide aid for preventing and mitigate future flooding due to extreme weather.

1.2 Question formulation

The thesis originated in four questions where the first one was the main question.

- How should flood mitigation measures be assessed and what tools should be used?
- What flood mitigation measures should be investigated and evaluated?
- What resources in form of personnel time and input data are needed for the surface water management study?
- Would it be feasible to investigate larger areas?

1.3 Method

The chosen method for evaluating and further develop the available surface water management methodologies was to perform a case study. The case study had its starting point in the Plan B methodology but experience from other studies and methodologies were also used.

The case study was performed as a multi-loop process, see conceptual model in Figure 1-1. Since the thesis had its starting point in the Plan B methodology, which includes a computer model part, the top loop describes a classic model creation process where data about the system was gathered, simplifications were made and a computer model was created and calibrated. This model could then describe flood scenarios with precipitation data as input. The next step was to test four measures by implement them into the model and run new simulations.

From gathered experiences from other studies, two assessment methods were chosen to be tested in the case study. The first was based on key figures and the second was based on cost-benefit analysis.

The methodology evaluation was made by back loops where changes were made in work procedures and flood results assessment parameters before the work flow was repeated. By this back loops the methodology could be reviewed.

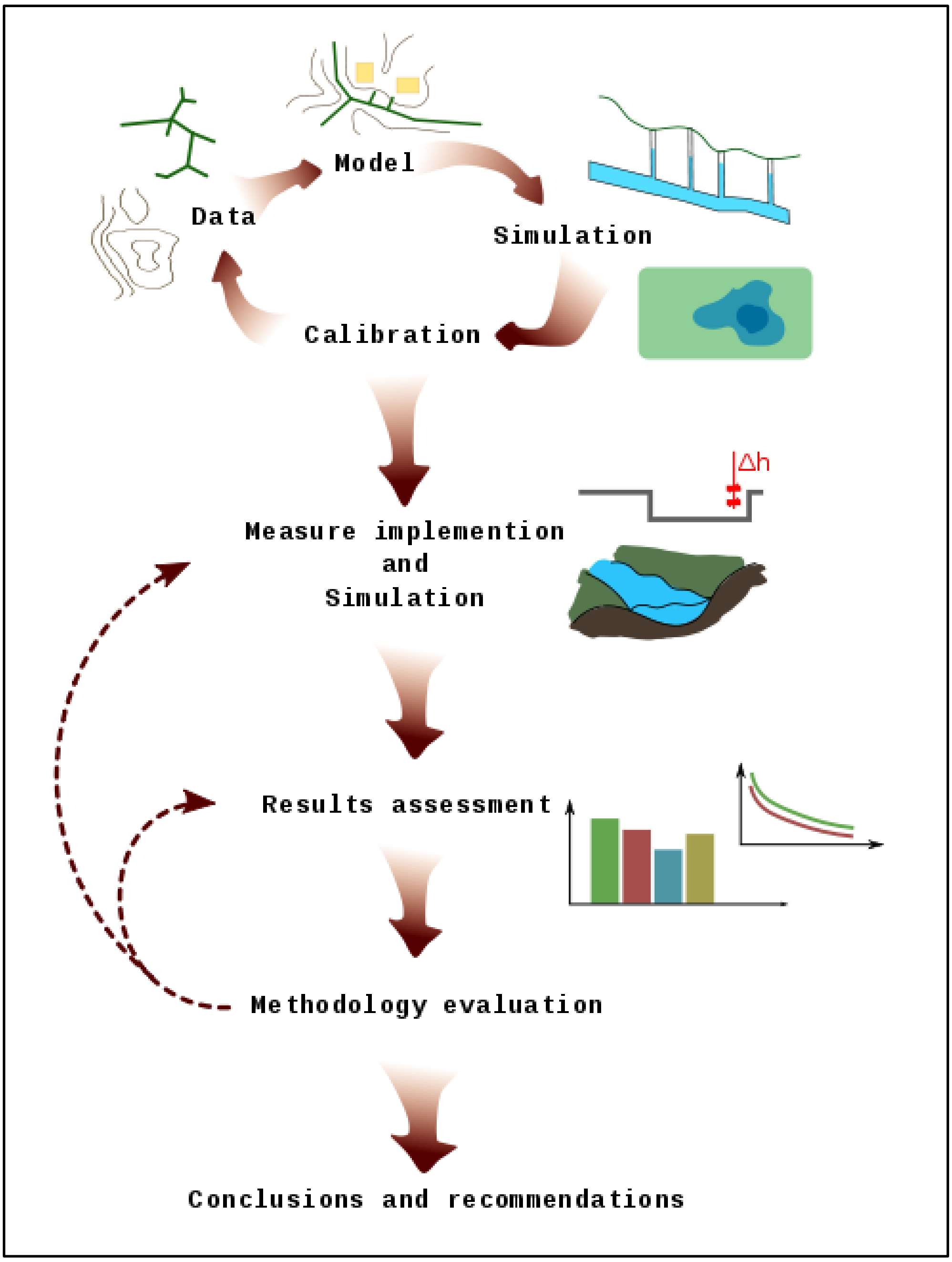

Figure 1-1. Conceptual model of the method used in this thesis for development and evaluation of the surface water planning methodology by case study.

1.4 Delimitations

The thesis was only considering pluvial (rain-related) floods. The focus was mainly on the implementation and assessment of flood mitigation measures in form of surface water control and only briefly handled general flood risk management.

The case study was limited to a rather small area in the south part of Gothenburg as the intention was to limit the thesis geographically rather than limit the depth of the study. Only stormwater and stormwater systems was analyzed through the whole thesis, meaning flooding by to the foul water system was not investigated.

1.5 Thesis outline

Chapter 2 gives an introduction to flood management and surface water control. It describes the Plan B methodology and also other methodologies on how to cope with flood risk. It also describes different flood mitigation measures and gives some examples on the same subject.

Chapter 3 describes in short the concept of the urban simulation tools. This chapter does not intend to in detail describe the theory behind the calculations performed within the simulations, but describes how the models work in general terms.

Chapter 4 describes the two assessment methods that has been used and tested through this thesis.

Chapter 5 describes the case study area and how a model was set up and calibrated for this area. The chapter also describes the four measures that were implemented into the model for mitigating the flood consequences. It also describes a fifth alternative that was tested as a mind experiment.

Chapter 6 shows the results from the simulations and from the assessment methods.

Chapter 7 evaluates and discusses the used methods and the whole case study. This is divided into five sections with evaluation and discussion on the methodology, the assessment methods, the measures, the mind experiment and flood management.

Chapter 8 ends the report with some conclusions, and thereafter follows references and appendices.

2 Flood management and control

Already 2000-3000 BC the ancient civilization of Mesopotamia had a system consisting of banks, channels and regulation constructions to ensure safe settlements and good working agriculture (Häggström, 1999). Thus, flood control is not a new engineering field but has off course evolved over history in both terms of technical solutions as well as in planning and operation.

Flood management is today a large field involving different competencies and many subjects, all with their own difficulties and challenges. The two greatest are how to identify and understand flood risk, and how to handle and reduce that risk by implementation of measures. An integrated approach that combines the whole field is preferred and recommended (DEFRA 2010, Jha et al 2012).

The following subchapters give a short introduction to flood management. Section 2.2 describes the Swedish study Plan B which was the main background and input for this thesis, and the methodology development had its starting point in that study.

2.1 About flooding

Flooding is a broad term which means that an area is under water. For some areas, floods can be a natural part of the hydrological cycle while for others, typically urban areas, floods are unwanted and can cause severe problems. The causes of a flood vary and can by the origin be categorized. In *fluvial flooding* water level rises in rivers due to volume rich rain events in the sub-catchments upstream and the water level overtop the riverbanks and starting to spread on land. In *coastal flooding* sea level rises in over land, often due to a combination of storm surges, wave action and tide cycles. *Pluvial flooding* is associated with heavy rain events and occurs when precipitation flows on the surface and ends up in temporary ponds in depressions, due to insufficient conveyance capacity compared to rainfall intensity. (Houston, 2011)

It is common to describe rain events by its frequency and consequently its return period. For example, a rain event with a certain precipitation intensity and volume that statistically occurs ten times within a century has an average return period of ten years and is thereby often called “ten year rain” (Butler & Davies, 2004). There are different statistical methods that cope with rain events return periods, often based on processed long precipitation measurements, and countries generally uses different methods that suits their climate. The above terminology is also used in flood management where a “100 year situation” is a flood extent that is expected once every 100 years. For pluvial flooding this could be corresponding to a rain event with a return period of 100 years, but for fluvial flooding, where more factors are influence the flood extent, this direct reading could not be done. For an example, an 80 year rain within a catchment that otherwise is relatively dry, may result in an only 20 year river flow downstream.

2.2 The Plan B methodology

Together with four Swedish municipalities and with partial funding from The Swedish Water & Wastewater Association, the consultant firm DHI has performed a study to develop a methodology for analyzing urban flooding. The purpose of the study was to describe a methodology that shifted the focus beyond the stormwater systems design case and perform preparations for those events when the capacity is reached and flooding is inevitable. By doing so, the flooding consequences can be minimized. This could be done by, if possible, controlling water flow patterns and storage areas by creating alternative flow paths or creating areas that can be allowed to flood and store water. These proactive preparations were addressed as Plan B. (Ahlman, 2011)

The Plan B methodology consists of four parts: analysis, description of consequences, measures and description of consequences for different measures. The first two parts treats current conditions and the second two parts treats changed conditions and what impact changes would have on flood scenarios. The analysis is made with an integrated 1D-2D model capable of modelling stormwater flow both on the surface (2D; calculations in two dimensions) and in the pipe network (1D; calculations in one dimension) and the interaction between these. The consequences are analyzed with help of geographical information system (GIS) tools and are assessed with key figures. (Ahlman, 2011)

The study report concludes that accurate ground elevation data is a key for good results, and to use laser scanned elevation models is recommended (interpolation from contour lines was partly used in the study). From the results of four case studies the report further concludes that strategically chosen measures can decrease the flood damages and that in dense exploited areas it is important to quickly transport the water away from the area since the limited possibilities for detention. The case studies also shows that confined areas are difficult to manage and find measures for. (Ahlman, 2011)

2.3 Other surface water management methodologies

The Department for environment, food and rural affairs (DEFRA), UK, has established a guideline for surface water management plans (SWMP). This contains an overarching framework to promote organizations and stakeholders to work together with solutions for surface water flooding problems in long-term action plans. The methodology is structured into four phases: preparations, risk assessment, options and implementation and review. In the preparations phase the requirements of a flood study is identified as wells as partners and stakeholders. The risk assessment aims at identify vulnerable areas and the identification process is divided into different scales. First risk assessments are done with, for instance, data on historic flood incident information and topographical screening, intermediate risk assessments include using models and when necessary detailed models is recommended to predict surface water flooding. The methodology does not state a first choice of modelling approach but lists different possibilities, among others 2D overland flow model and 2D overland flow model coupled with 1D network model. (DEFRA, 2010)

A Swedish development project from the consultant firm Tyréns AB has suggested a flood planning methodology for municipalities. It is based on the new national digital elevation model from the Swedish mapping, cadastral and land registration authority (Lantmäteriet) and the analysis is made in two separate working environments. The first analysis is made with GIS software that mainly analyzes areas topography, with the possibility to handle large areas and also to compare results with society functions in a convenient manner. The second analysis is made with a hydraulic model including both the network and the surface flow, a 1D-2D model for the example in the study. The intention is to use the first analysis option for identification of risk areas which then can be further analyzed in more detail with the hydraulic models. (Tyréns, 2010)

In an EU-funded project called FloodResilienCity (FRC), which is a cooperation project about flood management and urban planning, an approach to flood risk management is used which is called "4 A's": Awareness, Avoidance, Alleviation and Assistance. The objective in the alleviation step is to reduce flood risk, and one of the suggested main actions is to construct streets to be used as rivers during flood. This could be done by construct channels below the street and use permeable road surface, or canalization of the street by raised curbstones. (FRC, 2012)

Another methodology for investigate and prevent flooding is the Blue Spot concept developed by a transnational research programme including road owners in several countries. This is intended to be used by road owners and operators for large and important roads in non-urban areas. The concept consist of three levels where the first is a screening step where all depressions are identified by having a rain fall on a surface model and not allowing any infiltration or evaporation. Hence, all precipitation will be collected in depression, blue spots, and if these volumes are large and close to roads they are considered as threats. In the second level a rain sensitivity analysis is performed for the depressions. Before further analyses the number of blue spots is often decreased by risk analysis where the threats of the spots are evaluated. The third level consists of more detailed hydrodynamic models which includes both surface flows and flows through drainage system, which gives a more accurate calculation of the flood risk. There are examples of both 1D-1D models being used as well as 1D-2D. In the third level there is also a possibility to examine solutions to the flooding problems. (Hansson et al, 2010)

2.4 Surface water management measures

Flood risk can be much reduced by implementation of measures. These can be divided into two main categories: structural and non-structural measures. Structural measures reduce flood risk by constructed objects or modifications that control the surface water flow while non-structural measures reduce flood risk by keeping people safe through better planning and urban development. Examples on structural measures are embankments, barriers, conveyance of surface water and flood storage while examples on non-structural measures are emergency planning, awareness campaigns, flood warnings systems and land planning. (Jha et al, 2012)

Surface water planning plays a crucial role especially during intensive rain events in urban areas and can greatly reduce the risk from pluvial flooding. For undeveloped areas there are naturally better possibilities to introduce risk reducing design by safe flow route planning and sustainable urban drainage systems (SUDS). For developed areas there is often a greater challenge to implement solutions and reduce the risk because of, for instance, lack of space and difficulties to change existing constructions. But also small and inexpensive changes in the landscape may create relatively safe flow routes and alleviate flood consequences. (Houston et al, 2011)

When identifying potential surface water management measures the source-pathway-receptor approach can be useful, see Figure 2-1. At the source from where surface water flow starts, mitigation measures aims at reduce rate and volume of the surface water runoff. Possible measures are infiltration in storage, swales, permeable paving, green roofs, basins, ponds and wetlands. Pathways measures aim at mainly manage overland flows and its passage through the urban environment, but also underground flows. Overland flows can be controlled by flood route creation, using road networks, changed height of curbs and also by using car parks, recreation areas and parklands as flood storage areas. At the receptor the flood consequences can be mitigated by flood warning systems, improved resistance and resilience and preparedness with for instance temporary flood defense equipment. (DEFRA, 2010)

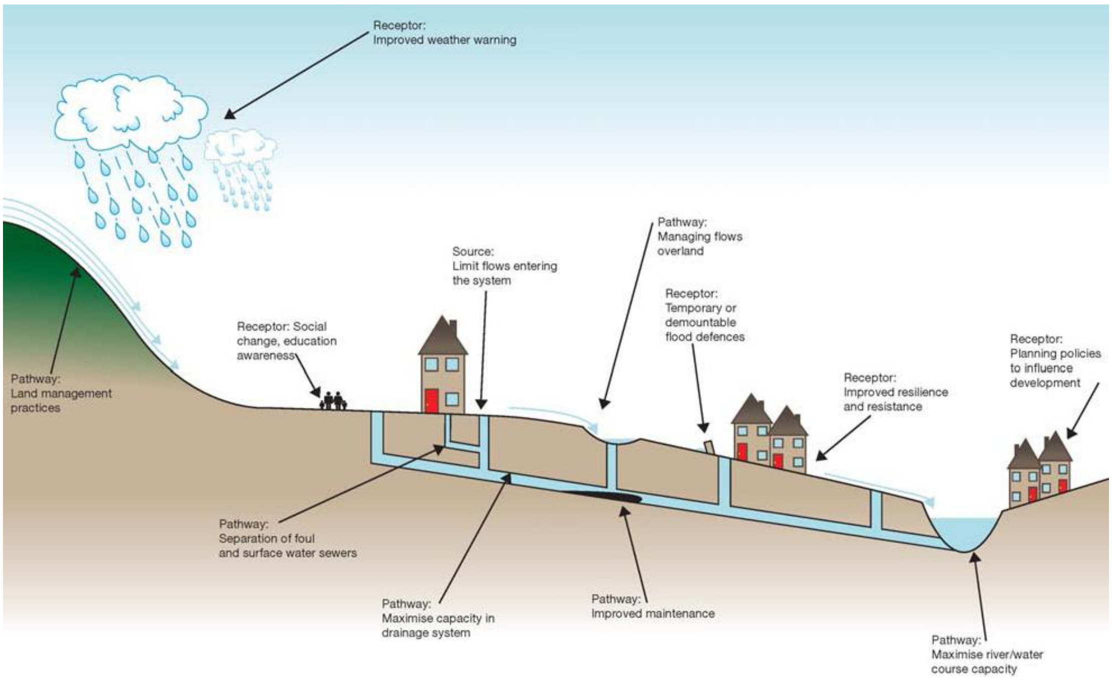

Figure 2-1. Mitigation measures that can be considered to reduce surface water flood risk (DEFRA, 2010).

SUDS are often mentioned in stormwater contexts and the term includes much more than just surface water planning. Traditionally the concept is much about stormwater treatment and constraining stormwater flow to wastewater treatment plants. The general idea is to create stormwater facilities that mimic nature’s way of treat water with solutions like detention ponds, swales, green roofs and infiltration trenches. The purpose with these facilities is also as much to reduce the speed, peaks and total amount of runoff, hence these facilities also have an alleviation effect on the flood risk (Butler & Davies, 2004).

There are many examples of wet ponds (pond with constant water mirror) which work as detention facilities as well as a natural treatment facility, but in recent years there are more and more examples of dry ponds. These are under normal conditions without water and often a green grass area for recreation, but during heavy rain these provide stormwater storage and thus unburden the stormwater pipe network (Svenskt Vatten, 2011). SUDS are generally facilities that are created for normal conditions and do not specifically refer to extreme events. Ponds and swales, which can be classified as SUDS, is however often designed for longer return periods and thus can handle more extreme events.

Drainage from urban areas can be classified into two systems, one minor and one major. The minor system handles the frequent storm events whereas the major system enters for the less frequent but more severe storm events, in other words the scenarios with longer return period. The major system may for instance include natural or constructed cannels, ponds and detention facilities, but also the road network. Usage of roads to convey runoff is in fact what in reality has occurred many times in history, but examples on formal acknowledgement is lately given to route heavy runoff via the road network. This means that roads may be planned and constructed so that the roads may work as cannels during heavy storm events (CSIR, 2000).

2.5 Assessment of surface water measures

Design and assessment of flood measures is a complex process that requires a broad view. Large structural measures can easily transfer risk both upstream and downstream. Therefore it is important to have a “top-down-approach” and make sure that measures perform well together as a part of an overall strategy, rather than finding the optimal solution for one specific alternative or problem area (FLOODSITE, 2009, Jha et al, 2012). It is also important to remember that it is impossible to entirely eliminate the flood risk as there will always be a possibility that a more severe rain event than designed for occurs.

