Master's thesis

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Modelling of stormwater measures and performance evaluation of underground detention basins

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Abstract

Climate change, urbanization and an increased amount of paved surfaces lead to challenges to the existing sewer system. To be better prepared for such events, several municipalities in Norway has made their guidelines for stormwater management. As a result of these guidelines, several detention basins are being built every year. There is a lack of knowledge related to their performances on the capacity of the sewer system and CSO discharges. This is the main background of this master thesis, where the performance of detention basins and Low Impact Development (LID) controls were investigated for an extreme rainfall event in addition to an average year of rainfall data. An already calibrated model in MIKE URBAN, given by DHI, was used for the simulations of a catchment area of Fagerheimsbukta in Trondheim. The main objectives of this master thesis are to investigate the effects of stormwater management practices, in addition to an evaluation of today's guidelines for stormwater management in Trondheim municipality in relation to the performances of underground detention basins.

The model of Fagerheimsbukta was rebuilt with new sub-catchments concerning existing development. Approximately all catchments were designed for the implementation of stormwater practices, with the establishments of service pipes and manholes which were connected to the existing sewer system. Three scenarios were defined, concerning today's situation, and implementation of detention basins and bioretention cells. The stormwater practices were designed based on today's guidelines of Trondheim municipality and other recommendations from literature. Different degrees of implemented stormwater practices were simulated, concerning 50% and 100% establishment of stormwater controls based on the total amount of sub-catchments.

The implemented stormwater practices led to an overall decrease in the risk of flooding in addition to a reduction of CSO discharges compared with today's situation. However, the scenarios with implemented detention basins show an increase in the duration for the simulation of the extreme rainfall event, which leads to a minimal reduction in the accumulated volume of CSO discharges. Moreover, a larger CSO volume reduction was obtained by the simulations of the extreme event of 50% implemented detention basins in comparison to 100% implementation of basins, with a reduction of 5,2% and 4,4% for the respective scenarios. This is a result of a combination of a general decrease in the maximum CSO discharge and an increase of the duration of the event, which leads to small changes in the accumulated CSO volume. Furthermore, the scenarios including implemented bioretention cells led to a large decrease in CSO discharges. However, limitations of defining different soil layers with its respective parameters for the implementation of bioretention cells in MIKE URBAN affected the availability of the functions to replicate the physical reality, which is illustrated in the results of this thesis.

Performances of the detention basins during an extreme rainfall event were analysed, where a maximum water depth of 0,6 meters was obtained, corresponding to half of its capacity. No relation between the basins water level and the input of today's design criteria was found in this thesis. Further investigation of the detention basins performances on CSO reduction and its capacity utilization should be done in order to evaluate the design criteria of today's guidelines. Furthermore, an investigation of other methods for LID implementation in MIKE URBAN should be done to obtain a better reflection of reality.

Sammendrag

Effekten av klimaendringer, hyppigere ekstremnedbør og urbanisering, fører til utfordringer for dagens avløpsnett. For å være bedre forberedt for slike hendelser i fremtiden, har flere norske kommuner utarbeidet retningslinjer for overvannshåndtering. Som en følge av dette, bygges det flere fordrøyningsmagasiner hvert år i Norge. Det er mangel på kunnskap knyttet til disse magasiners effekt på kapasiteten til ledningsnettet, samt overløpsdrift. Dette er hovedgrunnlaget for denne masteroppgaven, hvor effekten av fordrøyningsmagasiner og lokale overvannsløsninger (LOD) er undersøkt for en ekstremhendelse og et gjennomsnittsår. En allerede kalibrert modell i MIKE URBAN, gitt av DHI, ble brukt til simulering av et avløpsfelt knyttet til Fagerheimsbukta i Trondheim. Hovedmålet med denne masteroppgaven er å undersøke effekten av ulike overvannsløsninger, i tillegg til en evaluering av dagens retningslinjer for overvannshåndtering i Trondheim kommune.

Modellen over Fagerheimsbukta ble ombygget med nye delfelt knyttet til eksisterende bebyggelse og infrastruktur. Omtrent samtlige delareal ble etablert med stikkledninger og kummer og videre tilkoblet eksisterende ledningsnett. Tre scenarier ble definert, basert på dagens situasjon, samt situasjoner med implementerte fordrøyningsmagasiner og regnbed. Overvannsløsningene ble utformet basert på dagens retningslinjer for Trondheim kommune, i tillegg til andre anbefalinger fra relevant litteratur. Ulike situasjoner med 50% og 100% implementeringsgrad av overvannsløsninger i forhold til totalt antall delfelt ble simulert.

Scenariene med implementerte overvannsløsninger førte til en generell reduksjon i flomrisiko, samt reduksjon i overløpsdrift sammenlignet med dagens situasjon. Simuleringen av ekstremhendelsen for fordrøyningsmagasiner viste en økning i varigheten av overløpsdriften, noe som førte til en begrenset reduksjon i overløpsvolum. Samtidig ble det under simuleringen av ekstremhendelsen oppnådd en større reduksjon i overløpsvolum for situasjonen av 50% fordrøyningsmagasiner sammenlignet med 100% implementering, med en reduksjon på 5,2% og 4,4% for de respektive scenariene. Dette er et resultat av en generell reduksjon i maksimum overløpsutslipp i kombinasjon med en økning i varigheten, noe som fører til en relativt liten endring i overløpsvolumet. Scenariene med implementerte regnbed førte til store reduksjoner i overløpsdrift. Disse resultatene er påvirket av begrensninger i modellen som førte til store forenklinger ved implementeringen av regnbed i MIKE URBAN. Dette påvirket dens evne til å illustrere virkeligheten, noe som gjenspeiles i resultatene for simuleringene av regnbed i denne masteroppgaven.

Effekten av fordrøyningsmagasiner under en ekstremhendelse ble analysert, hvor resultatene fra simuleringene viste en maksimum vanndybde lik 0,6 meter. Dette tilsvarer halvparten av maks kapasitet. Det ble ikke funnet noen sammenheng mellom dagens designkriterier og vannstanden i magasinene. Det anbefales videre undersøkelser av fordrøyningsmagasiners kapasitetsutnyttelse og evne til reduksjon i overløpsdrift for videre evaluering av dagens designkriterier og retningslinjer. I tillegg anbefales videre undersøkelser av funksjoner for implementering av regnbed i MIKE URBAN, for å kunne oppnå resultater som gjenspeiler virkeligheten på en mer tilfredsstillende måte.

Preface

This thesis is the final product of the course TVM4905 – Water Supply and Wastewater Systems, Master's Thesis. It is submitted to the Department of Hydraulic and Environmental Engineering at the Norwegian University of Science and Technology (NTNU). The topic of the thesis is modelling and evaluation of different stormwater practices for local stormwater management. This thesis is written in cooperation with the municipality of Trondheim, which has helped me with supervision and materials.

I would like to express my gratefulness to my supervisor Sveinung Sægrov. Sægrov has given me advice and feedback, which has been very helpful for my understanding and work with this topic. I would also like to thank my secondary supervisor, project leader Birgitte Gisvold Johannessen at Trondheim municipality for all the encouragement, inspiration and helpful advice. Thank you for all the interesting conversations and motivation during this study.

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List of Abbreviations

CSO	Combined Sewer Overflow
DHI	Danish Hydraulic Institute
GIS	Geographic Information System
IDF	Intensity-Duration-Frequency
LID	Low Impact Development
MIKE 1D	Modelling engine in MIKE URBAN
MOUSE	Model for Urban Sewers
NTNU	The Norwegian University of Science and Technology
PE	Person Equivalents
PVC	Polyvinyl chloride
RDI	Rainfall Dependent Infiltration
WWTP	Wastewater Treatment Plant

1 Introduction

1.1 Background

Climate change, combined with urbanization and an increased amount of impervious surfaces, has caused challenges all over the world. The changing climate is expected to lead to an increased amount of extreme precipitation, both in its frequency and intensity (Hirabayashi, et al., 2013). These changes may lead to capacity problems on the existing sewer system and higher discharges through combined sewer overflows (CSO). Consequently, the flood damage tends to increase, especially in dense areas with buildings including basements (Nilsen, et al., 2011). Another aspect of CSO is related to health risks, which may happen both in work related to the removal of the water and the cause of polluting the receiving waters. This is especially unfortunate and unpleasant if areas around the connected recipient are used for swimming (NOU, 2015). CSOs are significant contributors to pollutions, such as untreated human and industrial waste and toxic substances (EPA, 2011). The reaction rate of these substances increases with the increase of temperature (Ødegaard, 2014).