Basically, to decide what option to choose the decision maker has to identify those interventions that reduce risk the most to the least societal cost, both in monetary, environmental and social terms (FLOODsite, 2009). As a tool to support this decision, cost-benefit analyses (CBA) have become standard in flood management schemes down in the continent (FLOODsite, 2009, Jha et al, 2012, DEFRA, 2010). The purpose is to assess the monetary value of all costs and benefits for the whole society through measures lifetime in order to determine if the total benefits are greater than the total costs. In other words, a CBA is used to see if an investment is socioeconomically justifiable. To predict surface water flooding, test measures and estimating consequences as average annual damage (AAD) a modelling approach is generally applied (DEFRA, 2010).

Realize a CBA is often time-consuming and tedious. All costs and benefits needs to be identified and quantified which is a complex task. Main costs are generally the building costs (investment) and the maintenance costs. Other indirect costs can also be identified, for instance decreased accessibility. The benefits are the reduction of, for instance, personal injury, damage to properties and commercial losses that the measure brings, and there exists different aids to monetary evaluate these. The benefits are harder to evaluate as direct and indirect impacts from flooding first need to be identified and thereafter how the measure options reduce these impacts. The benefits values are usually much more uncertain than cost values since they comprise future expectations. Further on, no list off benefits and costs can be totally comprehensive. All though, despite these limitations, CBA can be a powerful tool in the decision process and give the decision maker confidence in a choice between different options. (Jha et al, 2012)

Another method to compare measure options is to evaluate the results with different key figures that help the decision maker to interpret estimated flood maps. This was the method used in the Plan B methodology where four key figures were calculated: Number of flooded properties, flooded street surface (area, m^2), accumulated time for estate flooding and accumulated time for street flooded street surface (Ahlman, 2011). Other examples of key figures are number of properties flooded at a certain change or a reduction in the depth of flooding for a particular location. These key figures can be used as indicators and to demonstrate measures effectiveness. Compared to CBA this is a more direct hands-on approach that quickly can provide a decision maker with information on how different options achieve compared to each other (DEFRA, 2010).

2.6 Surface water planning examples

In August 2008 and July 2009 heavy rain events struck Dublin, Ireland, leading to several flood event spread over the city. The Wad river culvert exceeded its capacity why a special study started to investigate the whole Wad river catchment. Flood risk was assessed and different measure schemes including overland flow paths and storage facilities were investigated. The most effective scheme was determined to be a combination of surface flow control and flood storage in a golf course. From an area that during the rain events had over one meter depth of water, surface flow control was established by lowering street level and installing grid openings in a wall. By this, the earlier “bowl-effect” was removed and water would now instead gather in the adjacent golf course, see Figure 2-2. This scheme had a construction cost of 2.7 million Euros, a benefit-cost ratio of 1.7 and it was design to prevent flooding up to events with 100 years return period. (DCC, 2010)

In Växjö, Sweden, the municipality has founded a swale in a park that can flood without taking damage and thus unburden the stormwater network. The swale is shown in Figure 2-3 and that photo was taken from a wooded bridge that allows passage when the area is flooded. The swale is just one part of a bigger system of ponds and swales with an ability to convey and store surface water in the event a heavy rain.

In Poole, UK, a number of properties and two highways were flooded once every two years. The reason was identified to be inadequate capacity in the downstream surface water sewer and the conventional solution to the problem would be to upsize this sewer. Instead a surface water attenuation facility was implemented upstream in one of the watercourses that discharged to the surface water sewer which thus limited the flow entering the same. This solution was made through working in partnership and did not only mitigate the flooding but also improved the amenity in the recreation area were the solution was implemented. (DEFRA, 2010)

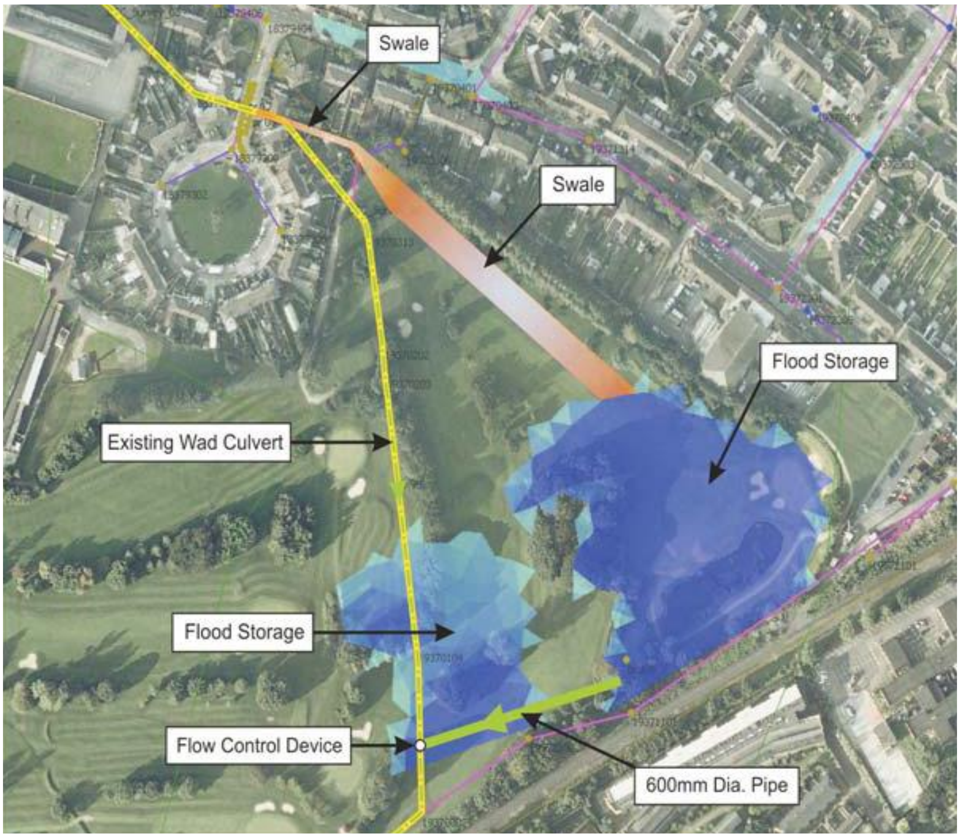

Figure 2-2. Flood alleviation scheme in Dublin. Surface flow paths is controlled and a known flood storage is established (DCC, 2010).

Figure 2-3. Example on swale with the possibility to flood (Photo by Josefine Trädgårdh).

3 Urban flooding simulation

Since urban flooding is an important problem and a growing issue around the world, there is a need of manage and prevent floods as well as flood-related disasters. Because of this urban flood modelling has become an important activity to forecast problem areas and flood behavior, evolution and extent. This chapter gives a short introduction to urban flooding simulation and the tools used in this thesis.

The hydraulics of an urban area is complex with both constructed and natural elements close together. In a rain event the water flow both above grounds on impermeable and permeable surfaces and below ground in pipe networks. The surface and underground flows are linked together with manholes and gully pots and water flows in several different directions, see Figure 3-1. Because of this, simulate water flows and floodplains in urban environments is not trivial.

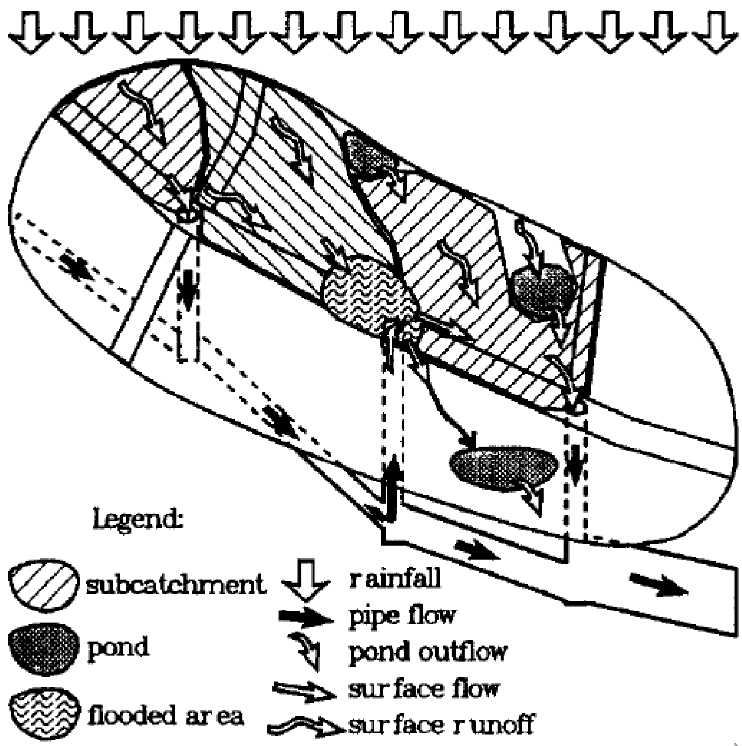

Figure 3-1. Conceptual figure over water flow paths during a rain event in an urban area. Water flows on both the surface and in the pipe network which has insufficient capacity which leads to flooding (Djordjevic et al 1999).

3.1 Development of urban flooding models

One of the early models that combined flow through the sewer system and on the surface was the dual drainage concept. It used a digital elevation model to analyze topography and to identify water pathways and natural ponds which then was used to create a surface flow path model. This surface flow model was coupled to a model for sewer/stormwater systems which already existed and had been used for some time

(Djordjevic et al, 1999). The dual drainage concept has later been addressed as 1D-1D modelling since it used one dimensional calculations in both above and under the surface.

With the development of GIS software and easier access to computing power, the development of urban flooding models has speeded up. 1D-2D models started to being used which model the surface flow in two dimensions, which means that the surface water in the model is not bound to predefined flow path but instead finds its own way bound to the topography. This approach enables a more realistic analysis of overland flow, especially in extreme events (Maksimović et al, 2009).

3.2 Theory behind the modelling software

The modelling software used in this thesis, MIKE FLOOD, consists of two coupled models, a one-dimensional model for the stormwater network, MIKE URBAN, and a two-dimensional model for the water flow on the ground surface, MIKE 21. (2007)

The models are physically based, which means that water movement is calculated by solving mass and momentum conservation equations. This approach has the good property that once the model is calibrated, changes in the physical characteristics can be reliably described without the need of new calibration (Maksimović et al, 2009). Thus, if the model is calibrated and validated, meaning that the results correspond well to reality, the effect of changes in pipes dimensions, connected catchments, topography and so on can be credible evaluated. However, it should always be kept in mind that models still are simplifications of reality.

3.2.1 The one-dimensional network model

Pipe flow simulations are calculated in one dimension in MIKE URBAN, which uses MOUSE (Model Of Urban SEwers) as calculation engine. This is a computational tool for simulations of unsteady flows in pipe networks which can handle alternating free surface flow (channel flow) and pressurized flow conditions. The calculations are performed solving free surface flow equations (Saint Venant) using implicit numerical finite difference solutions. These algorithms provide solutions for multi connected, branched and looped networks. The physical basis behind the equations is continuity in flow and conservation of momentum. (DHI, 2004)

The model uses only free flow equations by implement a fictitious narrow vertical slot extension of the pipe cross section. By this the pressurized flow can be described with the same algorithms as free flow, and thus a smooth transition is ensured between the two flow regimes. (DHI, 2004)

The finite differences are calculated in time and space. For space, a computational grid of equal spaced staggered h and Q points (pipe water level and pipe flow) is created along the pipes. This means that the model alternates between calculate pipe flow and water level, but always starts and ends with the water level in the manholes, see Figure 3-2. This is done for each time step. (DHI, 2004)

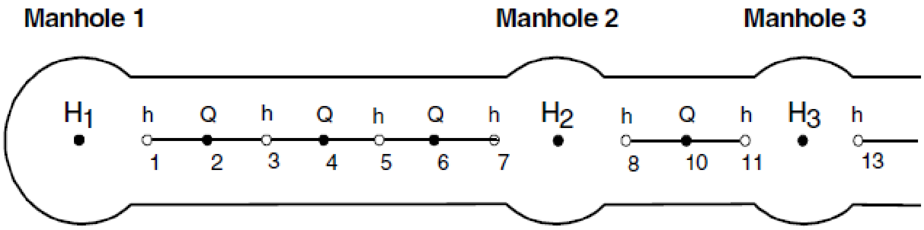

Figure 3-2. Computational grid in the pipe network, consisting of stacked points of h (water level in pipe) and Q (flow) (DHI, 2004).

The numerical analysis is performed in a six point Abbott scheme, see Figure 3-3, where six points are used to solve the implicit differential equations. For the continuity equations the scheme is centered over the level points, and for the momentum equations the scheme is centered over the flow points. (DHI, 2004)

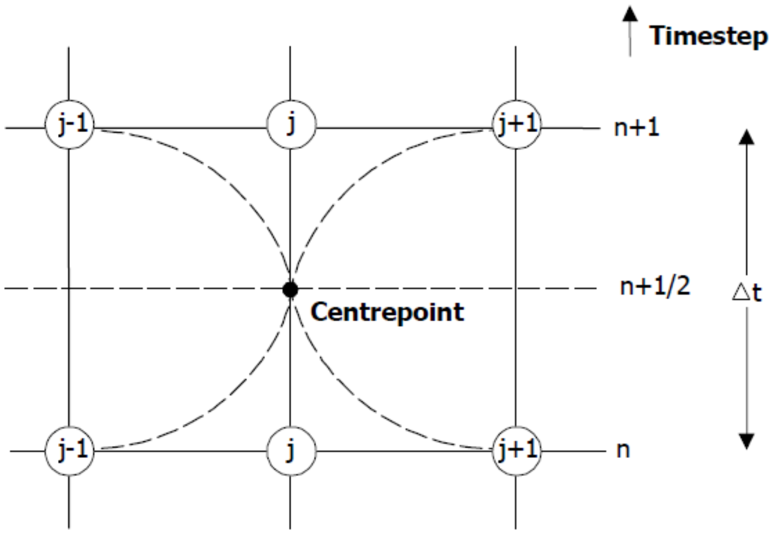

Figure 3-3. The principle of the six point Abbott scheme used for the finite differences approximation of the flow equations in the 1D model (DHI 2004).

3.2.2 The two-dimensional ground elevation model

Ground surface flow is simulated in the software MIKE 21, which is a numerical modelling system that simulates unsteady two dimensional flows. Just as for the MOUSE model, MIKE 21 uses continuity and momentum conservation equations for solving the flow and water levels at different time steps. (Mike by DHI, 2011)

To solve the equations of continuity and momentum conservation in time and space the model uses alternating direction implicit (ADI) technique. This means that the equations are solved in one dimensional sweeps that alternates between x and y-directions, which is illustrated in Figure 3-4. For best approximation, the sweeps are performed both “up and down” and the numerical analyses are performed at time step n+½, as for the 1D model. (Mike by DHI, 2011)

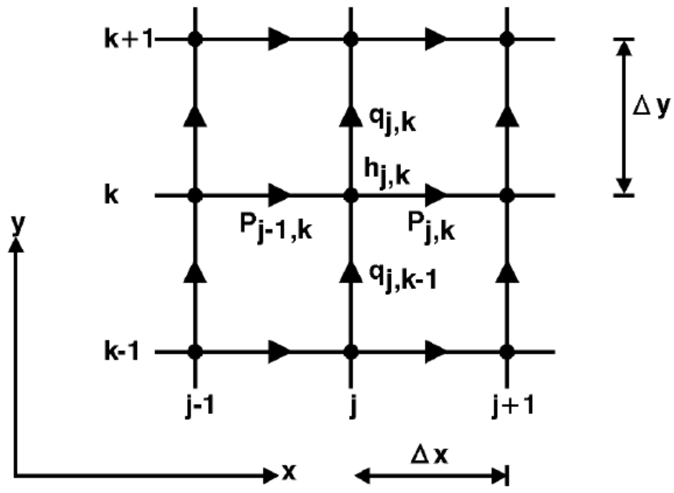

Figure 3-4. Difference grid showing the principle of the alternating technique for solving continuity and momentum conservation equations. p and q is the flux in the different directions, and h is the water level (Mike by DHI, 2011).

3.2.3 The 1D-2D coupled model

The one-dimensional and two-dimensional models can be coupled using different links depending on the surrounding conditions for the location where a flow takes place between the models. For underground pipe networks the links are called "urban links" and these describes the interaction when the pressure line in the pipes exceeds the top of a manhole or when an overland flow enters the network through a manhole, see Figure 3-5 and Figure 3-6 (DHI, 2007).

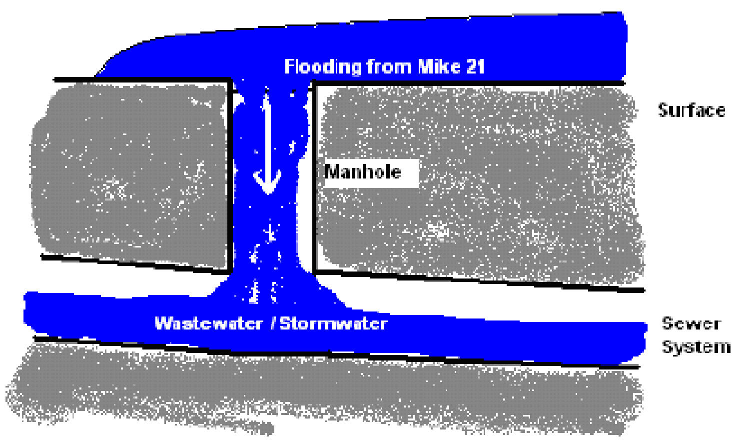

Figure 3-5. Overland flow reaches an open manhole which leads to an interaction where water is transported from the 2D model to the 1D model (DHI, 2007).

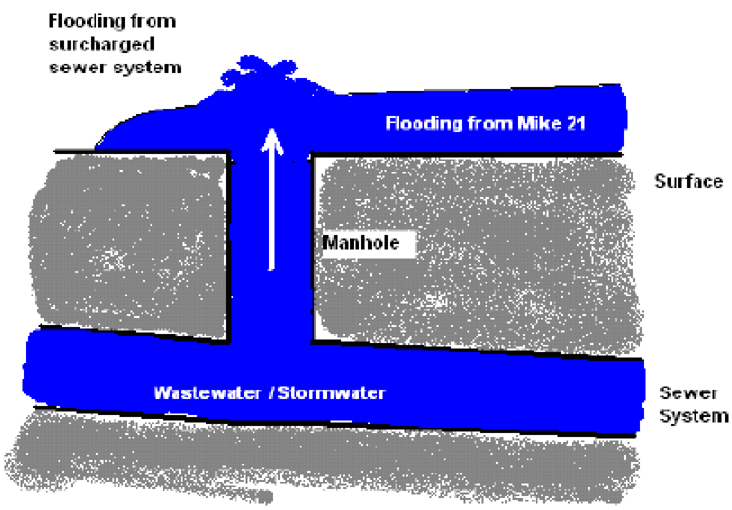

Figure 3-6. The capacity in the storm water network is reached which leads to an interaction where water is transported from the 1D model to the 2D model (DHI, 2007).

The regular MOUSE model ends at the top of the manholes and flooded water are gathered in a fictitious bowl to be returned into the network system when the load has decreased. In the coupled model the flooded water instead spread on the surface which does not only provide a picture on surface water flows and flooded areas, but also more accurate describes the flows in the network for these conditions. This because of the flows on surface between manholes are included, but also because of the energy levels in the network become closer to reality.

The manholes in the 1D model are coupled to nearest grid square in the digital elevation model, see Figure 3-7, which then forms and extra input or output in the difference grid in Figure 3-4. The interaction between the models can be controlled by different parameters like maximum flow and orifice area and this can be used to, for instance, limit the flow through a trench intake if it is assumed to be clogged by leafs and debris (Mike by DHI, 2011).

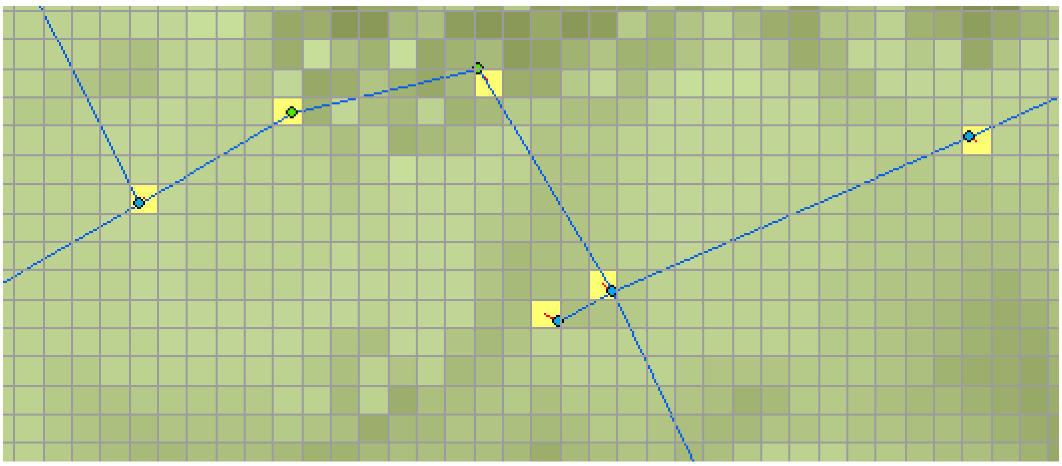

Figure 3-7. The network models manholes coupled to the grid squares in the digital elevation model.

Precipitation can in the coupled model be handled in two different manners, either by the 1D-model or the 2D-model. This means that either the 1D model feed the 2D-model with excess water, or the 2D model feed the 1D model with the runoff water that reaches the manholes (Mike by DHI, 2011). Both alternatives have their advantages and disadvantages. Most common is to use the first alternative where runoff flow to the network model is calculated by the ordinary and well established time-area method. This is relatively easy to calibrate a gives a decent description of reality. The second alternative gives in theory a more realistic model since the precipitation falls directly on the digital elevation model and water finds its own way to the manholes. However it is harder to estimate runoff flow velocities since resolution increases, and thereby calibration also becomes trickier. The calculation effort also increases and thereby the expenditure of time.

4 Assessment methods

The purpose of having assessment methods is to simplify the interpretation of simulated flood results and help decision makers to rank flood mitigation measures. Based on gathered experience two different methods were chosen as tools for assessment of the flood mitigation measures. The two methods had similarities as they both were GIS-based analyses of the simulated flood extent results. The first method was a key figure-based assessment much similar to the one used in the Plan B methodology. The second was a cost-benefit analysis based method similar to those recommended in many flood management schemes, see Section 2.5.

These two methods were chosen to be evaluated regarding user friendliness and usefulness in the outputs.

4.1 Assessment method 1: Key figure assessment

The intention with this assessment method was to have a simple number that can help a decision maker to interpret the flood extent maps received from simulations and provide a quick comparison of measures. In theory this assessment method is simple. The efficiency of the flood mitigations measure was evaluated by using two key figures: number of flooded buildings and total flooded street area measured in square meters. Flooded street area was defined as the amount of street area covered with water of more than five centimeters depth. In the same manner flooded buildings was defined as buildings with more than five centimeters water at the buildings border.

The limit of five centimeters is set with a couple of influencing factors in mind. First, the accuracy in the used digital elevation model (DEM) was about five centimeters, meaning that flood extent with depth below becomes more uncertain. Second, only maximum flood extent and depth are analyzed. Low depths are many times also short in time, meaning that water advancing and retreat in a short period of time. Third, the low water levels are filtered to better display and analyze problem areas. A couple of centimeters of water on a street are generally not a problem, and the same applies for water standing outside a house foundation.

Figure 4-1. Flood extent overlaying land use layer.

The key-figure calculations were performed with GIS analyses where a land use layer displaying buildings, streets, and green and forest areas was used together with the results files for the maximum flood extent, see Figure 4-1. With help of GIS software the street area covered by water as well as the number of buildings that are intersected by the flood extent could be easily calculated. The GIS analyses were performed with the software ArcGIS.

4.2 Assessment method 2: Cost-benefit assessment

The intention with this assessment method was to perform deeper analyses of the measures impact and present a result that gives a decision maker an immediate view on the measures efficiency in relation to its cost. The main process in the used cost-benefit analysis (CBA) assessment was to estimate the cost for the society (including the individuals) of a flood scenario under current conditions and estimate a societal benefit by evaluate how the cost changes with changed conditions.

A CBA assessment is generally complicated since the methodology implies a comprehensive assessment and it is hard to establish generalized costs for all the impacts on the society. It was early found that no full CBA assessment was would be able to perform within this thesis due to the time limitation. But, to still be able to test and evaluate the method, it was perform in a simplified manner.

The CBA calculations were performed as GIS analyses and were handled by the software DHI Flood Toolbox (an add-on to ArcGIS). The same land use layer that was used in the key-figure assessment was also used in the CBA assessment. A damage table with generalized damage costs per unit area was defined as input to the damage assessment, see Table 4-1. These generalized damage costs was supposed to reflect the societal costs for certain flood depths. Flood Toolbox uses the damage table, the land use layers and the simulated flood results to calculate a total damage cost for each flood scenario (one damage cost for each simulated return period). To establish the figures in Table 4-1 is a complicated process in itself why simplifications and assumptions were made. In these simplifications different intervals were used which is why the figures in Table 4-1 does not change for each step.

Table 4-1. Generalized costs values for the damage assessment within the CBA.

Flood depth [m]	Costs [SEK/m^2]			
	Buildings	Streets	Green areas	Forrest
0-0,05	0	0	0	0
0,05-0,10	3400	0	0	0
0,10-0,20	3400	210	0	0
0,20-0,50	4400	210	20	0
0,50-1,00	7000	340	50	20
>1	7000	750	100	50

To establish the generalized building damage cost, statistics on flood cases were used. From summaries of old flood cases it could be read out that the average cost for one building suffered from a flood was roughly 30 000 SEK (Göteborg Vatten, 2012a). The generalized values were established with help of GIS analysis that counted the number of square meters of different flood depths and also the number of affected buildings. With the approach of deeper flood levels having larger damage cost but that also smaller flood depths having relatively large damage cost, different flood depths were given generalized values so that the total damage cost divided with the number of affected buildings would be slightly over 30 000 SEK. This to also cover some of the cost for the individual (like for instance goods with emotional value), though no real references where available that declared what this cost might be. This way of establish the generalized costs was performed as a smaller iteration process were different scenarios where compared and the costs adjusted until them and the results were considered reasonable.

The generalized damage costs for the streets was taken from an example were the software has been used (DHI-WASY 2011). These costs reflect the damage to the street caused by the water but also the impact the flood has on the street users in terms of accessibility, time losses etc.

The generalized damage costs for green and forest areas have been estimated with the defined building and street costs in mind. They were chosen to be low as it is reasonable that a green area can flood without any high costs. However it would still be worth something for the residents to not have the green areas and forests in their neighborhood flooded. These costs can also include extra erosion, destroyed flowerbeds, clean up of debris etc.

The result from the evaluated scenario costs can be plotted in a damage probability curve, see example in Figure 4-2. The damage cost for the intermediate return periods is by the software estimated and an average annual damage (AAD) cost is calculated as the integral of the damage probability curve. (DHI-WASY 2011)

The benefit is calculated as the difference in AAD of the compared scenarios and the reference scenario. The benefit is then compared to the annual measure and maintenance cost. In these calculations no interest is included, meaning that the annual investment cost simply is calculated as the investment cost divided by the prospected life length and that the benefits are equal for all years.

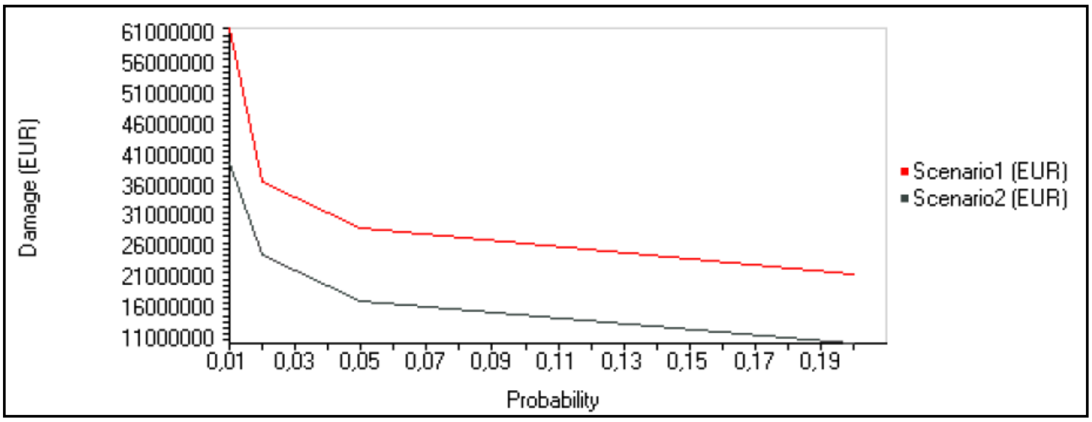

Figure 4-2. Example on a damage probability curve for assessment of flood measures. In Scenario2 the damage cost has been lowered and thus has a lower damage probability curve than Scenario1. (DHI 2011)

Last in the CBA, a net benefit cost ratio (NBCR) was calculated according to equation (4.1). The NBCR compares the benefits to the cost and indicates which alternative that gives most revenue compared to the investment. A NBCR above zero indicates a profitable investment for the society and the higher number the more profitable.

$$NBCR = \frac{Benefit - Cost}{Cost} \tag{4-1}$$

5 Case study

The methodology development was performed as a case study and for this a relevant case object with a history of pluvial flooding was needed. To apply and develop the methodology in its full extent, good quality data for both the storm water network and ground elevation was needed to be available for the case object area. The suburban area of Kopparkärret was chosen as it fulfilled these criteria.

5.1 Area description

Kopparkärret is a part of the southwest suburbs of Gothenburg and is a residential area consisting of single and linked houses, see Figure 5-1. The area is bounded by a highway in the east, forest and natural areas to the south and east, and a similar living area in the north. The area is stretched in north-south direction, about 800 meters long and 300 meters wide. The total analyzed area with adjacent natural areas is about 50 hectares.

Figure 5-1. Left, the Gothenburg area with the studied area marked out. Right, aerial photo over Kopparkäret (Eniro, 2012).

The area has a duplicate sewer system where the storm water system, see Figure 5-2, consists mainly of concrete pipes in dimensions from 225 millimeters to 600 millimeters. Recipients are a trench in the south west corner that connects to a stream via a culvert and a small stream flowing from east to west through the north part. The stormwater system is thus split into two parts – one that flows to the north and one that flows to the south, from now on referred to as the north and south system.

The area has been exploited in stages, with most houses being built during the seventies for the northern part and the last ten years for the southern part. Within the area there are three joint properties whose internal network system is not owned by the municipal water utility which means that information regarding these systems is not in the databases used for building the model.

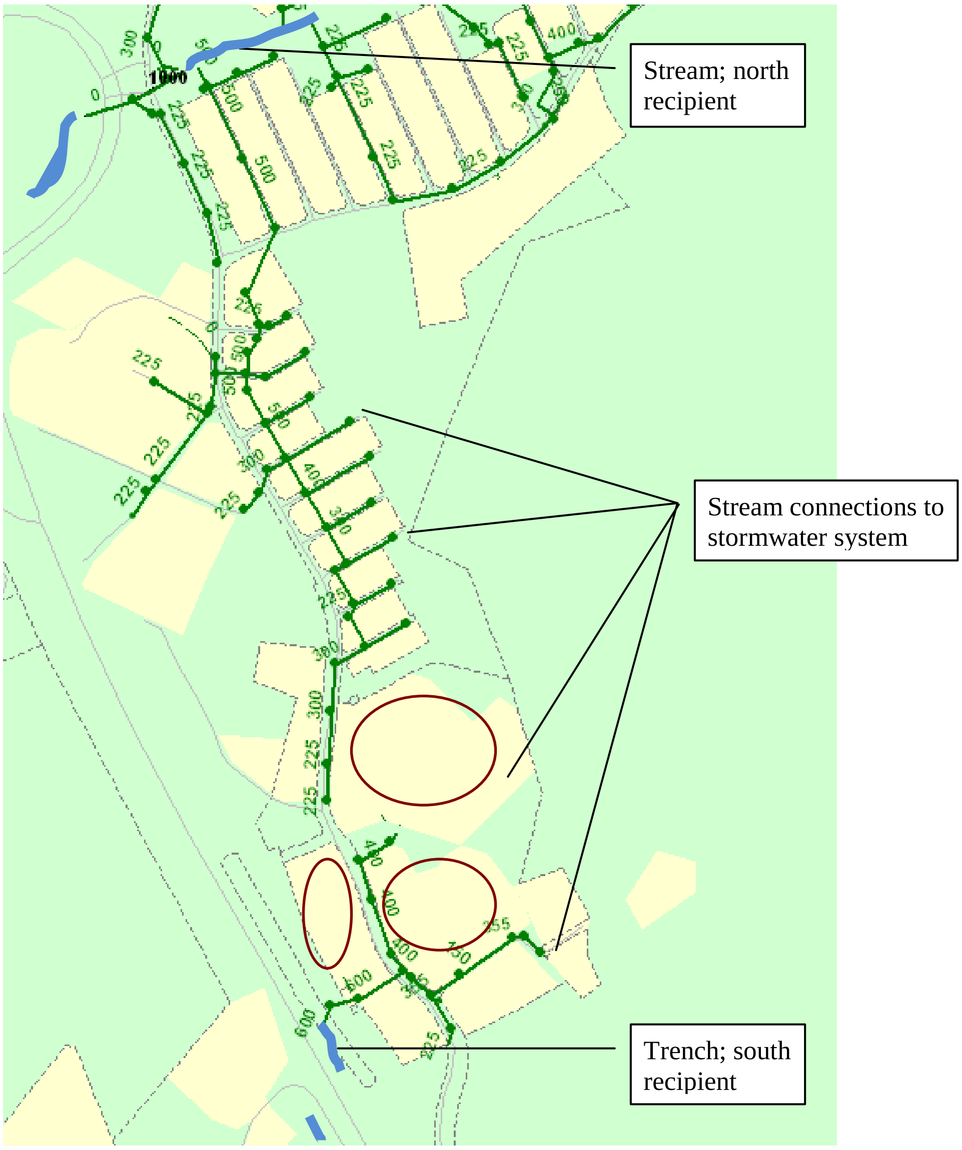

Figure 5-2. The storm water system for the studied area of Kopparkärret. In the area there are three joint properties, marked with ellipses, whose systems are not included in Gothenburg Waters databases. Note that the system is divided into a north and south part (Göteborg Vatten, 2012b).

To the east there is a large natural area bounding the residential area. This natural area is declined down towards the houses and consists of the typical Swedish west coast nature with mixed coniferous and deciduous trees and plenty of outcrops. The natural area is drained by a number of streams the flows down to the residential area and there is lead into the stormwater system. There are four main steams which connect to the stormwater system at the streets Hovås Nypongång, Hovås Slånbärsgång, Hovås Mullbärsgång and Uggleåsvägen.

5.2 Flooding history

On Sunday the 14th of August 2011, warm and humid air flew in from south and a heavy rain storm struck the Swedish west coast. The local variations were large in both intensity and volume; the Swedish meteorological and hydrological institute's weather stations measured 56 mm precipitation west of Kungsbacka, but only 23 mm in Gothenburg (SMHI, 2011). Gothenburg Waters own precipitation gauge measured 57 mm in just less than five hours, which corresponds to a return period of 28 years. A resident in the district Askim, not far from Kopparkärret, measured a precipitation of 80 mm in five hours, which corresponds to a return period of 80 years (Göteborg Vatten, 2011).

Within the case area about 25 estates reported flooding and most of them were in the southern part, see Figure 5-3, but more people where affected since the roads also were flooded, see Figure 5-4. The flooding was concentrated to the north joint property, the street Hovås Mullbärsgång, and the border of the southeast joint property, the street Uggleåsvägen. From eye witness reports it can be concluded that the flooding was much due to large surface water flows that came down from the surrounding higher situated natural areas and could not be transported away from the area quickly enough. Northeast of Hovås Mullbärsgång there is an intake for a stream coming down from higher natural areas to the stormwater system, and it was captured on film how this intake was overflowed and the stream continued on the ground to the estates, see Figure 5-5. None have however reported that water came up from the foul water systems which indicate that these systems worked properly.

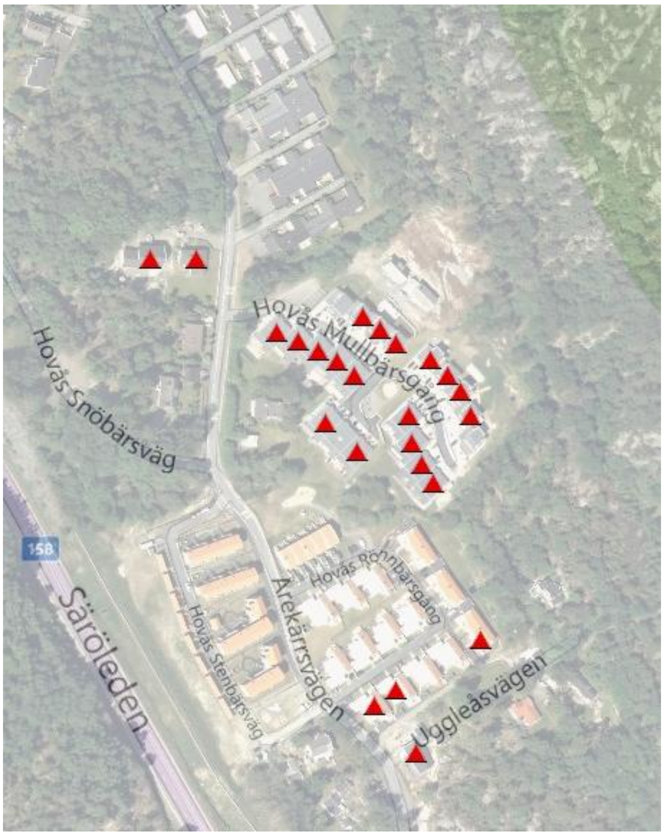

Figure 5-3. Estates in the south part of the area that has reported flooding due to the heavy rain 14.08.2011.