The summer of 2018 in Norway is for many remembered as hot and dry, which had an average temperature 3,1 Celsius degrees above normal. In combination to precipitation amounts of 74% below normal, this led to several problems related to drought and water quality, which affected the agriculture and the ecological state in water bodies (Norwegian Meteorological Institute, 2019). Moreover, the temperature in Norway has increased with one degree Celsius since 1900, and studies show that the amount of precipitation also tends to increase in the future (NOU, 2015). To deal with these problems, the implementation of stormwater management practices has become important. During the 1970s, driven by state regulations and local concerns, engineers began to focus on measures to control local flooding concerning new development projects (Dietz & Arnold, 2018).

Traditional measures for handling such challenges are focusing on disposing the water quickly through pipes and into central detention facilities. Some water utilities are now focusing on viewing stormwater as a resource to be used beneficially and returned to its natural pathways through different distributed controls. Such solutions are often known as low impact development (LID) and green solutions (WEF Press, 2012). LID practices include permeable pavements, green roofs, rain gardens, infiltration swales, bioretention areas and cluster development (Damodaram, et al., 2010). Yet, several studies have been done for the evaluation of CSO discharge and other environmental and economical benefits with the use of stormwater management. A study from China showed that LID designs are more effective in flood reduction during the heavier and shorter rainfall events. A combination of various LID techniques with conventional flood control is although recommended to control the entire spectrum of storm events because of the different performances with respect of peak intensity (Qin, et al., 2013). However, considering the most extreme storm event for the drainage design system would result in a system too expensive to build and operate.

Hence, designing systems which are technically feasible and economically viable with least compromises to people's lives and properties is essential for stormwater management (Bisht, et al., 2016).

Networks including detention basins is a traditional measure to retain and reduce total runoff volume and peak (Thomas et al., 2019). Such controls are today often implemented as a consequence of regulations adopted by local authorities. These regulations are rapidly growing in many countries (Petrucci et al., 2013), and is also a topic of interest for water utilities in Norwegian municipalities. The implementation of LIDs and green solutions is a large focus area for stormwater management in Oslo. Descriptions in their guidelines promote the common responsibility for developing the city in a sustainable direction with the use of new solutions (Oslo Municipality, 2017). Requirements on detention of the stormwater during new development projects is also a requirement in Trondheim municipality, where several detention basins have been built the last years as a result of their guidelines. As well as for many other urban cities, is the population in Trondheim expected to increase. Within 2040, is the city expected to consist of 236 000 inhabitants, which is an increase of around 30% compared to the population in 2013 (Trondheim Municipality, 2013a). Furthermore, does this lead to challenges related to the capacity of the existing pipe network, which also is affected by the consequences of climate change.

Freni et al. (2014) claim computer modelling to be the most effective tool for the design and optimisation of sewer systems and wastewater plants. However, the input conditions and the model parameter used are always crucial. Rainfall has been advocated as one of the most important inputs to develop runoff response. Hence, the right choice of precipitation depth for designing any urban drainage system is important (Bisht, et al., 2016). Several software packages are available within the stormwater modelling field. One of the challenges when it comes to modelling LID measures is the translation of the complex and highly variable natural processes into a computerised system that allows a simple evaluation of the measures at a range of scales applicable to urban management (Elliott & Trowsdale, 2007).

MIKE URBAN is an urban water modelling software developed by the Danish Hydraulic Institute (DHI) for modelling water distributing networks and collection systems (DHI, 2017a). When modelling a collection system, either the SWMM5 engine or the MOUSE engine can be used. MIKE URBAN is based on a database for storing network as well as hydraulic modelling data. There are two ways of implementing LID measures in MIKE URBAN, where the first option is catchment based and the last network based. However, both options run with the MIKE 1D engine, exclusively (DHI, 2017c).

1.2 Thesis description and objectives

Several underground detention basins have been built in the municipality of Trondheim for the last 10-15 years. There is a lack of information about these structures, both with respect to operation and the effect of the management practices. There is also a need for evaluation of the effect of underground detention basins compared to other stormwater management practices, such as green roofs and bioretention cells.

Based on these research needs, the objectives of this thesis are as follows:

- Model and investigate the effects of different stormwater solutions in relation to CSO discharge for one short term high-intensity design rain and by simulating the system performance with one year of precipitation data.
- Compare and investigate the effect of different stormwater solutions with respect to flooding and discharge capacity.
- Evaluate the functionalities and the efficiency of underground detention basins, in addition to their performances in the relation of today's guidelines for stormwater management in Trondheim municipality.

These research objectives will be further investigated and analysed in this thesis with the use of a hydraulic model for a catchment area in Trondheim municipality.

2 Theory

2.1 Stormwater management

Urban stormwater management has a significant ecological, economical, and social importance. Lack of stormwater management may cause large damages concerning the infrastructure, health and environment. The extent of the damages depends on the management of the stormwater and the vulnerability of the surrounding buildings and infrastructure (Eckart et al., 2017). A report of NOU (2015) estimates the total costs of 1,6 to 3,6 billion NOK per year due to stormwater damages. These damage costs tend to increase in relation to the increased amount of extreme precipitation events (NOU, 2015). To be better prepared for such events, stormwater management practices should focus on flood reduction, reduce of CSO discharges and provide security for the citizens (Ødegaard, 2014).

Traditionally when dealing with stormwater management, improvement of water quality and flood control, has been seen and treated as two separate issues (Dietz & Arnold, 2018). Consequently, a traditional approach to urban stormwater management has been to use grey infrastructure and sewers to convey the stormwater through a centralized system as fast and safely as possible. The increased focus in bringing the developed watersheds to pre-development hydrological conditions has also made an increase in the use of low impact development (LID) controls, which also tend to create resiliency to adapt to climate change and improve the water quality (Eckart et al., 2017). In order to maintain the hydrological patterns in a best way, it is necessary to limit the runoff volume that leaves the site, in addition to restoring the groundwater and baseflow recharge (Emerson & Traver, 2008). Moreover, the performance of LID controls is dependent on the seasonal variation and initial conditions. In addition, is their performances related to the type and design of the measure, precipitation characteristics and the location of the watershed (Kristvik et al., 2019).

Another aspect related to LID controls is their effects on the volume control and CSO reduction, which has been investigated in several studies. A study done by Lucas and Sample (2015) in Virginia compared the effects on green and grey solutions. While the grey solution reduces the number of CSOs most, the green solutions obtained the smallest overflow volumes, lowest peak flows and were concluded as the most resilient system (Lucas & Sample, 2015). Moreover, another study done in Shanghai, concluded a combination of grey and green solutions to be the most feasible solution related to CSO reduction (Liao et al., 2015). A similar study has also been done in Norway, where the importance of blue-green solutions in future stormwater management is illustrated, and how coupling of LID structures in series can significantly reduce the required detention volumes. A combination of traditional detention basins and LID controls are preferable for volume and pollution control in addition to peak flow control, as the study shows that such a combination reduces the required downstream detention basin volumes substantially when applied in series. (Kristvik et al., 2019).

2.2 Stormwater management in Norwegian municipalities

Urbanization with an increased amount of paved surfaces in combination with climate change and more heavier rainfalls makes challenges on today's stormwater management practices. Procedures are often implemented in the municipal guidelines, and a study done by Groven (2015) shows that 70% of the municipalities in Norway has set requirements and guidelines of handling stormwater related to new development projects the last five years. The same study shows that 90% of the respondents perceive their municipality to be vulnerable to stormwater damages, and this may be related to population density and a larger amount of paved surfaces (Groven, 2015). As mentioned, the guidelines for stormwater management in Norway are mainly for new development projects. The municipality's right to give an order of stormwater measures to the owner of a property today is currently limited. The legislation is primarily aimed at new measures, and have therefore a less significance for already developed areas (NOU, 2015).