Figure 5-4. Water accumulation in the crossing of Årekärrsvägen and Hovås Rönnbärsgång during the rain 14.08.2011. Photo by My Ring.

Figure 5-5. The stream intake north east of Hovås Mullbärsgång is overtopped during the rain 14.08.2011 and water flows on the surface into the estate gardens. Photo by Fredrik Gorthon.

Figure 5-6. A man walks in knee-high water at the street Hovås Mullbärsgång during the rain 14.08.2011. Photo by Fredrik Gorthon.

5.3 Model set up

The model used for the analyses consists of two parts, a one-dimensional network model and a two-dimensional ground elevation model. These were configured separately before they were coupled.

First intention was to also include the most northern area in the model, see pipes without catchment background in Figure 5-7, but this was later decided not to be included since the flows to this area would be very hard and tricky to describe due to a complicated catchment including bogs and defuse borders. This choice was motivated with the intention described in Section 1.4 that the study would rather be limited geographically than in depth.

5.3.1 Network model

The pipe network was imported from GIS databases into the 1D network model, which was partly automated by the software. However some modifications and further model building was necessary to create a complete network model. The neighborhood of Hovås Mullbärsgång, which suffered the most in the rain of August 2011, consists of a joint property which means that the pipe network is not own by the municipal water utility. Therefore this network was not available in GIS databases at Gothenburg Water but had instead to be inserted into the model by hand from record drawings from the Urban Planning Department. Within the modeled area there are two additional joint properties where the pipe network data was not as easily accessible but for these a rough approximated network was drawn. Based on observations and their location, it can be assumed that the network for these locations does not need to be totally accurate in order to reliably simulate the flooding due to a heavy rainfall, since the problems occurs downstream these locations. The network model can be seen in Figure 5-7.

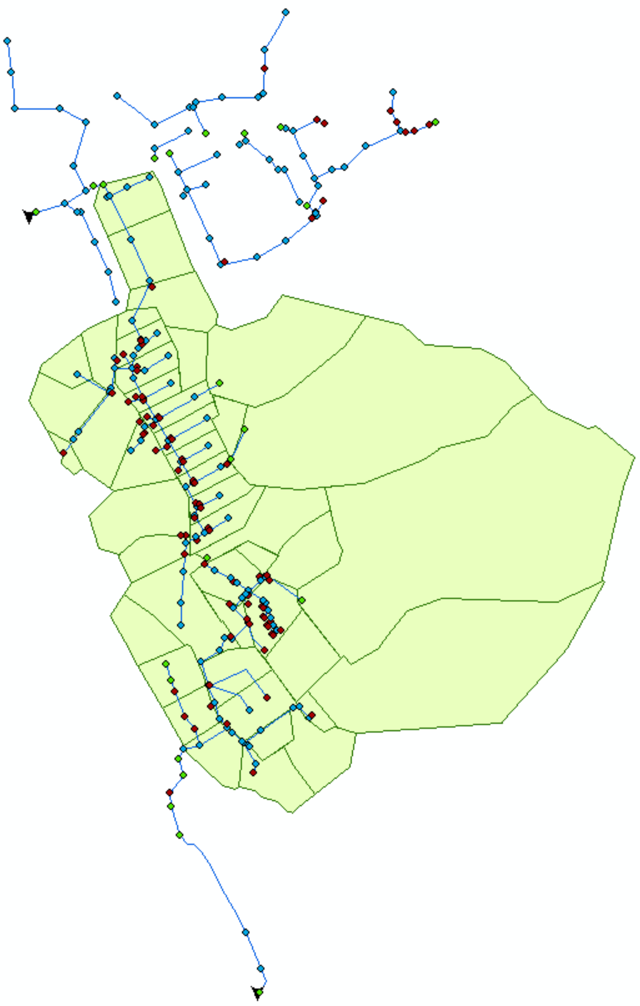

Figure 5-7. The network model for the case area during the model building process. In the figure one can see closed manholes (blue), gully pots (red) and trench intakes (green). Yellow fields are catchments used in the time-area method and the black triangles are the outlets where the model ends.

The simulation software needs nodes at all pipe branches and therefore a number of fictional manholes were needed to be placed at all branch pipes. The extra volume this creates was considered to be negligible.

For the time-area method that the model uses to simulate the precipitation runoff, catchments needed to be defined in the network model. This was done by studying different maps and elevation contours, since the DEM was not available at that stage. The catchment was however later checked against the DEM and some adjustments were done. The catchments varied in size and were defined so that the included area would be of fairly similar type, i.e. big natural areas and impervious areas were not mixed since this could lead incorrect estimations of time-flow-patterns. The catchments imperviousness were calculated as a weighted value from GIS layers where roofs, roads and green areas where given different imperviousness. The initial loss was at first set to 0.5-5 mm depending on the catchments characteristics, and the time of concentration was calculated by using a fixed value on the runoff velocity.

Recipients are a stream in the northern part and a trench in the southern part. The stream flows through a culvert and has after that a high capacity, and therefore the model ends after the culvert. At the south the pipe network ends in a trench, but the water then flows into another pipe under a highway and then through another trench and thereafter another pipe before the water reaches a stream in far south. Since it not could be excluded that these pipes could limit the capacity and dam up the system, they were added to the model. This is shown as the “tail” in Figure 5-7.

5.3.2 Elevation model

The Swedish mapping, cadastral and land registration authority (Lantmäteriet) is by the time of this reports printing in the middle of the process of creating a new national elevation model with much higher quality than former models. The whole country has been scanned by airborne laser and some regions are finished and could be used for instance to analyze flood risks, which is one of the main intended uses. However, the Urban Planning Department of Gothenburg has ordered an airborne laser scanning of the city area with even higher quality. It is specified to have a point density of 10 points per square meter which can be compared to the new national elevation model which have a specified scanning point density of 0,5-1 point per square meter (Lantmäteriet, 2012). In the higher quality scanning the mean error in plan shall be less than 15 centimeters and the accuracy in height +/- 5 centimeters on hard surfaces (Göteborg Stad, 2010). This scanning has been used as input to the elevation model. The elevation model was created from a point cloud, which is the form of the unprocessed data from the airborne scanning, and set up in form of a raster where every cell is assigned a value on the elevation. Grid size was initially chosen to be two by two meters both in the creation of the DEM and in the calculations performed within the simulation software. However, during the processes of building and calibrating the complete model, it was discovered that the DEM had inadequate quality to be able to properly describe the surface water flows. This was partly because of the chosen resolution but mostly because of the software that processed the point cloud created a DEM that was partly to smooth and partly to sprawling, and overall quite flaky.

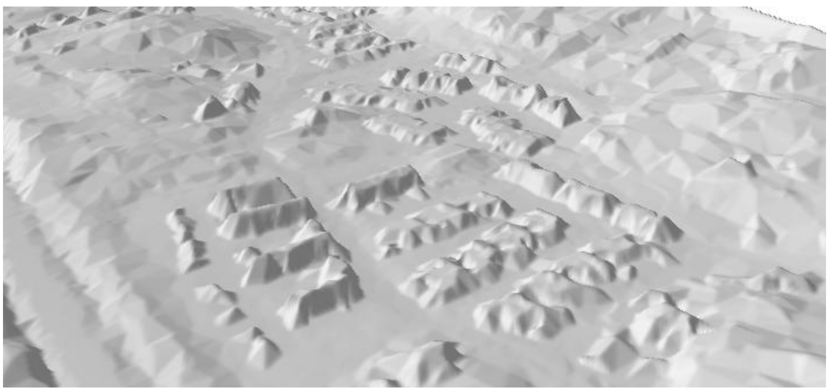

Figure 5-8. The digital elevation model (DEM) projected in 3D.

The DEM was reprocessed with different software that used GIS-layers as help in the processing of the point cloud which made it easier to classify the different points. The final used DEM had instead a grid size of 0.5 meter in square, but in spite of this low cell size some building "melted together". These flaws were manually removed by editing the DEM. Final used DEM is shown in Figure 5-8.

5.3.3 Coupled 1D-2D model

The completed network and elevation models were joined into a coupled model. This was done by linking chosen manholes (open manholes like gully pots) in the network model to the nearest grid square in the DEM, see Figure 3-7. An overview of the coupled model is shown in Figure 5-9.

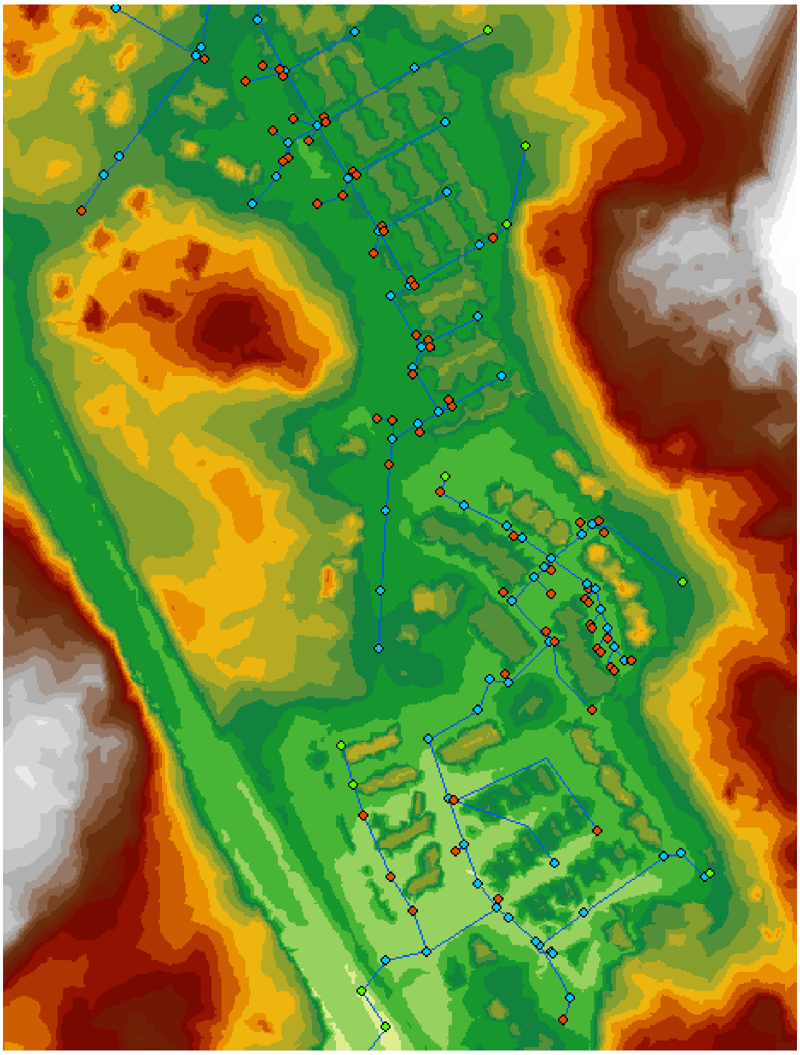

Figure 5-9. The 1D network model coupled with the 2D elevation model.

The connection parameters can be set individually for every coupled manhole. In this model three parameters sets was used; one for all sealed covers (i.e. parameters defines manholes not coupled), one for all gullies on impervious ground and one for all gullies on pervious ground (i.e. trench intakes). Some of these couplings where individually adjusted in the calibration, see further in Section 5.3.5.

5.3.4 Delimitations in the model

Due to lack of data showing the correlation between precipitation and the flows in pipes and on surface, the model was only calibrated and not validated. Basically, there was only one big rain event available with good documentation, which has been used for the calibration of the model. For a more reliable model two events are required; one to be used for calibration and one for validation. Normally a longer measurement period is used where half of the period is used for calibration and the other half for validation. For the purpose of this thesis, only calibration was regarded adequate.

As the intention of this thesis was to limit the geographical extension rather than the depth of the study, some parts of the original intended model area were left out, as mentioned above in beginning of Section 5.3.

5.3.5 Calibration of model

The rain event in August 2011 was used for calibration of the model. A precipitation time series from a private rain gauge placed in Askim, about 4 km from the modeled area, was used as precipitation input, see Figure 5-10. The result was calibrated against data on water depths and spreading in the form of eye witness reports and photographs as no real measurements on flows were available.

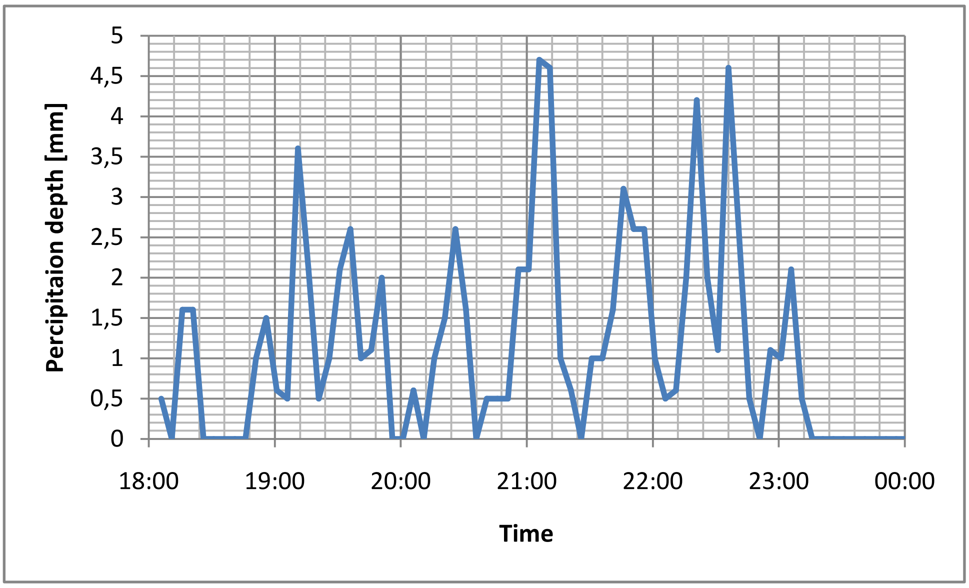

Figure 5-10. Precipitation time series from 14.08.2011 used in the calibration simulations.

The model was first ran as a pure 1D model since this was much faster than running the coupled model and thus some calibration and assumptions could be handled quicker. The first 1D result showed that for a main pipe in the north part the pressure line exceeded ground level, see Figure 5-11, which was correct as flooding did occur in this part. If the capacity was exceeded as much as in the real rain event could however not be determined before the first coupled 1D-2D simulation results. For a main pipe in the southern part the pressure line did not exceed ground level thus causing no flood, which was not correct since this area was subjected to severe flooding during the real rain event. Ergo calibration was obviously required for this part. In this manner a pre-calibration was performed for the 1D model, mainly by adjusting the catchments imperviousness to increase the runoff.

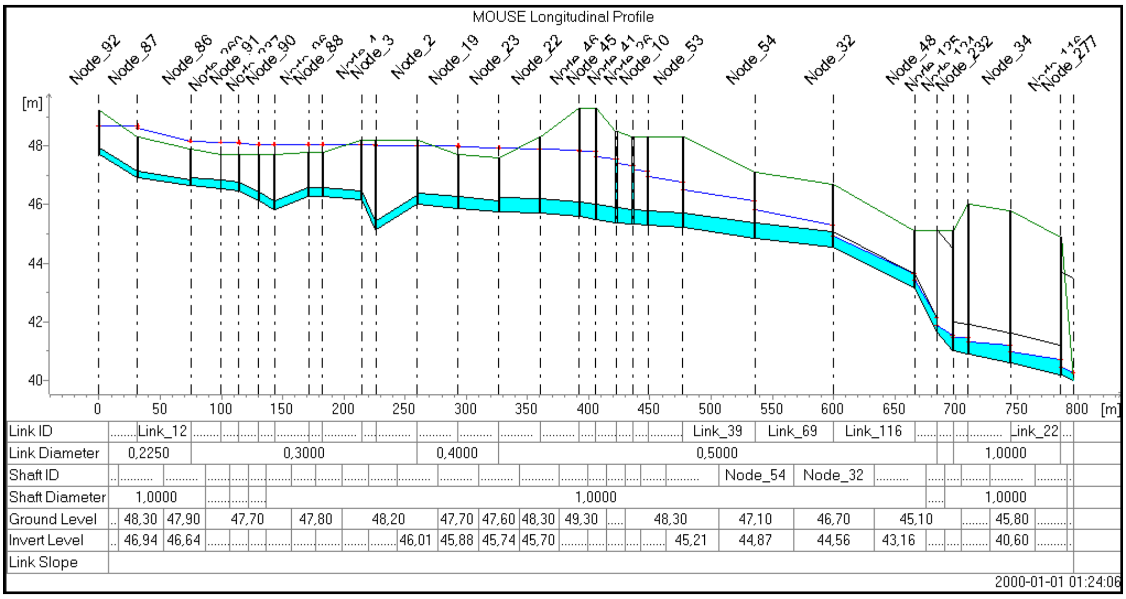

Figure 5-11. First model results from 1D network simulation, north part of stormwater system. Pressure line exceeds ground level which was according to the real rain event.

Setting up the coupled model turned out to be a struggling process as the numerical calculations reached instability in several occasions. These instabilities were so-called “blow-ups” where the water depth in the 2D model in just a few times steps reached extreme and unrealistic levels, well above 10 millions meters. When these high levels are reached the software automatically aborts the simulation, and the cause of the instability is not obvious to point out. After many different set-ups and trial runs the most likely reason to the blow-ups were considered to be the high resolution that lead to the numerical calculations becoming very sensitive. This meant that very small changes, like moving a manhole two meters, could make the model stable. Biggest stabilizing action was the exchange of the DEM, but still some blow-ups occurred. The instability was then handles by moving manholes small distances in a manner that would have negligible affect the models simulated results.

The first simulation results from the coupled 1D-2D simulation showed a flood evolution (how the flood started and how it spread) that did not coincide well with the real rain event. For the area around Hovås Mullbärsgång the results showed some similarities with the real rain event but showed much less spread and much deeper levels locally, see Figure 5-12. The area around Uggleåsvägen showed no flooding at all which was also incorrect compared to the real scenario. As is seen in Figure 5-4, this area had a severe water gathering in the street and a couple of estates suffered from flooding.

Much of errors in flood extent were explained by the errors in the first DEM. Its sprawling surface made the water gathers in deep ponds and prevented the water from spreading in a natural manner. With the new DEM used in the model the simulated water flow paths were more realistic and the locally deep depressions were removed.

The major part of the calibration was to adjust the imperviousness of the catchments. Since model was showing no flood, or lower flood extent than the real scenario, the modeled runoff was too low and hence the imperviousness was too low. The natural areas to the east of the residential areas consist of a fairly high amount of outcrops, which also makes a higher imperviousness reasonable. The very first assumption was an imperviousness of 10% in the natural areas catchments. This was gradual increased to 40%, but still the simulated result was not satisfactory.