Four of the biggest cities in Norway; Bergen, Oslo, Trondheim and Stavanger have all made their guidelines on stormwater handling when dealing with new development projects. This is to make sure that new projects with increased paved surfaces will not lead to any capacity problems to the connected municipal pipelines. Successful stormwater management strategies follow an approach that evaluates the resource, land-use and design decisions with respect to their potential effects on watershed health, protection of people and property, economic growth and the volume of runoff increases (WEF Press, 2012). A report from the national association Norwegian Water, recommends stormwater management in relation to the use of the three-step strategy. This strategy consists of natural infiltration from smaller rainfall intensities in step 1, detention and regulation of larger rainfall intensities in step 2, and safe diversion on the surface with the use of secured flood paths in step 3 (Lindholm et al., 2008). The most optimal return period for a specific area could be obtained by optimizing the costs in relation to the expected damage associated with the rain event and the implementation of a stormwater system capable to manage the rain event (Paus K., 2016).

A rainfall event with high intensity may cause large damages and economical consequences related to basement flooding. Numbers from the report of NOU (2015), shows that Norwegian municipalities take responsibilities of 5 to 10 percent of the registered basement floodings. The insurance companies take responsibility for the remaining cases, which has unclear causes of damage (NOU, 2015). A minimum of 0,9 meters from the basement level to the connection point of the top pipe, is required to avoid basement flooding (Lindholm, et al., 2008).

The municipal plans of Oslo and Trondheim require use of open stormwater management which is infiltrated and returned to the vegetation and ground, in addition to being included as a positive element in the local environment (Trondheim Municipality, 2013a; Oslo Municipality, 2017). Implementation of LIDs for the handling of rainfall intensities in step 1, is in this relation an effective stormwater solution. There exists today only guidelines for implementation of strategies for step 2 and 3, but not for the handling of rainfall intensities under step 1. Also, do these plans not clearly define the required rainfall intensities, which in practice makes it challenging to document the fulfilment of the given legislation and regulations (Paus K., 2018).

Lack of clear and documentable requirements in some development areas is also a tendency that contributes to late scheduling of stormwater management in the construction process. Moreover, this could result in more expensive solutions and might also lead to closed drainage systems that consist of pipes and underground detention basins, which in principle only takes care of handling the functions of step 2 of the three-step strategy for stormwater management in Norway (Paus K., 2018). Furthermore, several detention basins are built every year as a result of the guidelines for stormwater management for handling rainfall intensities for step 2. The following chapter shows an overview of the stormwater practices built in the municipality of Trondheim.

2.2.1 Practices in Trondheim municipality

Guidelines for stormwater management in the municipality of Trondheim requires detention of the increased amount of stormwater in new development projects. Exceptions from this rule could only be done where it can be documented that the development will not cause any capacity problems in the connected municipal network. The minimum requirement related to volume is given as the water depth multiplied with the reduced area. Figures are made for both separate and combined sewer systems, and the maximum water flows out of the system are given. The requirements for combined systems are significantly stricter compared with separate systems that are connected to a recipient, where the requirement for detention starts with a reduced area of 300 m^2 for combined systems, and 500 m^2 when it is connected to a separate system. Other design criteria for detention basins are further explained in chapter 2.3.

An overview of the number of different stormwater solutions built in Trondheim is shown in Figure 1. This figure illustrates pipe systems to be the most common stormwater solution, followed by detention volumes made of plastic structures. The design objective of these detention basins is to limit a site’s postconstruction peak flow rate to or below its predevelopment level, which is often designed for a 20 years rainfall event. Some uncertainty related to the results in this figure should be considered, as the work with the registration of stormwater solutions is not completed (Gjeset, 2018).

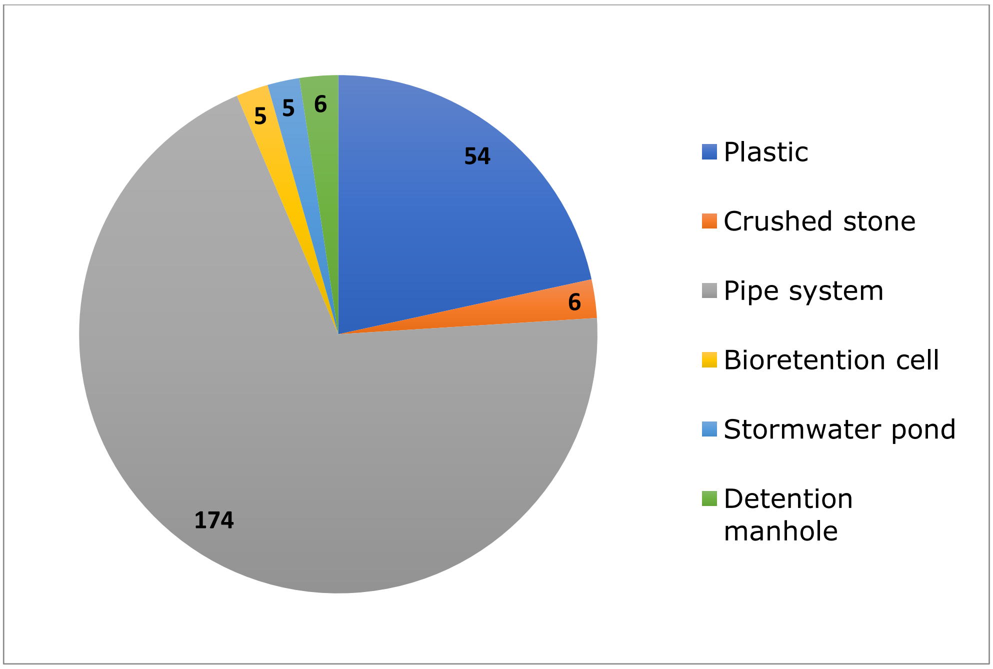


Figure 1: Overview of stormwater management solutions built in the municipality of Trondheim (Gjeset, 2018).

2.3 The dimensioning of underground detention basins

Implementing underground detention basins is a common solution that is often used for stormwater management. The rational formula, shown in equation 2.1, is used to determine the discharge of the stormwater for catchments smaller than 50 ha. Hydraulic models should be used for catchments greater than 50 ha and in cases where the catchment contain special conditions. Such models should also be considered where the consequence of wrong dimensioning is large (Trondheim Municipality, 2015).

$$Q = K \cdot C \cdot I \cdot A \tag{2.1}$$

Q	=	maximum discharge (l/s)
K	=	safety factor for climate change projections (-)
C	=	runoff coefficient (-)
I	=	rainfall intensity (l/(s · ha))
A	=	area (ha)

The rainfall intensity is found with the use of local IDF curves, which includes a specific concentration time. The runoff coefficient could be found with the use of a given table, but local conditions should also be taken into account. If the catchment consists of areas with different runoff coefficients, an average runoff coefficient should be determined. A runoff coefficient smaller than 0,3 should not be used (Trondheim Municipality, 2015).

With the use of figures from the guidelines of stormwater management in Trondheim municipality, which are shown in Appendix A and the reduced area shown in equation 2.2, the necessary detention volume and flow control could be obtained. An efficiency factor of 0,7 is set to the flow control. This affects the interaction between the volume and the outlet discharge and is included in the figure (Trondheim Municipality, 2015).

$$\text{Reduced area} = C \cdot A \tag{2.2}$$

A lifetime of 100 years is expected for such structures. Different technical solutions are available, but the use of detention basins of a pipe-system or plastic material is most common, as shown in Figure 1. Another option for the design of a detention basin is the use of crushed stone or other coarse stone materials, which are able to infiltrate some of the stormwater into the ground. These structures do normally have a pore volume of around 30%. The disadvantage of the use of this solution is the possibility of clogging in the system. This solution is normally not an accepted stormwater solution in Trondheim (Trondheim Municipality, 2015). Clogging is also a common problem for the basins that consists of a plastic system, which has a pore volume of around 95%. To avoid clogging, sand traps should be installed upstream and downstream of the structure. Instalment of an oil separator upstream is also important to avoid clogging of the system (Ødegaard, 2014).

2.4 The dimensioning of bioretention cells

Bioretention cells, which is also often referred to as biofilters or rain gardens, is defined as a landscape depression that consists of a surface ponding layer, vegetation, soil layer, storage layer, overflow structure and an optional underdrain system (Liu et al., 2014). Such measures include ensuring a natural water balance through infiltration, evaporation and retention in vegetation. Moreover, bioretention cells reduce the water conveyed to the wastewater treatment plant as a result of the infiltration processes. Bioretention cells are mainly within step 1 of the Norwegian Water's three-step strategy for stormwater management, which is based on capturing a fraction of the annual precipitation (Paus K., 2018).

There is a lack of documentation in guidelines concerning rainfall intensities within step 1 of the three-step strategy, in addition to corresponding requirements for stormwater management practices, as mentioned in chapter 2.2. One example of handling this is the establishment of requirements related to the average annual precipitation. These guidelines could consist of requirements of LID controls handling 95% of the annual precipitation, or an amount of treatment required before the water is discharged to the connected recipient. A study of Paus (2018) shows that rainfalls designed to take care of 99% of the annual precipitation, represents in average 81% handling of the rainfall from a two years return period. In contrast, rainfalls designed to take care of 95% of the annual precipitation, corresponds to handling only 35% of the amount of rainfall from a two years return period. However, a large amount of the annual precipitation is taken care of with the use of this dimensioned rainfall and would lead to a reduction of the necessary area for the stormwater solution (Paus K., 2018).

Bioretention cells are mainly suitable for small catchments, and guidelines from the U.S. recommends the catchment area not to be larger than 0,8 ha. Based on design suggestions from Braskerud & Paus (2014) for designing and construction of bioretention cells for a Nordic climate, the necessary cell surface area could be obtained using the following equation:

$$A_{bio} = \frac{A \cdot c \cdot P}{h_{max} + (K_{sat} \cdot t_r)} \tag{2.3}$$

A_{bio} = the bioretention cell surface area (m^2)
A = size of the catchment area (m^2)
C = the average runoff coefficient of the catchment area (-)
P = the amount of precipitation that the cell must be able to manage (m)
h_{max} = the maximum water level at the cell surface (m)
K_{sat} = the saturated hydraulic conductivity of the bioretention media (m/h)
t_r = the duration of stormwater flow into the cell (h)

Figure 2 shows the principles of a bioretention cell design with the recommended design values. H_{max} is typically between 15 to 30 cm and is particularly important for the cell's capacity to manage runoff from rainfalls with high intensity, as well as in cases where the infiltration capacity is reduced due to for example frozen media or ice covering at the cell surface. The recommended depth of the bioretention media is between 40 and 80 cm.

A Ksat value of at least 0,10 m/h is recommended to prevent the temperature from constraining the infiltration capacity of a bioretention cell for cold climates.

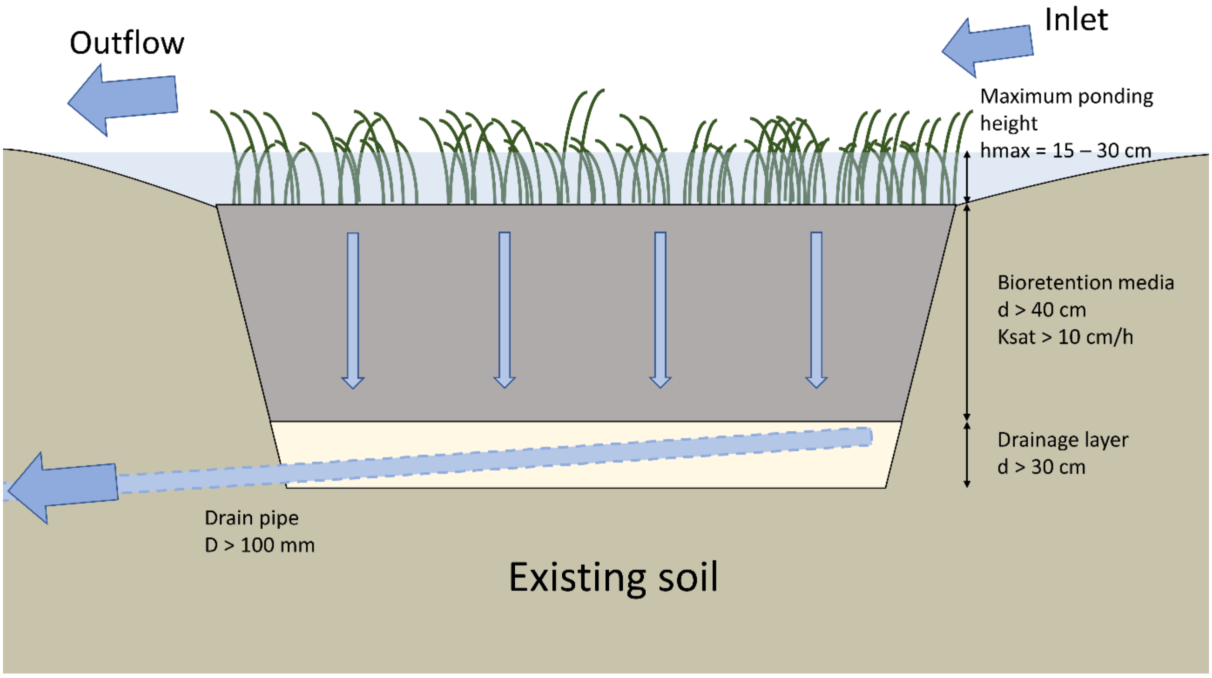


Figure 2: Principles of a bioretention cell design (Braskerud & Paus, 2014)

The table below from the same study of Paus (2018) illustrates rainfall design values in relation to handling a certain amount of the annual precipitation data from Risvollan, Trondheim. These values are based on precipitation data from January 1999 to December 2008 with a time resolution of 10 minutes. Values for 15 and 45 minutes are estimated by taking the average of the adjacent values (Paus K., 2018).

		Rainfall duration (min)											
		10	15	20	30	45	60	90	120	180	360	720	1440
% of precip. handled	50%	0,1	0,2	0,2	0,3	0,4	0,5	0,6	0,8	1,0	1,7	2,6	3,9
	65%	0,2	0,3	0,3	0,5	0,6	0,8	1,0	1,3	1,7	2,6	4,0	6,1
	80%	0,3	0,4	0,6	0,8	1,0	1,3	1,7	2,1	2,8	4,3	6,5	9,7
	90%	0,5	0,7	0,9	1,2	1,6	2,0	2,7	3,3	4,3	6,6	9,8	14,4
	95%	0,8	1,0	1,3	1,7	2,3	2,9	3,8	4,7	6,1	9,3	13,5	20,0
	99%	1,8	2,4	2,9	3,6	4,6	5,5	6,8	8,4	10,7	16,5	24,4	34,4

Table 1: Rainfall design values (mm) for handling a specific amount of the annual precipitation, based on data from Risvollan, Trondheim (Paus K., 2018).

2.5 MIKE URBAN

MIKE URBAN is a GIS-based modelling system used for modelling and the design of water distribution networks and collection systems for wastewater and stormwater (DHI, 2017a). The system is well implemented and used in the municipality of Trondheim in cooperation with DHI, which was the main reason for the selection of modelling software in this master thesis, in addition to an already available and calibrated model. Such hydraulic models might be a useful tool for evaluating the current state of the sewer system, and for an evaluation of the effects of implemented stormwater management practices. Furthermore, an important part of successful modelling is related to the model calibration and verification, which must ensure that the computed results fit the flow observations within a reasonable level (DHI, 2017c).