With the hypothesis that the natural areas has larger surface water storage than the developed areas and that when this is full almost all precipitation turns into runoff, catchments parameters was changed. An initial loss of 20 mm and an imperviousness of 80% were assumed for all the natural areas catchments, and the time of concentration was also increased. Also, the time area curve was for the natural areas catchments changed from a linear to an exponential as it is reasonable that for the natural areas in question that flow from the area increases exponential and not linear. These changes together made the runoff to be greater and concentrated later in time. After simulations the imperviousness was lowered to 60%, which then gave a result that corresponded relatively well in both flood extent and depths in the created ponds, see Figure 5-13. For the purpose of this thesis, which is to develop the methodology and not to perform a complete investigation for case studied area, the result were found satisfactory.

Some calibration was also made in the network system. In the aftermath of the flood event it was discovered that the stormwater pipe under Uggleåsvägen held lots of sediments. This pipe had therefore a decreased capacity during the rain event and was given a fictitious diameter in the model to simulate this condition when reproducing the flood event in the calibration process. Also, the flow capacity at the stream intakes were adjusted to some extent as it was reported of debris and constrained inflow, but most intakes and gully pots were given standard values.

The rain event of the 14th of August 2011 had a large local variation which means that it could have fallen greater precipitation over the case study area then over the rain gauge situated about four kilometers away. There were also indications on it being so and that the used rain series then would be and underestimation on the fallen precipitation. However, this would mean that the calibration is an overestimation on the runoff and on the surface water derived from the precipitation. Thus the model was on the safe side.

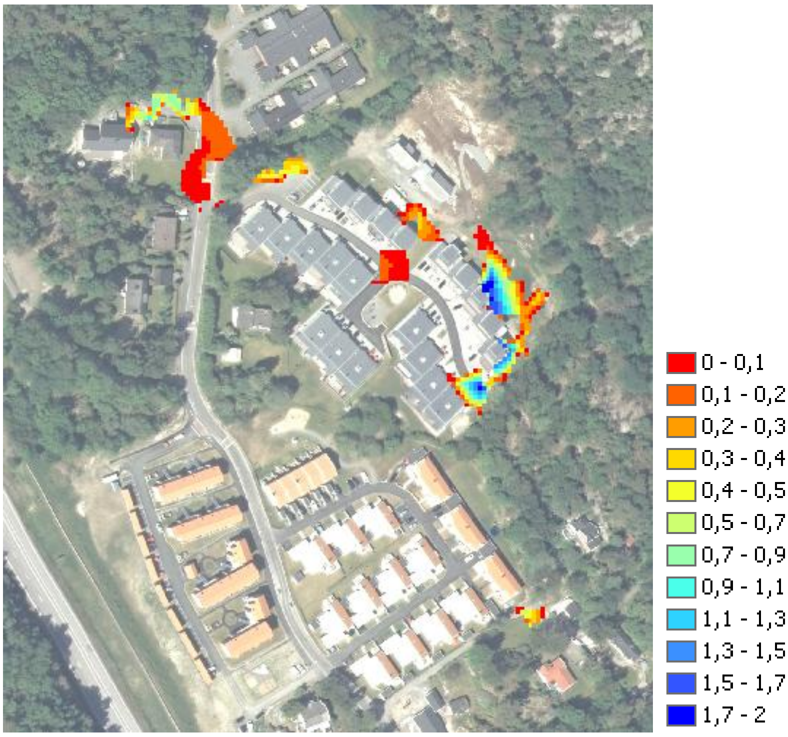

Figure 5-12. First 2D simulation results showing maximum flood depth in meters; a result that did not correspond with the real rain event. Scale is showing flood depth in meters.

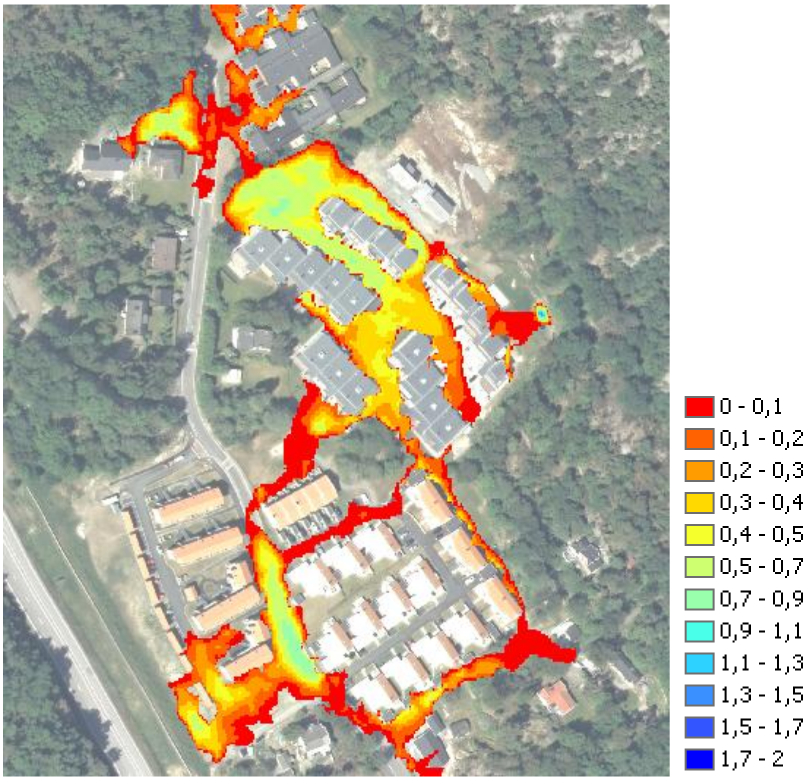

Figure 5-13. Results from final calibration showing maximum flood extent with the rain series from 14.08.2011 as input. Scale is showing flood depth in meters.

5.3.6 Simulation set up

After the calibration was finished some changes were made in the model to account for measures that had been taken in the area after the storm. The pipe at Uggleåsvägen that was given a fictitious dimension was restored to the original dimension because of regular flush and inspection of the pipe had been scheduled to cope with sediments gathering in the pipe. For the same reason the stream intakes were restored as these after the storm will have a more regular inspection.

In the simulation Chicago design storm (CDS) rain series were used. CDS rain series are continues rain series developed to be used in design calculations and analyses of existing facilities and is recommended to be used in computer modelling by The Swedish Water & Wastewater Association (Svenskt Vatten 2011b). Four rain series were used with 100, 50, 20 and 5 year return period, all with duration of four hours. Total simulated time was set to eight hours. As an example the 50 year rain series is shown in Figure 5-14. The calculations behind the CDS rain series is shown in Appendix 2.

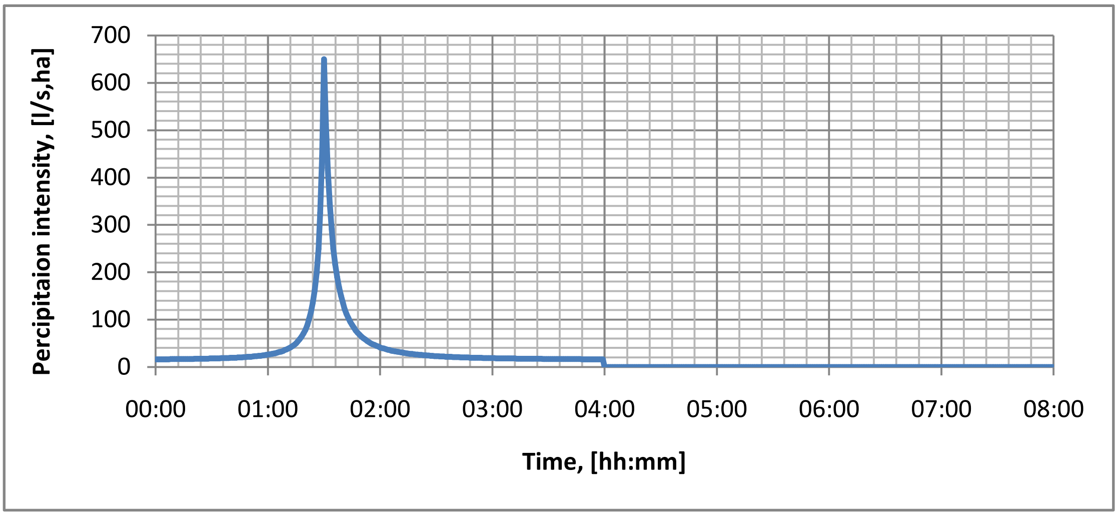

Figure 5-14. CDS-rain with 50 year return period that was used in the flood simulations.

5.4 Measures

With a calibrated and working model, the measures were implemented by modifying the DEM and the network model. Four alternatives were identified and chosen by observing flood evolution, studying the topography and comparing experiences.

The studied area is partly confined which makes conveyance of water hard. Storage within the area is easier to implement, but the capacity is limited due to small available spaces for constructing swales or dry ponds.

The measures were intendend to decrease the flooding impact of the part of the area that suffered the most by the heavy rain event 2011-08-14, see Figure 5-3. The location of the alternative measures is shown in Figure 5-15 and their caracteristics is further described in the following subchapters.

The building and maintenance cost for the alternatives was roughly estimated by using thumb rules given by experienced engineers (Ekberg, 2012).

Figure 5-15. The location of the measures of the four alternatives.

5.4.1 Alternative 1

First alternative was a dry pond flood storage located southwest of Hovås Mullbärsgång, combined with a trench that leads water from the stream intake, when overtopped, in the forest around the estates. Today the location of the intended dry pond is a green area that holds a playground. This function would be possible to remain also if the area would be lowered to work as flood storage. A gully pot with large capacity was placed in the middle of the pond which allowed drainage of the pond after a rain event as well as unburden the network system in high flows and insufficient capacity downstream. The investment cost for the alternative was estimated to 310 000 SEK and the yearly maintenance 5 000 SEK. An overview of the measure can be viewed in Appendix 4.

5.4.2 Alternative 2

Second alternative was a dry pond storage located west of Årekärrsvägen, across from Hovås Rönnbärsgång. Today this location is a green area, playground and partly parking space, which all could be remained if the area is remade into a dry pond. In this alternative there is also an additional pipe installed under the noise barrier that creates a connection between the dry pond and the trench on the other side, making it possible to use as flood storage. This dry pond also included a gully pot for both drainage and unburden of the network system. The investment cost for the alternative was estimated to 170 000 SEK and the yearly maintenance to 5 000 SEK. An overview of the measure can be viewed in Appendix 5.

5.4.3 Alternative 3

The third alternative was the combination of alternative 1 and 2. The investment cost was thus the sum 480 000 SEK and the yearly maintenance 10 000 SEK.

5.4.4 Alternative 4

The forth alternative was a combination of bigger measures. These were made with no consideration of the measures feasibility or convenience, but to gain high performance. The purpose of including this alternative was to create a major difference between the assessed measure alternatives and by that be able to better test the assessment methods.

The fourth alternative included a dry pond northwest of Hovås Mullbärsgång where a current small stream was widened into a swale. The adjacent parking space is also lowered to create flood storage. From the stream intake in the forest a trench was founded around the estates, similar to the one in alternative 1, but this one ended on the street Hovås Rönnbärsgång instead. This street was lowered to simulate raised curb stones, and conveyed the water out to Årekärrsvägen where the water could flow into a large pond. This pond was similar to the one in alternative 2, but was connected to an additional bigger and deeper pond. This alternative also had a pipe under the noise barrier to create a possibility of using also the trench on the other side as flood storage.

The investment cost was estimated to 1 330 000 SEK and the yearly maintenance 10 000 SEK. An overview of the measure can be viewed in Appendix 6.

5.4.5 Alternative 5

As a spin-off a fifth alternative was added in a later stage, initiated mainly by a curiosity. The idea was born from the notation that a 500 millimeter pipe would have about 70% higher capacity than the current 400 millimeter pipe under Hovås Mullbärsgång. This increment of the dimension leads to about 25% higher material cost which with the thumb rule that the material cost is 20% of the building cost leads to an increment of the investment of 5%.

This alternative was in fact not an alternative but a mind experiment to investigate the impact on flooding consequences if increased pipe dimensions would have been chosen when the area first was developed. Ergo the investment cost for this alternative was not defined as the cost for rebuilding the network but instead what it would have cost extra to from the beginning choose an increased pipe dimension. This additional cost was estimated to 170 000 SEK, and the yearly maintenance was estimated to be increased by 2 000 SEK. An overview of the measure can be viewed in Appendix 7.

In this alternative the whole main pipe from the stream intake at Hovås Mullbärsgång to the trench at the west side of the road was changed in the model. 340 meters of 400 millimeter pipe was changed into 500 millimeter, 90 meter was changed from 600 millimeter to 700 millimeter and 50 meters was changed from 600 millimeter to 800 millimeter.

As this alternative was composed as a mind experiment the result is not presented under the result chapter but instead in the discussion chapter, see Section 7.4.

6 Case study results

In total 24 simulations were made; for each alternative plus the “zero alternative” (the current situation with no measures implemented) one simulation was made for each return period of 100, 50, 20 and 5 years. The calculated maximum flood extent for the 100 year rain event is shown in Figure 6-1 and also in larger scale in Appendix 8. In Appendix 3 there are also maps showing maximum flood extent for the zero alternative for all four return periods.

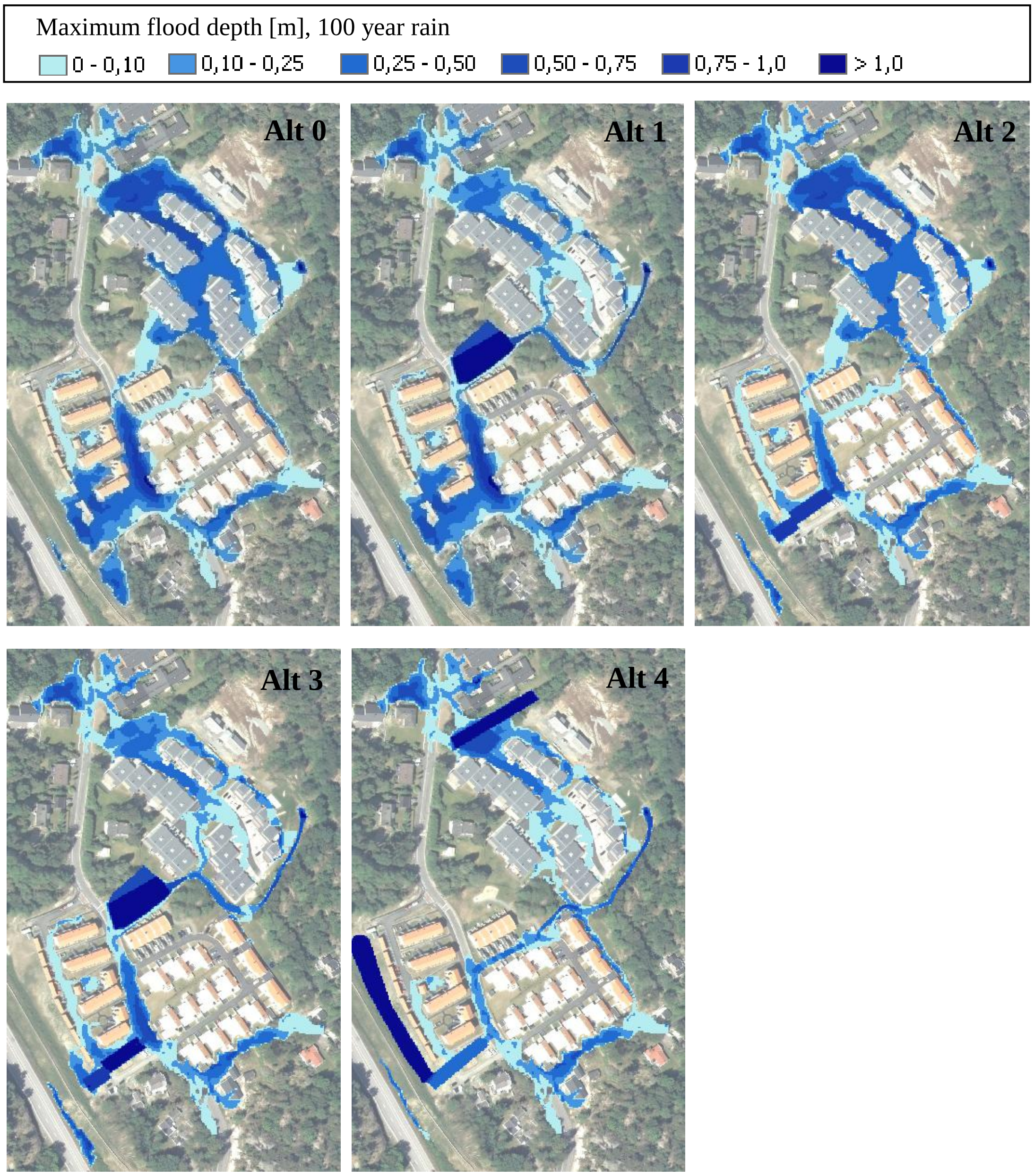

Figure 6-1. Flood depths for a 100 year rain event. Note the larger depths where pond has been implemented. For larger figures, see Appendix 8.

6.1 Assessment method 1: Key figures

The key figures of maximum flooded street area and maximum flooded buildings were calculated by GIS analyses of the maximum flood extent. The results are shown in Figure 6-2 and Figure 6-3. Also the relative key figures compared to the zero alternative were calculated and is shown in Figure 6-4.

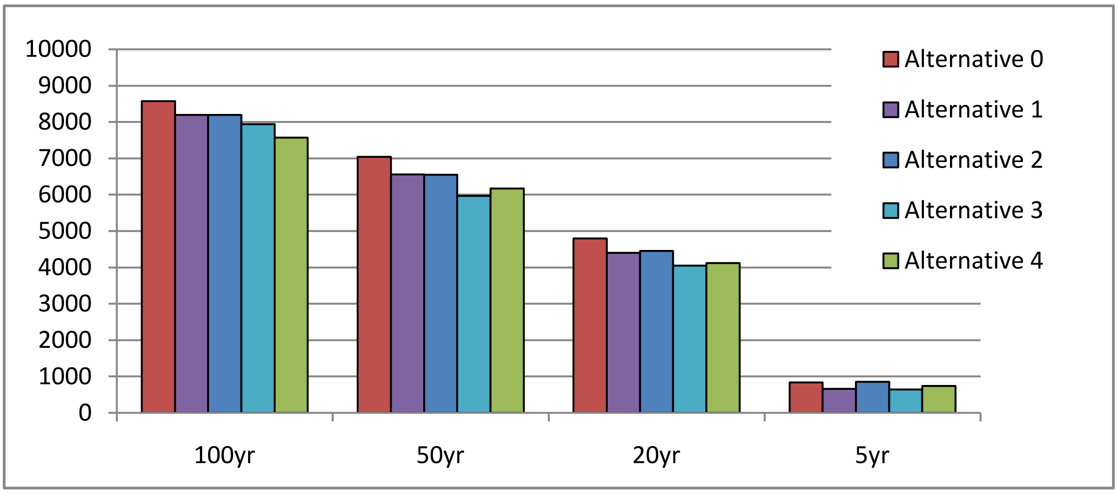

Figure 6-2. Flooded street area (m^2) for the alternatives, for return periods 100, 50, 20 and 5 years.

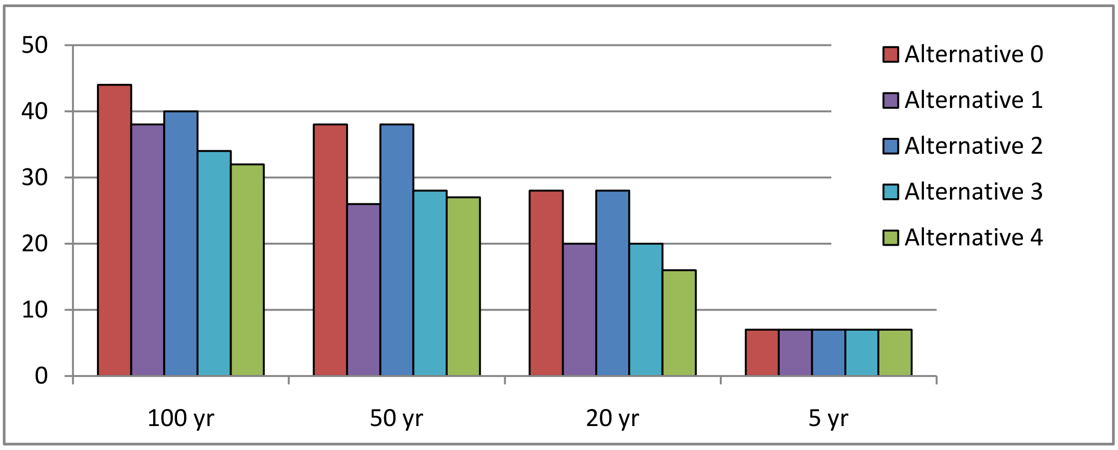

Figure 6-3. Flooded buildings for the alternatives, for return periods 100, 50, 20 and 5 years.

Best performance according to the key figures has alternative 4 but alternative 1 and 3 is not far behind. Alternative 2 has a similar mitigation effect on the flooded street area as alternative 1 but much less mitigation effect on the number of flooded buildings.

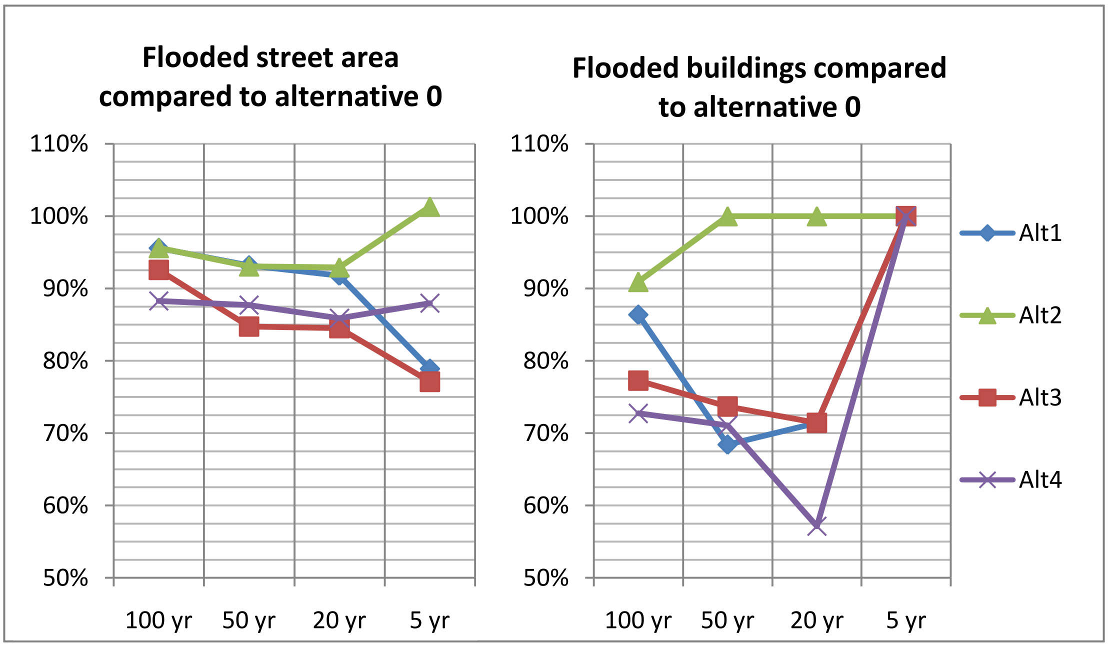

Figure 6-4. Relative of key figures; measures results compared to the zero alternative.

The key figures indicated that seven buildings become flooded for the rain with five year return period for all the alternatives. This was surprising as no reports of the stormwater system having inadequate function during normal conditions have been heard.

6.2 Assessment method 2: Cost-benefit analysis

The cost-benefit analyses were performed by DHI Flood Toolbox by using damage tables and analyzing the maximum flood extents. The resulting total damage costs from the different scenarios (alternatives and return periods) are shown in Table 6-1 and the damage probability curves in Figure 6-5. The resulting cost-benefit analysis is shown in Table 6-2.

Table 6-1. Damage cost (SEK) for each simulated scenario.

Scenario	100 yr	50 yr	20 yr	5 yr
Alternative 0	3 355 304	2 436 025	1 416 554	111 306
Alternative 1	2 896 425	1 839 235	1 047 393	101 220
Alternative 2	3 011 827	2 305 595	1 344 798	116 806
Alternative 3	2 603 291	1 695 300	978 443	100 125
Alternative 4	2 463 633	1 819 353	987 442	104 545

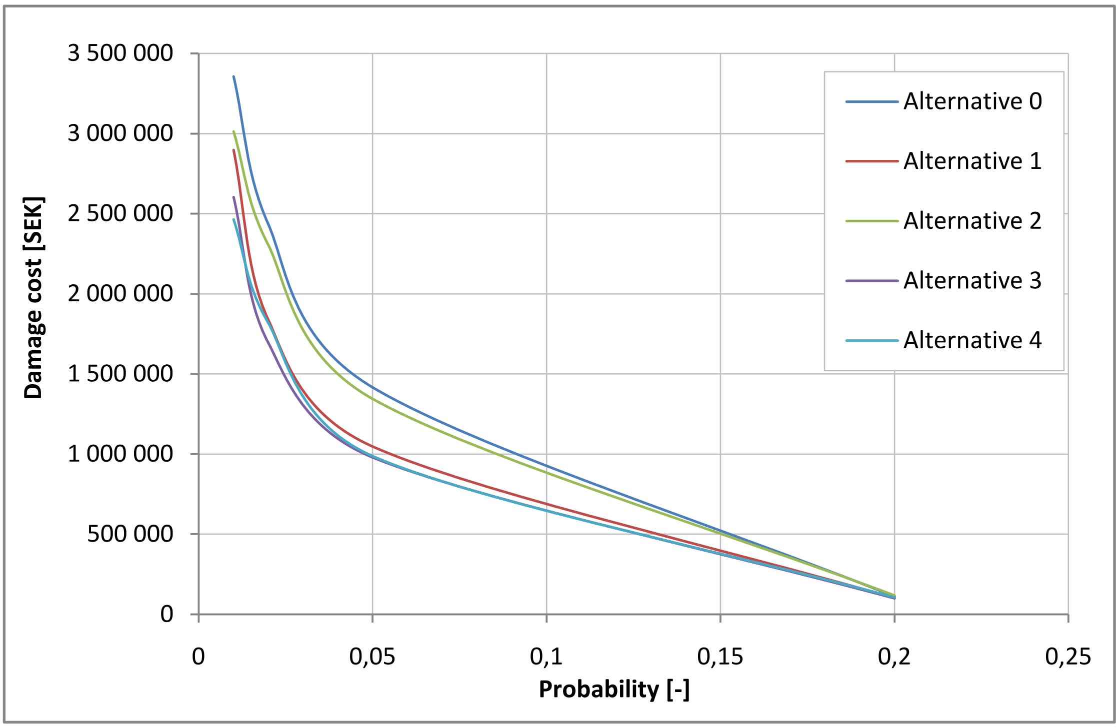

Figure 6-5. Cost-probability diagram of the zero alternative and the four measure alternatives.

Table 6-2. Costs and benefits for the four measure alternatives.

Scenario	AAD	Benefit	Costs	Balance	NBCR
Alternative 0	201 335				
Alternative 1	153 124	48 211	11 200	37 011	***3,3***
Alternative 2	190 963	10 371	8 400	1 971	***0,2***
Alternative 3	142 492	58 843	19 600	39 243	***2,0***
Alternative 4	145 416	55 919	36 640	19 279	***0,5***

As seen in Table 6-2, the CBA assessment states that all of the alternatives would be profitable investments since all NBCRs are positive, and that the most profitable would be alternative 1 with a NCBR above three. Least profitable would alternative 2 with a NCBR just above zero. Alternative 3, which was the combination of the alternative 1 and 2 was the second most profitable investment with a NCBR of two.

7 Evaluation and discussion

With a completed case study the work procedure could be evaluated and the used methods discussed. The evaluation and discussion has been divided into five categories. Section 7.1 handles the proactive flood methodology, Section 7.2 the tested assessment methods and Section 7.3 the measures themselves. The mind experiment of alternative 5 is handled separate in Section 7.4 and the chapter ends with Section 7.5 on some general insights on flood management that has been observed through the case study.

7.1 The proactive flood methodology

One of the questions specified in Section 1.2 was the resource need for the flood study. It can be concluded that the main resource used was time. Building and calibrating the model in the case study took about 180 hours for a relative inexperienced engineer. About 40 hours of these were devoted to solve different problems in the software, mainly the instability in the 2D simulations. The model was not completely calibrated as time was running short because of the encountered problems and moreover a fully calibrated model was not considered necessary for the purpose of the thesis.

Building the network model took some time as the whole stormwater system was not available in databases but parts had to be inserted manually. The network for the joint property of Hovås Mullbärsgång and around 60 gully pots over the whole area had to be inserted manually so the model would be useable in flood analyses. The other two joint properties were simplified and pipe locations assumed. These manual insertions of model elements were time consuming parts of the model building.

If a model for a real flood analysis project shall be built an important consideration has to be made. If the model is only going to be used for flood analysis some simplifications can be made that speeds up the building process but not affect the simulations significantly. However, when the computer model is built, it can be used for other purposes, for instance performance analyses during normal conditions, and therefore it can be a good investment to build it correctly and according to reality. If a network model already exists for an area of interest, a coupled model can be set up fairly easy. Setting up the network model is by far the most demanding task in the model building work and before starting this task it must be considered to what accuracy the model should be built and in which tasks the model will be used.

To set up the simulations took about 20 hours and the actual simulations took about 50 hours and were ran during night for saving time. The assessment of the simulations took about 4 hours for the key figure assessment method and about the same for the cost benefit analysis assessment method. This time refers only to the assessments itself in terms of calculating the key figures and the NCBRs and not the time it took to set up the assessments. However, if these methods were to be standardized, time for set up these assessment methods should not be necessary for the assessments of the results from a singular flood study.

The last question specified in Section 1.2 was if it would be feasible to study a larger area. As the performed case study only covers one fairly small area this question has no clear answer, but the question could still be discussed. Basically there is no real limitation in the methodology that restrains the size of the studied area. Instead there is a question of time available for the flood study as a bigger model is harder to manage. In the case study the studied area was decreased as part of it had a catchment that was tricky to model and with a bigger model this kind of challenges would probably occur more and then has to be dealt with. Also, with a larger area the risk of network not exist in databases increases and thereby also the risk for time consuming manual editing increases. However, for some areas models may already exists or is partly built and the situation then becomes different.

The author believes that a larger area would be feasible but it is hard to determine how much larger that are feasible as it depends on the conditions for the specific area. If very large areas are wanted to be analyzed, large districts of or even whole cities, it would most probably be more effective to start with GIS analysis or a pure 2D simulation as a initially flood risk analysis to identify risk areas, as the examples in Section 2.3, instead of setting up a large coupled 1D-2D model. By using these screening methods flood prone areas can then be identified and be further analyzed with 1D-2D modelling.

7.2 The assessment methods

The purpose of the assessment methods was to simplify the interpretation of the flood maps and provide decision makers with a tool that can provide a measurement on the impact of a flood mitigation measure. For comparison two different assessment methods were chosen, one based on key figures and one based on CBA. Both methods have been proven to be easy to use regarding the workflow of receiving an assessment result from a simulated flood result and thus fulfills part of the stated criteria. However, the methods requires different amount of work for set up. During the work with the case study a flaw in the technique for the interpretation of the flood maps was found, which is described further in Section 7.2.1.

The CBA assessment methods ease the task for a decision maker as it provides a figure that immediately tells which option is the “best”. However, every decision maker must remember that CBA is only a tool and should not be used as an absolute truth. Above all the CBA is based on estimations on costs and benefits which in the case study were quite uncertain. To clarify this and to illustrate how the ingoing estimations influence the results, a sensitivity analyses was realized and is shown in Section 7.2.2. Last in Section 7.2.3 the methods usability’s are discussed from experience gained in the performed case study.

7.2.1 Technique for interpretation of flood result maps

One of the key techniques in both assessment methods is how they interpret the flood maps. Both the assessment methods use the same input in form of a land use map and maximum flood extent maps. The overlap between the land use categories and the flood depth is then calculated in GIS analyses and used in the assessment methods. However, here is a potential source of error. It was identified that the land use maps and the DEM did not always synchronize, see example in Figure 7-1. This means that even if water is gathered at the foot of an elevation curve that corresponds to a building and in reality would not cause a flood, the GIS analyses would still interpret this as a flooded building. In this case the assessment results would be an overestimation of reality. Further on, in the DEM the buildings looks like rounded hills. If the DEM would have been a perfect picture of reality the buildings instead would have been blocks in the DEM and surface water would then flow around the building in the model, see Figure 7-2. Thus, if the DEM and land use layer in this case would have synchronized perfectly, no flood depths would ever been interpreted as a flooded building as the flood maps never would have cut in over the buildings in the land use maps so long as not the whole building is under water. In this case the assessment results would be an underestimation of reality.

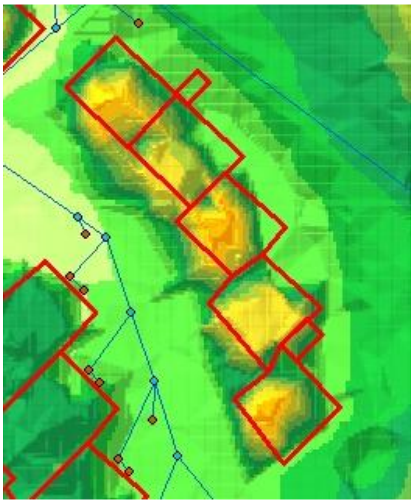

Figure 7-1. An example on the DEM and the land use layer not synchronizing.

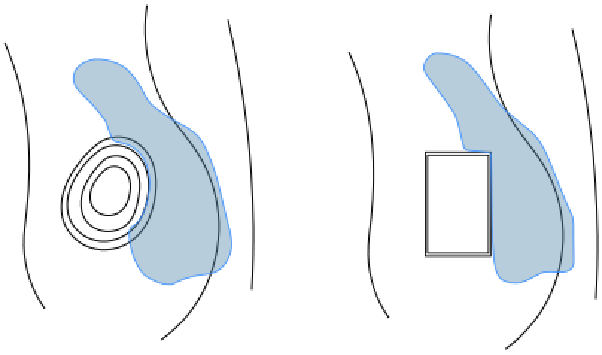

Figure 7-2. Schematic figure on surface water spread on DEM with different accuracy. For same conditions, the model to the left would be interpreted as flooded building whilst the model to the right would not.

Neither in the model nor in the assessment methods were the buildings elevation levels included. This means that a building with basement was treated the same way as a building that was founded above ground level which creates an uncertainty in the damage assessment results. In the case study most of the troubled estates was founded with the ground floor at the same level as the surrounding ground level, which means that error in the case study is not significant. Instead, for the conditions in the case study is the used five centimeters limit is found reasonable. But if areas with other conditions were to be studied this issue can be important to include in the assessment process.

7.2.2 Sensitivity analysis of the CBA assessments

Unfortunately there was not time within this thesis to perform a proper evaluation of all generalized values in the damage table. A proper evaluation would involve collecting and handling large amounts of historical flood events as well as take advantage of knowledge and experiences of similar assessments. In order to evaluate how the ingoing parameters impact on the CBA results and to provide some guidance for future studies sensitivity analysis were realized.

The first sensitivity analysis was to remove the cost for green areas and forest from the damage table. The hypothesis that these damage cost had a small contribution to the total damage cost turned out to be correct as the change had a negligible impact on the CBA results. Hence, with the generalized costs used in the case study, the total damage cost for green areas and forest is small compared to the damage cost for buildings and streets. The sensitivity analysis continued with also decrease the generalized costs for streets as this could be assumed to be chosen to high in the first place. For this a new damage table with generalized costs was created, shown in Table 7-1.

Table 7-1. Damage table with decreased damage cost for streets, green areas and forest.

Flood depth [m]	Costs [SEK/m^2]			
	Buildings	Streets	Green areas	Forest
0-0,05	0	0	0	0
0,05-0,10	3400	0	0	0
0,10-0,20	3400	0	0	0
0,20-0,50	4400	20	0	0
0,50-1,00	7000	50	0	0
>1	7000	100	0	0

The simulated flood results showed a more severe flood for the five years rain event than was expected since the system is designed for a ten year scenario and should be able to handle the five year event better. The severe results were therefore believed to be partly the result of the model not being fully calibrated and validated. To compensate for this the results from the CBA were modified by decreasing the total damage cost with 50% for the 20 year scenario and with 80% for the 5 year scenario, which consequently lead to lowered AAD values.

Also the impact on increased investment cost was investigated. The results from the CBA were modified by increasing the investment costs 50% (maintenance costs were remained the same) and calculate new NCBRs. This modification was made in combination with decreased damage cost for streets, green areas and forest as well as in combination with the decreased total damage for the 20 and 5 year scenarios.

The CBA assessments were calculated with different combinations of the above discussed changes. Table 7-2 shows the results from the use of the alternative damage table (Table 7-1) and Table 7-3 shows the results of using the same damage table with an increased investment cost. Table 7-4 shows the results from the use of the original damage table (Table 4-1) with decreased evaluated damage costs and Table 7-5 shows the results from the use of the original damage table with decreased evaluated damage costs and also an increased investment cost.

Table 7-2. CBA results with decreased damage cost for streets, green areas and forest.