A MIKE URBAN model consists of the following hydraulic elements (DHI, 2017c):

- Nodes and structures: manholes, basins, storage nodes, outlets
- Links: pipes and canals
- Weirs
- Orifices
- Stormwater Inlets
- Pumps
- Valves

2.5.1 Hydraulic network modelling with MOUSE

Different runoff models are available within the simulation engine, where the time-area method and kinematic wave method are examples of two common options (DHI, 2017b). The computations in the time-area method are based on the simple treatment of hydrological losses and runoff routing by the time-area curve. While this model is based on minimum data requirements, is the kinematic wave a surface model based on moderate data requirements. The computations are in this method based on the comprehensive treatment of hydrological losses and runoff routing by the kinematic wave formula (DHI, 2017c).

The continuous runoff from MIKE URBAN can be modelled as a simple specification of a constant additional flow or as a MOUSE Rainfall Dependent Infiltration (RDI) computation. The latter provides detailed, continuous modelling of the complete land phase of the hydrologic cycle. Precipitation is routed through four different types of storages: snow, surface, unsaturated zone and groundwater. This leads to a long-term analysis that is applied to look at periods of both wet and dry periods in addition to inflows and infiltration to the sewer network. The RDI computation can be combined with any of the surface runoff models. The MOUSE RDI model uses a large number of parameters related to the different storage zones, which can be default defined or user-specified (DHI, 2017c).

2.5.1.1 Boundary conditions

A model boundary condition can be defined as an external interference, which forces the behaviour of the computed variables within the model domain (DHI, 2017c). The boundary conditions represent various types of water loads related to rainfall, infiltration, wastewater and more. Boundary conditions other than water loads are related to water levels, air-temperature and evapotranspiration.

MOUSE uses three groups of boundary conditions (DHI, 2017c):

- Catchment loads: Air temperature, evapotranspiration, rainfall and flow, where sediment and temperature properties are included in the latter.
- Network loads: Water level and discharge. Represents all kinds of hydraulic loads.
- External water levels: Specification of external water levels interacting with the collection system.

2.5.1.2 Simulation

There are two types of simulations in MIKE URBAN – runoff and network simulation. The network simulation uses the runoff simulation as a base input. The runoff simulation simulates the runoff generation based on the catchment loads, and the surface runoff model needs to be defined in this simulation. The network simulation simulates the behaviour of the network when the runoff simulation result file is applied to the model. A model type for the dynamic simulation needs to be defined, consisting of these options: dynamic wave, diffusive wave and kinematic wave (DHI, 2017c).

For the simulations, there are two different options of numerical engines: the classic MOUSE (Modelling Urban Sewers) engine and the MIKE 1D engine, where the latter is a result of a re-engineering and merger process of the calculation capabilities of the collection system and the river simulation packages (DHI, 2017e). The modelling of LID technologies are only possible with the MIKE 1D engine, as the MOUSE engine does not support these functions (DHI, 2017a).

2.5.2 Required basin geometry inputs

To be able to implement underground detention basins in MIKE URBAN, a basin geometry must be defined, which is specified with at least two sets of three values (DHI, 2017a):

H	=	elevation (m)
Ac	=	wetted cross-section area, perpendicular to the main flow direction. Used in calculation of the flow velocity in the structure, assuming uniform velocity distribution (m^2)
As	=	surface area used for calculation of volume (m^2)

The definition of this geometry is the same for the soakaway node, which is used for designing and modelling bioretention cells. Figure 3 shows an example of a basin geometry with the corresponding Ac and As. The latter is used for the calculation of the volume, while Ac is used for the calculation of the flow velocity in the structure, assuming uniform flow distribution (DHI, 2017d).

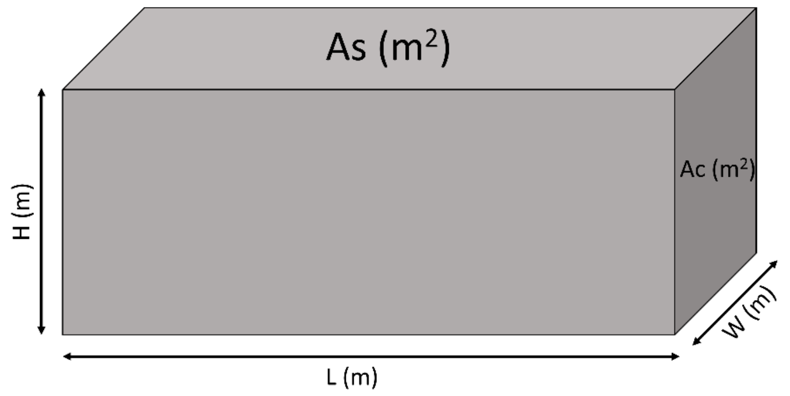


Figure 3: Parameters required for the basin geometry in MIKE URBAN.

The first set of values corresponds to the structure bottom, while the last set corresponds to the surface level. This is further illustrated in Table 2, where values for a basin geometry with a volume of 21 m^3 is shown, including standard dimensions of 1,2 meters for the height and width of the basin. Intermediate values between these are linearly interpolated. Since the MOUSE engine associates the first H-value to the bottom level of the node, the geometry can be reused several places in the model (DHI, 2017c).

H (m)	Ac (m^2)	As (m^2)
0,0	0,0	17,5
1,2	1,44	17,5

Table 2: Example of defining the different values of a basin geometry.

In the guidelines from the municipality of Trondheim for stormwater management (Trondheim Municipality, 2015), a flow regulation for controlling the discharge out of the detention basins are required. In MIKE URBAN, this can be implemented by inserting a regulation in the selected link. This regulation could either be a maximum discharge as a function of the water level in a user-specified node or a maximum discharge as a function of the water level difference between two user-specified nodes (DHI, 2017c). For the first option, the control is applied only within a specified range of water levels. If the water level exceeds this specified range, an unregulated flow applies. The specified range should therefore cover all expected water levels. If the water level reaches outside this range, a sharp transition between the Q defined by the control function and the unregulated Q would occur causing numerical instabilities (DHI, 2017d).

This is further illustrated in the following equation (DHI, 2017d):

$$Q_{reg} = \begin{cases} \min\{Q(H_A), Q_{nat}\} \text{ for } H_{min} \leq H_A \leq H_{max} \\ \text{else} \\ Q_{nat} \end{cases} \tag{2.4}$$

Q_{reg}	=	applied and regulated discharge (m^3s^{-1})
$Q(H_A)$	=	discharge by the regulation function (m^3s^{-1})
Q_{nat}	=	unregulated, natural discharge, obtained as an explicit estimate, based on the known water levels in the previous time step on each side of the regulation point (m^3/s)

H_A	=	water level at the control point, A (m)
H_{min}, H_{max}	=	water levels at the control point A defining the range in which the regulation is to be applied (m)

For situations where detention basins are implemented in the model, the basins represent control point A.

2.5.3 Implementation of LID controls in MIKE URBAN

There are two possibilities of modelling LID practices in MIKE URBAN (DHI, 2017c):

1. Modelling of LID at the screening level (catchment based).
2. Detailed hydraulic modelling of individual LID structures (network based)

The first option includes deployment of LID control to each catchment, where it is possible to specify the area of the individual control and the size of the catchment's impervious area that is funnelled into it. Seven different LID controls can be modelled, such as bioretention cells, green roof, infiltration trenches, porous trenches and more. Depending on the LID control, different structural layers can be modelled. For bioretention cells, this includes: surface, soil, storage and underdrain layers with its individual properties and functional elements. Examples on such properties are storage height, vegetative cover, surface roughness, soil thickness, porosity and infiltration capacity. This method is only allowed for the kinematic wave runoff model and runs with the MIKE 1D engine (DHI, 2017c).