Scenario	AAD	Benefit	Costs	Balance	NCBR
Alternative 0	89 142				
Alternative 1	48 735	40 406	11 200	29 206	***2,6***
Alternative 2	87 100	2 041	8 400	-6 359	***-0,8***
Alternative 3	48 283	40 859	19 600	21 259	***1,1***
Alternative 4	49 108	40 034	36 640	3 394	***0,1***

Table 7-3. CBA results with decreased damage cost for streets, green areas and forest and 50% increased investment cost.

Scenario	AAD	Benefit	Costs	Balance	NCBR
Alternative 0	89 142		0		
Alternative 1	48 735	40 406	14 300	26 106	***1,8***
Alternative 2	87 100	2 041	10 100	-8 059	***-0,8***
Alternative 3	48 283	40 859	24 400	16 459	***0,7***
Alternative 4	49 108	40 034	49 960	-9 926	***-0,2***

Table 7-4. CBA results with 20 year damage decreased 50% and 5 year damage decreased 80%.

	AAD	Benefit	Costs	Balance	NCBR
Alternative 0	154 836				
Alternative 1	118 371	36 464	11 200	25 264	***2,3***
Alternative 2	145 731	9 105	8 400	705	***0,1***
Alternative 3	109 308	45 528	19 600	25 928	***1,3***
Alternative 4	110 525	44 311	36 640	7 671	***0,2***

Table 7-5. CBA results with 20 year damage decreased 50%, 5 year damage decreased 80% and investment costs increased 50%.

	AAD	Benefit	Costs	Balance	NCBR
Alternative 0	154 836				
Alternative 1	118 371	36 464	14 300	22 164	***1,5***
Alternative 2	145 731	9 105	10 100	-995	***-0,1***
Alternative 3	109 308	45 528	24 400	21 128	***0,9***
Alternative 4	110 525	44 311	49 960	-5 649	***-0,1***

The sensitivity analyses shows that alternative 1 and 3 is profitable during all parameter changes with alternative 1 always more profitable than profitable than alternative 3. This due to alternative 2 already established lesser mitigation effect. Alternative 2 and 3 receives negative NCBRs for some of the parameter sets which make these alternatives to be uncertain investments.

7.2.3 Usability of assessment results

As mentioned are the two methods, in terms of work flow, both easy and quick to apply on simulated flood result. However, and unfortunately, this does not mean that the methods give an immediate answer on which option that would be “the best”.

If the problem with the interpretation of the flood maps would be solved and a technique is found that reliably answer the question whether a single building become flooded or not, then the key figure assessment would not only be quick and easy but also trustworthy to the same extent as the flood model. It would then be a very handy tool and a good aid for comparison of measure alternatives. But the decision maker still needs to do further assessments as only the change in two key figure is not enough to make a decision. Even if a full CBA would not be used, economy still needs to be included in the decision process.

The CBA assessment method is also quick to perform, but it demands much more work beforehand to establish the damage tables. If the generalized damage tables would be established and the interpretation technique issue solved, the CBA assessment method would not only be quick but also reliable and demand very little additional assessment. This is what makes a CBA a strong tool in general; with all benefits and costs evaluated and weighted, the final NCBR can be used to rank choices according to their profitability. However, to establish a reliable damage table with generalized costs that enable evaluation of all costs and benefits is a verbose and not trivial task. Without a reliable damage table the CBA assessment results become uncertain and can also instill a false security of the decision maker since the result looks so neat. Because of this, CBA assessments should always be critically reviewed.

It is unclear how big the effort would be to establish a reliable damage table and how valid it would be for different kind of areas. Same generalized costs can probably not be used for buildings in suburban areas as for city centers. The natural question that then arises is if it would be worth the effort to establish a reliable damage table. Flood Toolbox which was used for the CBA assessment, is developed for regional flood analyses and it is because of that adapted for more generalized costs and coarser resolution of land use maps.

From the work made in the case study, the author’s opinion is that for projects of size similar to the case study, it would not be worth to realize a full CBA. If the issue of interpret flooded buildings was solved it would probably be more efficient, and enough, to use the key figure of decreased number of flooded building and in an economically evaluation use this together with a generalized cost for one flooded building as benefit. It can be assumed that this would be sufficient in suburban areas were damage to buildings form the biggest damage cost. Also to only evaluate the damage cost for buildings and no other costs would in a societal perspective be an assessment on the safe side.

The case study was done on a domestic area. If an area with for instance commercial and industrial components would be studied only damage costs to building would not be a fair evaluation as a flood also with lead to disturbed productivity and lost income. When evaluating those kinds of areas, this need to be taken into consideration and a CBA assessment could then be a valuable tool to use.

7.3 The measure alternatives, 1 to 4

Based on the four mitigation measures that has been investigated some observations can be made. It can be concluded that the idea of having a proactive approach and plan surface water flow paths and storage areas is a working strategy to mitigate flood consequences. As it is written in Section 2.4, Houston et al (2011) means that also small changes can mitigate flooding which can be confirmed when analyzing the simulated results in this study. The alternative 1 gives not much less mitigation effect than alternative 4, but with much less construction effort and investment cost.

At the same time was alternative 2 the cheapest and easiest option that also gave the least mitigation effect. The storage capacity was not much smaller than in alternative 1 but the lesser effect was instead due to the location of the pond. The pond was located in order to decrease the flood depth on the nearby road, which it also did, but this had small effect on the flood in the most suffered area, the street Hovås Mullbärsgång, and had almost none mitigation effect on flooded buildings.

Alternative 4 was an extreme option with one very large pond and one smaller, similar to those in alternative 1 and 2. In fact the large pond in alternative 4 was not even full for the 100 year scenario but could store even more surface water. At the same time the other north pond was filled and part of Hovås Mullbärsgång still flooded.

These observations together indicate that measures, big or small, have low efficiency if they are not carefully planned. It is important to establish an overall strategy and above all that surface water flow routes is planned so water actually ends up in the intended storage areas. When an area is heavy suffered from flooding, measures must be planned close to this area to facilitate mitigation of flood consequences. Alternative 2 indicates that it is inadequate to construct a flood storage downstream a problem area if not conveyance of surface water is planned as well. Due to the high rain intensity, only flood storage would not be enough since the network is not capable of carrying the high flows.

Further on, by studying the flood maps from the simulations of the alternatives it can be concluded that higher efficiency could have been reached with not much extra effort. The trenches and conveyance of surface water could have been further optimized and a higher mitigation could have been reached for more or less the same construction cost.

7.4 The mind experiment of alternative 5

Alternative 5 was a spin-off to the original purpose of the thesis but is still highly connected to flood management. In the alternative the pipe dimensions were increased for a part of the case study area and the result was evaluated with the same methods as the other alternatives. Flood maps are shown in Figure 7-3 and the results from the key figure assessment in Figure 7-4.

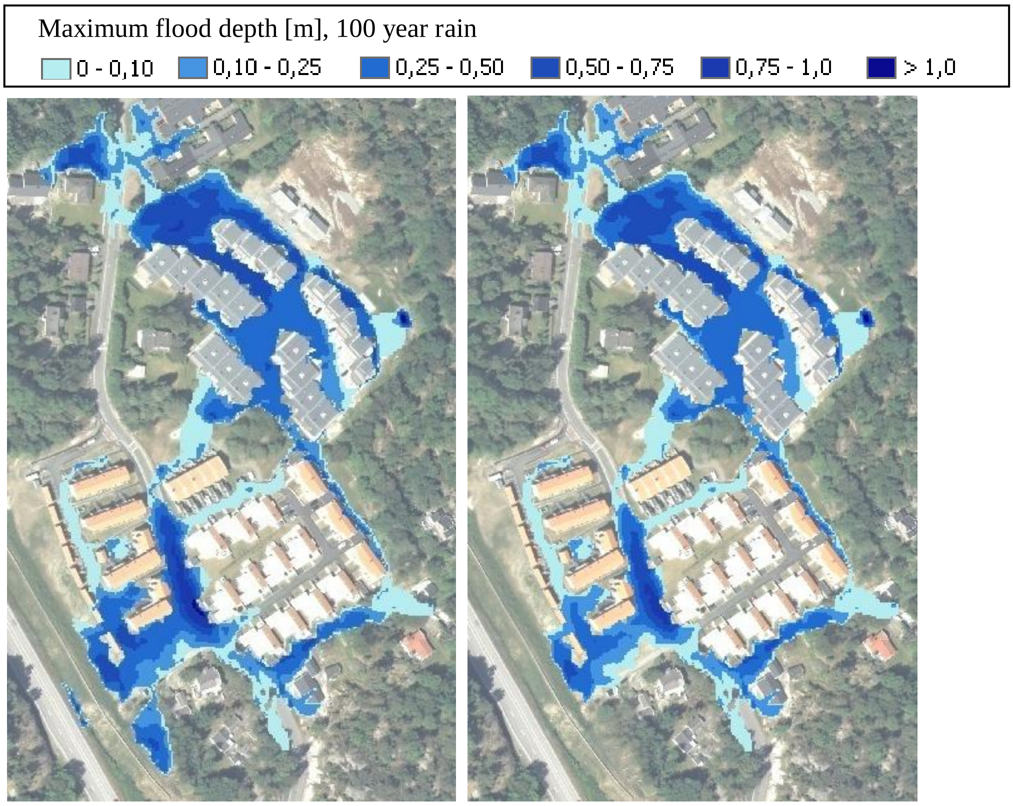

Figure 7-3. Flood depths for a 100 year event. Left, alternative 0. Right, alternative 5.

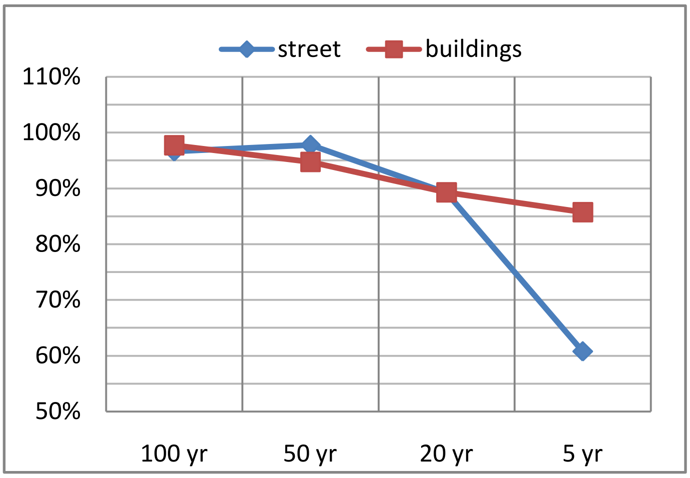

Figure 7-4. Relative key figures for alternative 5 compared to alternative 0.

The same sensitivity analysis that was performed for the CBA of alternative 1 to 4 was also performed for the mind experiment. The result is shown in Table 7-6 to Table 7-10.

Table 7-6. CBA results with original parameters.

Scenario	AAD	Benefit	Costs	Balance	NBCR
Alternative 0	201 335				
Alternative 5	176 680	24 655	5 400	19 255	***4,6***

Table 7-7. CBA results with decreased damage cost for streets.

Scenario	AAD	Benefit	Costs	Balance	NCBR
Alternative 0	89 142				
Alternative 5	79 599	9 543	5 400	4 143	***0,8***

Table 7-8. CBA results with 20 year damage decreased 50% and 5 year damage decreased 80%.

	AAD	Benefit	Costs	Balance	NBCR
Alternative 0	154 836				
Alternative 5	136 039	18 797	5 400	13 397	***2,5***

Table 7-9. CBA results with 20 year damage decreased 50%, 5 year damage decreased 80% and investment costs increased 50%.

	AAD	Benefit	Costs	Balance	NCBR
Alternative 0	154 836				
Alternative 5	136 039	18 797	7 100	11 697	***1,6***

Table 7-10. CBA results with decreased damage cost for streets and 50% increased investment cost.

Scenario	AAD	Benefit	Costs	Balance	NCBR
Alternative 0	89 142		0		
Alternative 5	79 599	9 543	7 100	4 143	***0,3***

The key figures assessment shows that an increased pipe dimension mainly has a mitigation effect on the lower return periods which is natural since the capacity only was increased in the pipe network and no measures was taken against surface water flows. In other words has the point of when flooding occurs been moved forward but when flooding do occurs it evolves in a very similar manner as before.

There were, as already mentioned, lots of uncertainties in the CBA assessment but in all of the sensitivity analyses, the NCBR remained positive for this special alternative. Most notable is that it also stayed positive when the damage cost was decreased (and thereby the total benefit was decreased) for the five and twenty year scenarios where the increased pipe dimension had the highest impact. As the CBA had lots of uncertainties should conclusions be drawn carefully, but the tables above indicates that it would be profitable to choose larger pipe dimension than what is meeting the design requirements. It should be pointed out that this is an indication from a case study on a confined area. Also it should be pointed out that this is a mind experiment where the CBA had the increased investment cost on the original construction as cost input. To rebuild the pipe network afterwards would probably not reach the same results.

7.5 Flood management

Some notes on flood management can and should also be made after the performed case study in this thesis. First the author wants to point out that it is important to remember to mind about the eventual flood consequences in the planning process of new development areas. Design criteria often states minimum values and recommendations and should not always be adopted without critical review. In the studied case a stream had been diverted into a culvert which was designed after the recommended ten year return period. A bigger dimension on the pipe would not have hindered the flood from the rain storm in August 2011, but it would probably have done some mitigation effect on the flooding as shown by the mind experiment described in the section above. More important, little attention had probably been given the consequences of the case of the intake being overtopped.

The estates at Hovås Mullbärsgång have no socle but instead the entrances are in level with ground level and the slope from the street is very low. This is of course a very good building approach when it comes to accessibility, but in case of a flood this is could be catastrophic as water easily flows into buildings. To quickly evaluate what the difference would be with a higher socle, the flood limit in the key figure assessment was changed from five to thirty centimeters, meaning that the buildings would not be classified as flooded until thirty centimeters of water stands at the border of the building. A clear lowering of the number of flooded buildings in the whole area could be observed with this changed limit, see Figure 7-5.

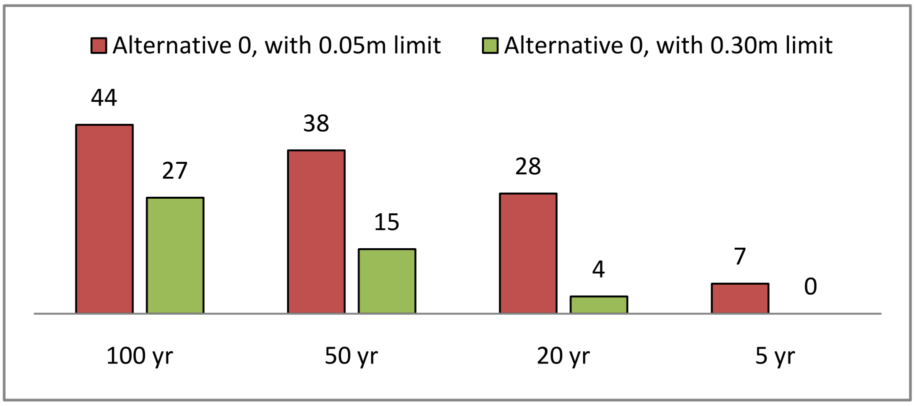

Figure 7-5. Number of flooded buildings for the 100 year event in the zero alternative with changed assessment criteria.

Traditionally, design criteria are defined according to probability, namely return period for rain events. This method has been used for a long time and the intention is to design with an acceptable damage risk. However, risk consists of both probability and consequence and to define a threshold according to only probability is in reality not sufficient but it is a concept that works for most cases and that simplifies the design process. But when consequences are high even for low probabilities, more stringent design is demanded. This is illustrated in Figure 7-6. This is pointed out since this is something that can be learned from the case study. A stream was led into a pipe network with the intake situated on a hill above estates with low flood resistance. The consequences of the intake being overtopped were with other words high.

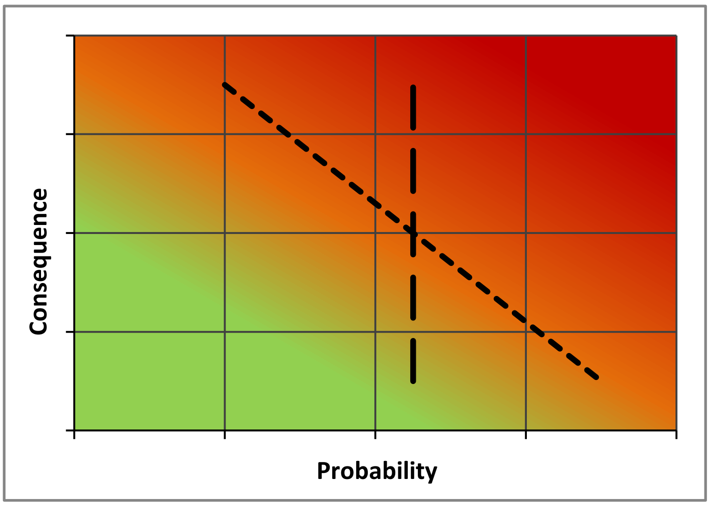

Figure 7-6. Risk chart where the risk is low in the lower left corner and high in the top right corner. Criteria according to probability (return period) shown with long-dash line and acceptable risk shown with short-dash line.

Last the author want to point out that flood management is a subject where it is hard to gain own experience as an engineer. Because of the design return period of proactive flood mitigation measures can be far longer than the stormwater networks design return period, perhaps even 100 years and more, many years can pass between a flood mitigation measure being constructed and actually being used. The confirmation on whether a measure was constructed right or wrong thus can come long after it was designed. Therefore an exchange of experiences and knowledge in flood management between municipalities, organizations and nations is important to create safe and secure urban areas.

8 Conclusions

- A proactive approach using models to simulate and evaluate surface water flows and to plan control measures that can mitigate flood consequences, is a by the author recommended methodology in flood risk areas.

- The assessment of flood mitigation measures is easiest done with key figures, but this technique needs to be further worked on for reliable use. If the flaw discussed is solved, the method can be a simple and useable tool in measure assessment and could also be used for simplified economical assessments. The author is recommending further development of the key figure assessment methods and to use this for projects and investigations similar to the case study.

- CBA assessment of flood mitigation measures demands much time resources to realize and is not efficient to use in smaller flood investigations. For projects covering larger areas and areas where the flood damage would be more complicated it can however be a valuable tool.

- An overall strategy is needed when aiming on mitigate flood consequences. To move a flood from suffered estates to ponds demands careful surface water planning over the whole suffered area which also means that different organisations and departments within the municipality needs to work together; Urban planning, park, water utility etc.

- When developing new living areas in confined spaces, the study indicates that it in long terms can be profitable to choose larger stormwater pipe dimensions than design criteria demands. However, confined areas should always be avoided to be built on. Better then to rebuild the area according to an overall surface flow strategy.

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Appendix 1 1D-2D simulation summary report

This appendix shows an example on a summary report produced by the modelling software. This example is for the current conditions (alternative 0) for a 100 year rain event.