The second option uses modelling with a network node type named soakaway in MIKE URBAN and runs with the MIKE 1D engine, exclusively. The soakaway node is allowed for all available surface runoff models in MIKE URBAN. A conceptual drawing of the soakaway node is shown in Figure 4, and the soakaway can be connected to the pipe network as a basin or any other node element. It is modelled as a regular basin, and a required basin geometry as explained above is necessary for the modelling of a soakaway. MIKE URBAN calculates the water level based on the inflow, outflow to the drainage network and the infiltration. Flow regulation is an option but has to be applied if the up level of the outgoing pipe is placed at the bottom of the soakaway. The boundary condition is related to the initial water level, and different options of infiltration are possible: no infiltration, constant infiltration and infiltration through the sides and bottom as a function of the water level. Other parameters that must be defined is porosity, infiltration rate, ground level, bottom level and cover type (DHI, 2017c). Cover type is further explained in the chapter below related to surface flooding.

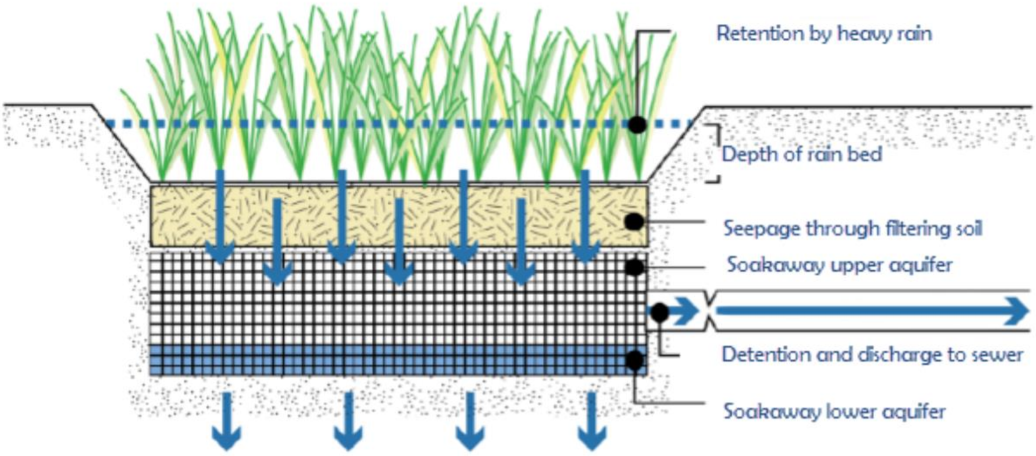


Figure 4: Model concept of the soakaway function (DHI, 2017c)

2.5.4 Surface flooding

Manholes, basins and soakaways are per default considered open at the top. This corresponds to a cover type equal to "normal". In situations where the water level in a node reaches the ground level, an artificial basin is introduced on the top of the node with a surface area 1000 times larger than the node's surface, thus simulating the surface inundation as shown in Figure 5 (DHI, 2017c). When the outflow surmounts the inflow from the node, the stored water from the inundation basin re-enters the system (DHI, 2017d).

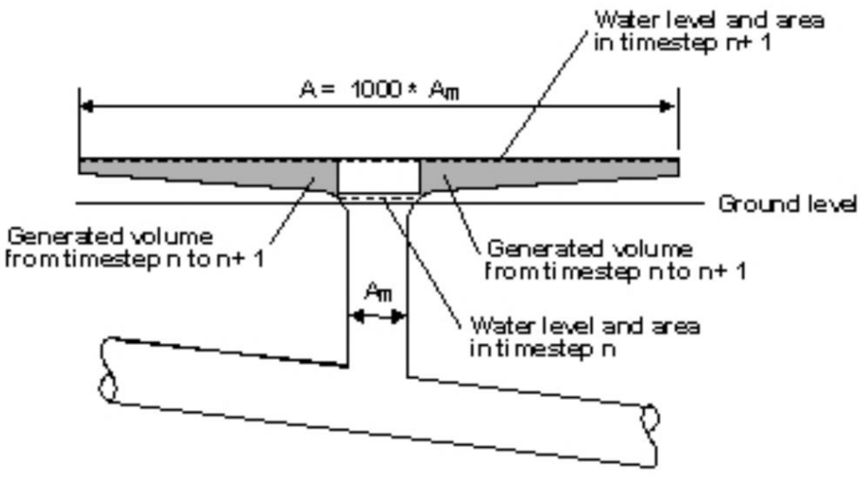


Figure 5: Surface flooding simulation related to cover type "normal" (DHI, 2017d)

In addition to the "normal" cover type, two other options are possible. Alternatively, a sealed node can be selected, meaning all water exceeding the ground level is taken out of the model. The pressure will rise without any water on the ground surface. The last option is to define the node as a spilling node. In this case, the water escapes irreversible from the model, if the water level reaches and exceeds the defined ground level of the node (DHI, 2017c).

3 Method

3.1 About the study area

The Lade area is located northeast of the city Trondheim in Norway. Trondheim is a coastal city with an annual mean temperature and precipitation of 5,5 °C and 950 mm, respectively (NCCS, 2016). The study area in this thesis is related to the catchment area of Fagerheimsbukta, which is a part of Lade in Trondheim. This area does mainly consist of residential buildings. A popular walking trail is located near the coast, which also has some popular bathing spots during summer time. The pipe network does mainly consist of combined sewer systems established around 1953-1995, where the main pipe material is concrete, in addition to a small amount of PVC.

An already calibrated MIKE URBAN model given from the Danish Hydraulic Institute (DHI) of the catchment of Fagerheimsbukta was used in this thesis. The location of the study area, which has a total catchment area of 22,54 ha and includes 1113 PE, is shown in Figure 6. There is a mixture of impervious surfaces and green areas. The average imperviousness of the total catchment area is equal to 33,85%, which is based on calculations from MIKE URBAN with the respect of existing development and roads. The calculation of the imperviousness in the model is further explained in chapter 3.3.1.

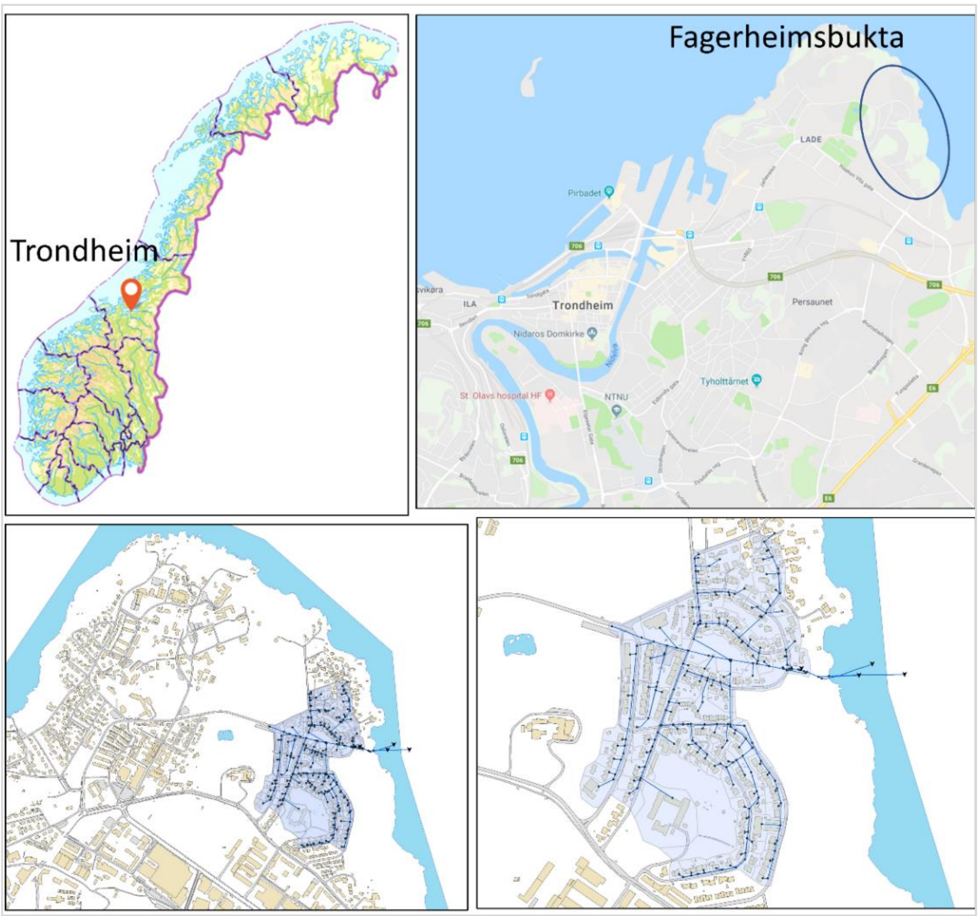


Figure 6: Study area of Fagerheimsbukta, Trondheim

Fagerheimsbukta contains an overflow weir, which produces CSO discharges during large rainfall events. Given data from the municipality of Trondheim, shows an average CSO discharge of 20 456 m^3 and an average discharge to the connected treatment plant of 222 335 m^3 per year, in addition to a CSO duration of 319 hours in average. These values are based on registered data from the connected treatment plant, Ladehammeren from the period 2009-2018 (Trondheim municipality, 2019). Figure 7 shows an overview of the amount of discharge to the treatment plant and the CSO discharge, in addition to the duration of the CSO events, based on the same data as mentioned above.

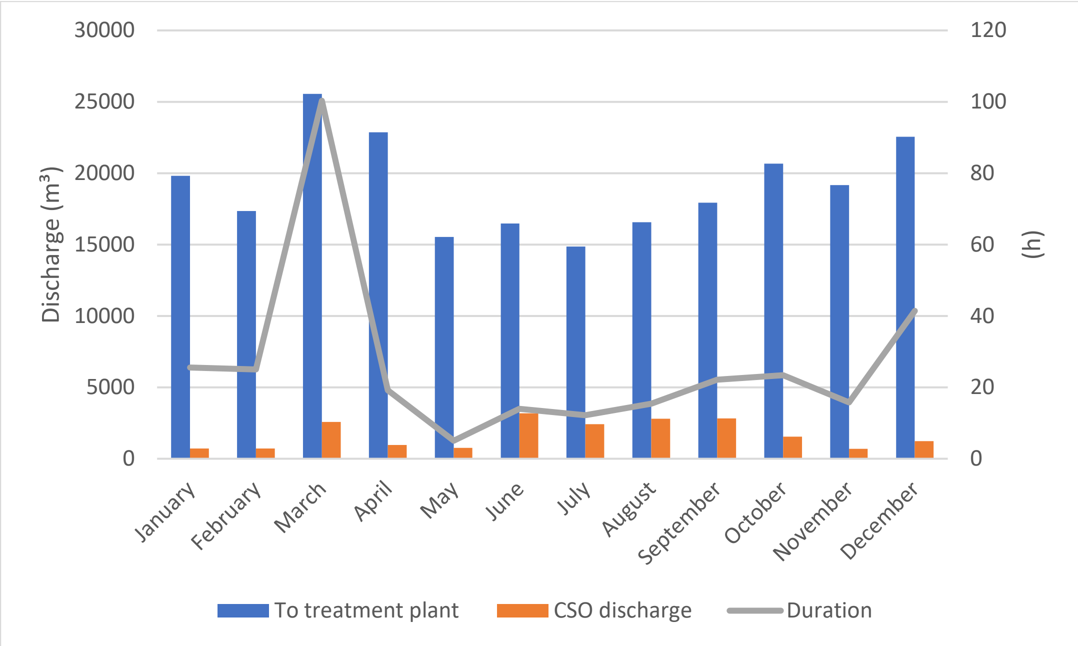


Figure 7: Discharges and duration of CSO events, Fagerheimsbukta (Trondheim municipality, 2019).

3.2 Three scenarios

To be able to evaluate the performance of different stormwater management practices, three different scenarios were implemented in the modelling procedure in MIKE URBAN:

- Scenario 0: No implemented stormwater practices (today's situation).
- Scenario 1: Implementation of underground detention basins.
- Scenario 2: Implementation of LID practices.

Concerning today's situation, the study area of Fagerheimsbukta does not exist in any stormwater management practices. To be able to evaluate the effect of the different scenarios, the structures were designed based on guidelines and recommendations from literature. A new division of the catchment area was necessary to be able to design the structures related to properties and already existing development.

Two different situations according to the percentage of implemented stormwater solutions in relation to the total amount of sub-catchments were obtained, hence a situation of 50% and 100% instalment of stormwater practices. Table 3 shows an overview of the different scenarios and their respective description.

Description		Scenario
Today's situation		0.
Basins	50% implementation	Basin50
	100% implementation	Basin100
LID	50% implementation	LID50
	100% implementation	LID100

Table 3: Overview of the different scenarios with the percentage of implemented structures concerning the total number of sub-catchments.

Implementation of stormwater management practices is often related to new development projects, and a catchment area consisting of 100% implemented solutions is therefore not a very realistic approach according to the current situation. However, with today's requirements of stormwater management for development projects, this scenario is a relevant topic of interest for the evaluation of the effects according to CSO discharges and capacity problems, in addition to the evaluation of today's guidelines.

The situation of 50% instalment of stormwater practices consists of 45 structures, compared to 90 structures for the scenarios of 100% implementation. These structures are distributed throughout the whole catchment area, which is illustrated with brown colour in Figure 8. The manholes visualised with green in the figure, are for scenario Basin100 changed to basins and for scenario LID100 changed to soakaways. Consequently, approximately every sub-catchment that exist of properties are implemented with a stormwater practice in scenario Basin100 and LID100. For better visualisation, is the remaining manholes deleted in Figure 8.

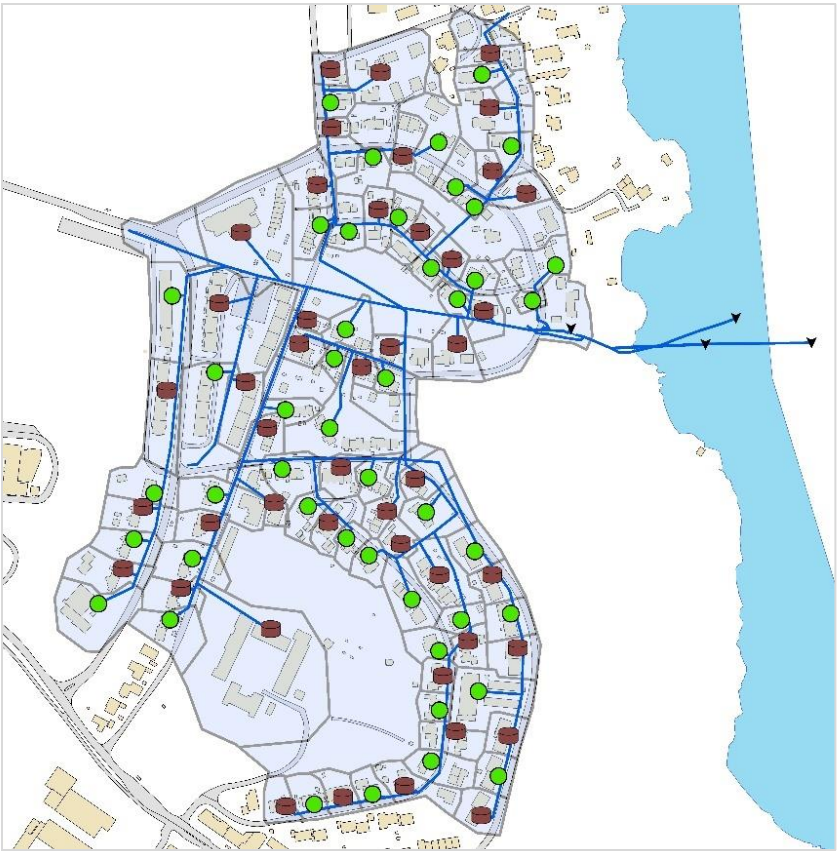


Figure 8: An overview of the placement of 50% of the stormwater measures for scenario Basin50 and LID50.