MOUSE HD Computation Engine v2011 Release Version (12.0.0.6004)

MOUSE Pipe Flow Simulation --- Status Report ---Dynamic Wave

Index of summary

File Overview

Working dir :	C:\muh\modellering\	-
Sewer network data (UND) :	MAIN-OL-100-Base.MEX	2012-04-23 11:14:32
Hydrological data (HGF) :	MAIN-OL-100-Base.MEX	2012-04-23 11:14:32
Additional parameters file (ADP) :	-	-
Dry weather flow data (DWF) :	MAIN-OL-100-Base.MEX	2012-04-23 11:14:32
Repetitive profile data (RPF) :	-	-
Runoff Hydrographs (CRF) :	MAIN-RO-100-Base.CRF	2012-04-23 11:12:16
Hotstart file (PRF) :	-	-
Result File (PRF) :	MAIN-OL-100-Base.PRF	2012-04-23 11:14:46
Reduced result file (PRF) :	-	-

Time Overview

Simulation start date :	2000-01-01 00:01:00	Calculation started :	2012-04-23 11:14:45
Simulation end date :	2000-01-01 08:00:00	Calculation ended :	2012-04-23 13:35:22
Save time step [hh:mm:ss] :	0:01:00	Calculation time [hh:mm:ss] :	2:20:37

Report is available in full color at http://publications.lib.chalmers.se/

Maximum time step [sec] :	0,5	Hotstart start date :	-
Minimum time step [sec] :	0,5		

Input Summary

Number of Manholes:	245
Number of Basins:	0
Number of Outlets:	2
Number of Storage Nodes:	0
Number of Circular Pipes:	235
Number of Rectangular pipes:	0
Number of CRS defined pipes:	6
Number of Pumps:	0
Number of Controlled Pumps:	0
Number of Weirs/Orifices:	0
Number of Controlled Weirs/Gates:	0
Number of Valves:	0
Number of Controlled Valves:	0

Nodes

Min Invert Level	Node_42	0,00 m
Max Invert Level	Node_121	63,24 m
Min Ground Level	Node_289	35,74 m
Max Ground Level	Node_55	66,31 m
Min X Coordinate	Node_35	1,473E05 m
Max X Coordinate	Node_131	1,4789E05 m
Min Y Coordinate	Node_289	6,3858E06 m
Max Y Coordinate	Node_58	6,3872E06 m
Total Manhole Volume		422,6 m3
Total Basin Volume		0,0 m3

Links

Total Circular Volume	525,2 m3
Total CRS Volume	897,9 m3
Total Length	5844,30 m

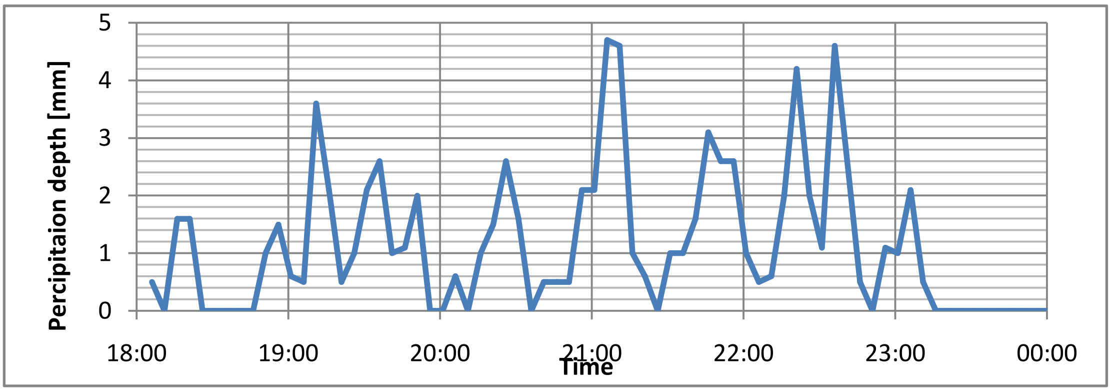
Simulation Result Summary

Continuity Balance

1 :	**Start volume in Pipes, Manholes and Structures**			**1,1 m3**
2 :	**End volume in Pipes, Manholes and Structures**			**184,5 m3**
3 :	**Total inflow volume**			
	Specified inflows			
	Runoff :	6728,9 m3		
	Non-specified inflows			
	Inflow from 2D overland :	23406,3 m3		
		30135,2 m3	**-->**	**30135,2 m3**
4 :	**Total diverted volume**			
	Volume not possible to extract :	-17,4 m3		
	Operational, non-specified outflows			
	Outlets :	15141,5 m3		
	Flow to 2D overland :	22668,3 m3		
	Excluding runoff diverted directly to 2D overland :	-7825,8 m3		
		29966,6 m3	**-->**	**29966,6 m3**
5 :	**Water generated in empty parts of the system :**			**52,7 m3**
6 :	**Continuity Balance = (2-1) - (3-4+5) :**			**-37,9 m3**
	Continuity Balance max value :		0,0 m3	
	Continuity Balance min value :		-64,3 m3	

Appendix 2 Rain series

Below is shown the rain series used in the calibration, taken from a private gauge in Askim, about four kilometers from the case study area.

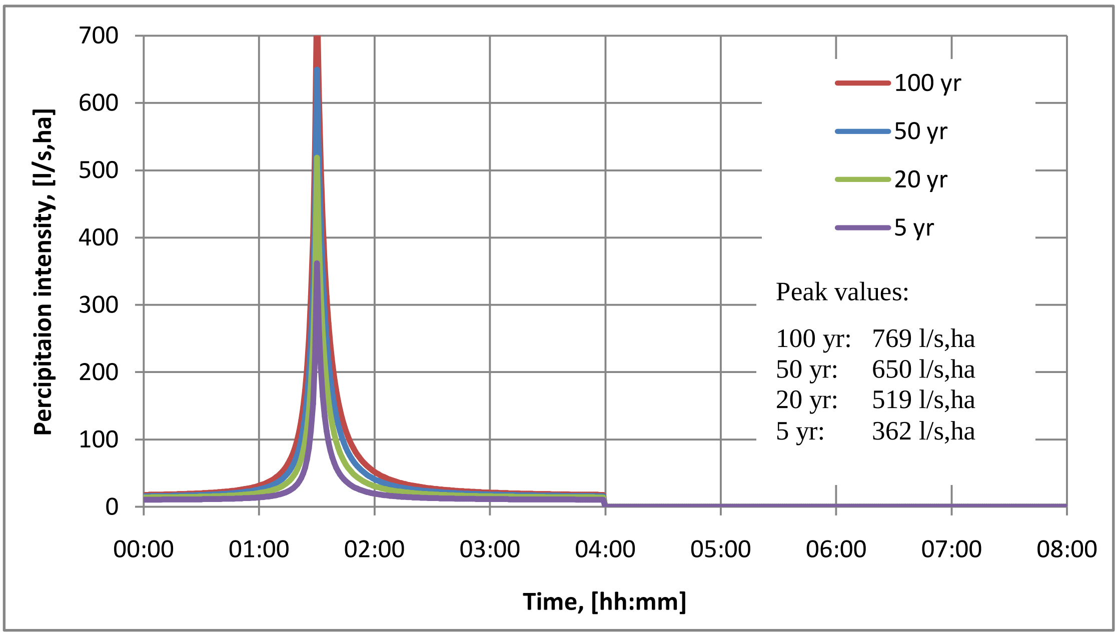


For the simulations of the measures, CDS (Chicago rain series) was used as is recommended by the Swedish water and wastewater association (Svenskt Vatten, 2011b). Below is shown the used equations for the rain intensity, the parameters used in the equations and a diagram over the intensity curves.

$$i_{before\ maximum} = \frac{a \times b}{\left(\frac{|t - rT|}{r} + b\right)^2} + c \qquad i_{after\ maximum} = \frac{a \times b}{\left(\frac{|t - rT|}{1 - r} + b\right)^2} + c$$

T	240 min
r	0,375
rT	90 min

Return period	a	b	c
100 yr	10169	13,5	15,8
50 yr	7874	12,4	14,5
20 yr	5563	11,0	13,2
5 yr	3303	9,4	10,3



Appendix 3 Flood depth maps, current conditions (alternative 0) for 5, 20, 50 and 100 year rains

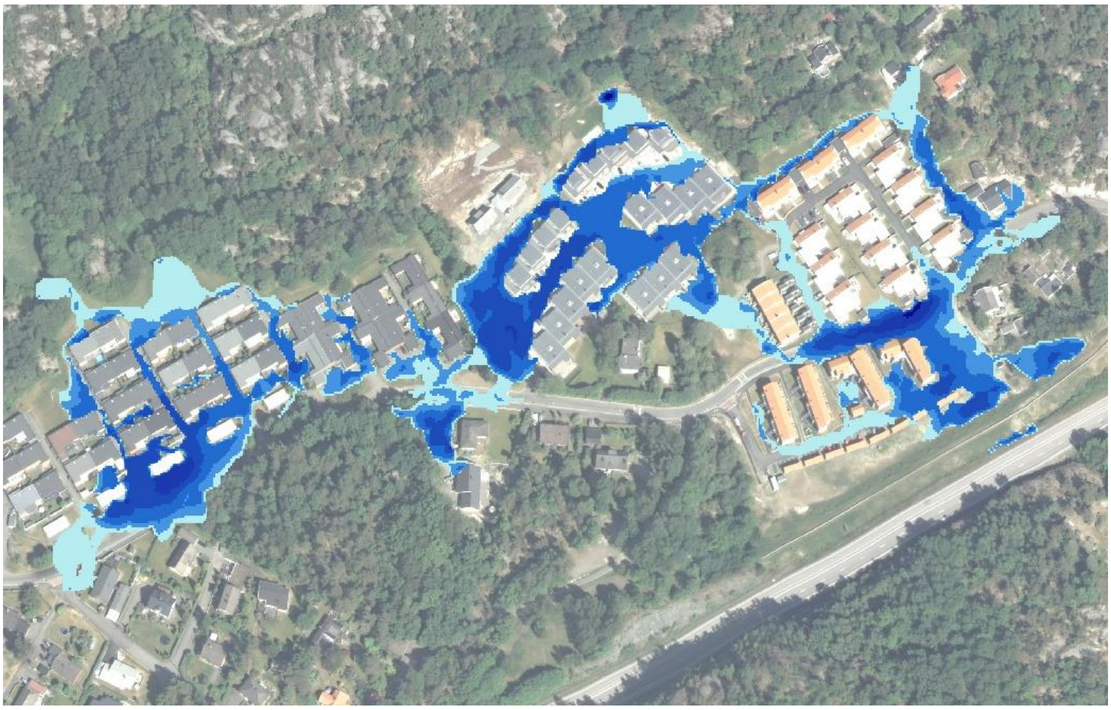
Maximum flood depth [m], 100 year rain

0 - 0,10 | 0,10 - 0,25 | 0,25 - 0,50 | 0,50 - 0,75 | 0,75 - 1,0 | > 1,0

5 year rain.

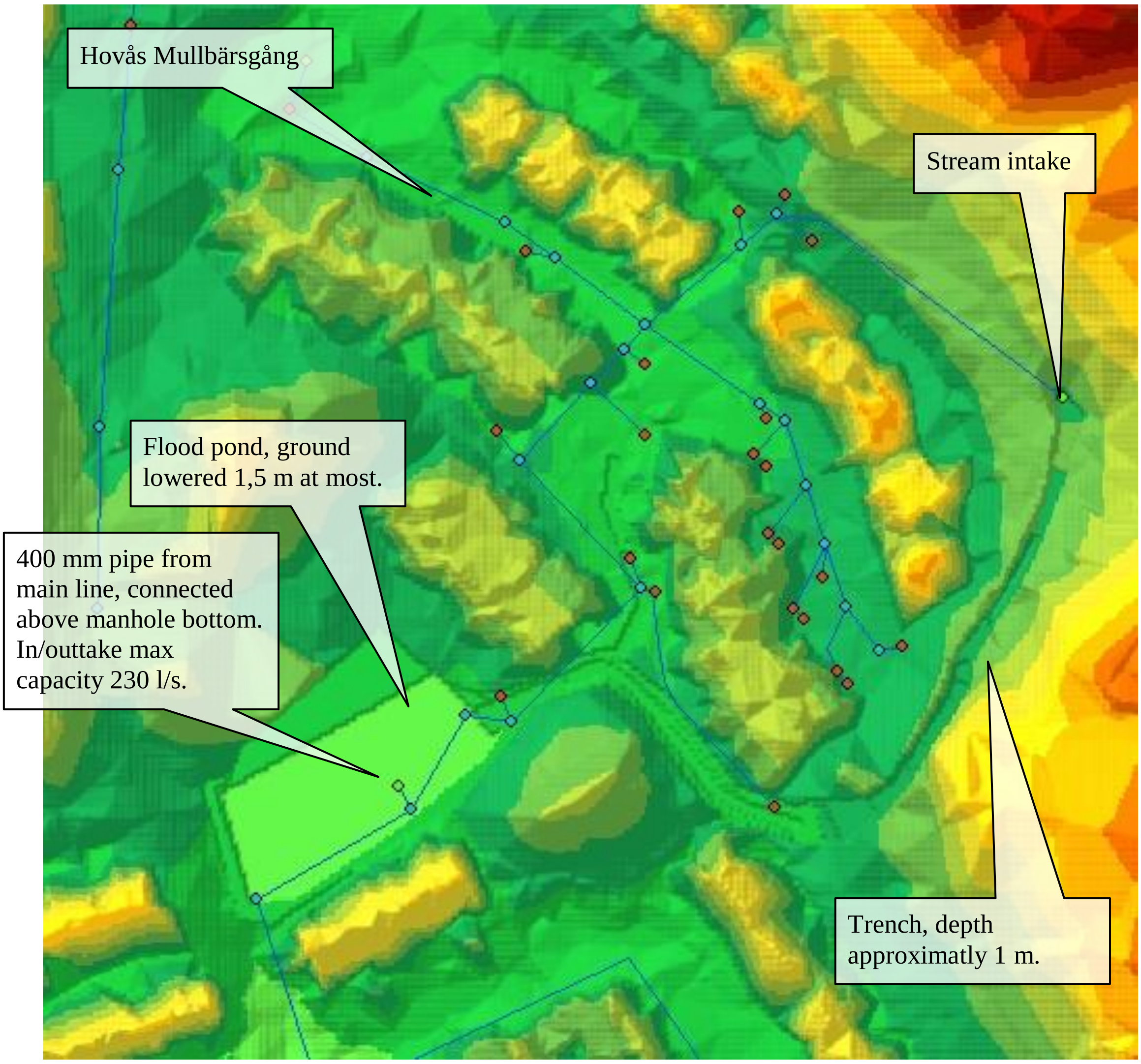
20 year rain.

50 year rain.

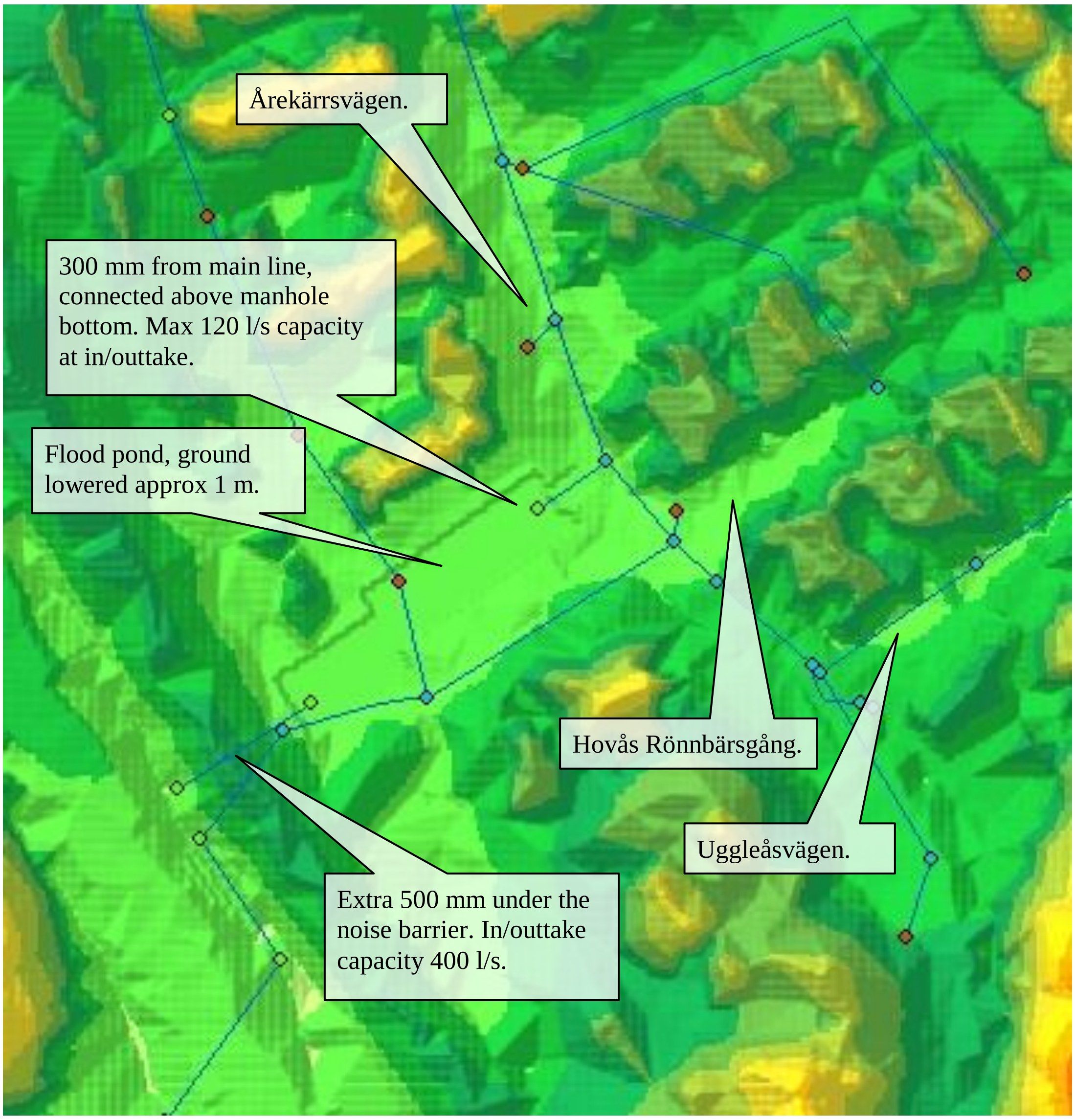


100 year rain.

Appendix 4 Alternative 1



Appendix 5 Alternative 2



Appendix 6 Alternative 4

Appendix 7 Alternative 5

Report is available in full color at http://publications.lib.chalmers.se/

Appendix 8 Flood depth maps, alternative 0-5, 100 year event

Maximum flood depth [m], 100 year rain

0 - 0,10 | 0,10 - 0,25 | 0,25 - 0,50 | 0,50 - 0,75 | 0,75 - 1,0 | > 1,0

Alternative 0.

Alternative 1.

Alternative 2.

Alternative 3.

Alternative 4.

Alternative 5.