The different scenarios illustrated above are simulated with the use of two different rain events with precipitation data from Trondheim for an average year and an extreme event. The latter has a return period of 20 years, including a factor of 1,2 for climate change projections based on data from the meteorological station Voll in Trondheim. Mike Urban develops a hyetograph of this rains with a time interval of 2,5 minutes. Regarding the average year, rain data from the average year was used. Data for the precipitation, temperature and evaporation from the meteorological station of Risvollan in Trondheim were used as boundary conditions in the model for the 2011 simulations. For the long-time simulations of this event, the MOUSE RDI model was used together with the time-area method to provide an accurate picture of actual loads on treatment plants and combined sewer overflows. For the simulations of the extreme event, the time-area method was used, where the runoff amount is controlled by initial loss, size of the contributing area and by a continuous hydrological loss (DHI, 2017b).

The maximum CSO discharges concerning the overflow weir at Fagerheimsbukta happen during summertime, as illustrated in Figure 7. However, the longest duration of the CSO events is obtained in March and December. As a consequence of this, the total year of 2011 was simulated in order to evaluate all aspects of the CSO discharges.

3.3 Model rebuilding

The existing model given by DHI consisted of a larger area of Lade in Trondheim with four overflow weirs (Djupvika, Fagerheimsbukta, Østmarka and Ringvebukta). Because of the scope of rebuilding the model, it was chosen to model the impacts of stormwater solutions for one of the overflow weirs mentioned above, and the area of Fagerheimsbukta was chosen. As a result of this, the existing sub-catchments with the related pipes and manholes connected to the other overflow weirs were deleted and removed from the model. The remaining catchment connected to the overflow weir at Fagerheimsbukta was mainly consisting of combined sewer systems. An advantage of this is the simplification of establishing and analysing result layers, which were one of the reasons for modelling the overflow weir at Fagerheimsbukta.

3.3.1 New sub-catchments, manholes and service pipes

To be able to implement the different stormwater solutions for the scenarios including detention basins and bioretention cells, a new division of sub-catchments were necessary. Furthermore, implementation of service pipes and manholes for each sub-catchment was necessary for the modelling of the different scenarios. Background layers of the existing development related to buildings, roads, manholes and service pipes were uploaded in the model. This simplified rebuilding of the model into sub-catchments related to existing development and led to a new division of 105 sub-catchments with a total area of 22,54 ha, compared to the original model which had a total amount of 20 sub-catchments.

With the new division of the total catchment area, the related stormwater solution to each sub-catchment was implemented. Service pipes from the sewer system to each sub-catchment were implemented, consisting of a diameter of 200 mm to avoid capacity problems of the connected system. An elevation of 20 cm from upstream to downstream of the service pipe was chosen in order to secure natural flow with the use of gravitation.

The service pipes from each sub-catchment were connected to the existing sewer system by implementing new manholes, where MIKE URBAN automatically interpolated the bottom level and ground level of the manholes. Figure 9 illustrates this new model setup for some of the sub-catchments with an illustration of the implemented manholes and service pipes.

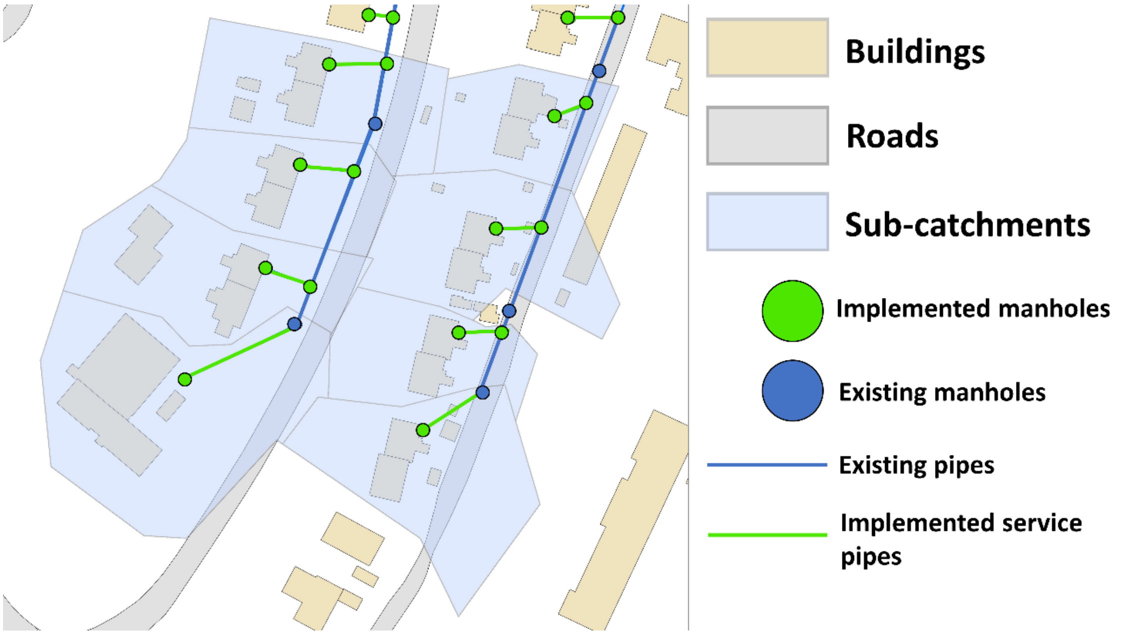


Figure 9: Overview of some sub-catchments with the respective pipe connections and manholes

For scenario 0, which is based on today's situation, manholes were implemented at the inlet from each sub-catchment connected to the implemented pipelines from each sub-catchment, as shown in Figure 9. Standard dimensions were developed for these manholes, consisting of a diameter of one meter and a standard height of 2 meters from bottom level to ground level. The "Nodes and Structures" editor in MIKE URBAN makes it possible to later define these manholes as basins or soakaways in order to model the other scenarios. A copy of the model was made for each scenario to simplify any necessary changes at a later stage. The new model set-up consists of 254 pipes, 105 sub-catchments and 249 manholes. 90 of these manholes were later changed to basins or bioretention cells, representing the situation of 100% implementation of stormwater measurements.

3.3.2 Imperviousness, RDI data and catchment connections

After the removal of the other overflow weirs with their respective sewer system and the new division of sub-catchment, there was necessary to recalculate the imperviousness of each sub-catchment, as well as developing new catchment connections. MIKE URBAN has several automated catchment tools, which was used to simplify and improve the efficiency of this work. The catchment parameter processing wizard was used for the recalculation of the imperviousness. This is an automated and reproducible way to calculate imperviousness, time of concentration and other hydrological parameters for hydrological models (DHI, 2017a). This function calculates the imperviousness with the use of the implemented background layers including buildings and roads, which were defined as 100% impervious. A hydrological reduction factor of 0,8 was defined, meaning that 80% of the impervious areas are directly connected to the network.

This reduction factor is included to take into account that not all of the water from the impervious surfaces is led to the draining system or other intakes.

For the simulation of the long-time scenario of the year 2011 with the use of the MOUSE RDI model, model data for the additional catchment flow and RDI had to be defined. This included defining parameter sets and the RDI area as a fraction of the actual model area. The RDI area is the amount of the sub-catchment which contains of pervious surfaces, and the area is obtained with the use of the following equation:

$$A_{RDI} = (100 - imp) \cdot r \cdot A \tag{3.5}$$

imp	=	the imperviousness of the sub-catchment (-)
r	=	reduction factor (-)
A	=	the total area of the sub-catchment (ha)

MIKE URBAN consist of a default RDI parameter set, which is illustrated in Appendix D. These parameters are used for the simulations of the scenarios for the average year. Some of these parameters are related to actual physical data and should for the calibration be based upon a comparison with historical measured discharges (DHI, 2017c). The validation of the MOUSE RDI parameters is a time-consuming task which requires expertise within the modelling software. Since the main focus of this thesis is to evaluate and compare the effects of the different scenarios, the correct definition of these parameters is not prioritised and the default settings are therefore used.

The catchment connection wizard in MIKE URBAN was used to connect the catchment to an item. This tool automatically connects all selected catchments to manholes based on a number of principles (DHI, 2017a). Each sub-catchment in the model was connected to the nearest node, which was defined as manhole, basin or LID, depending on the scenario simulated.

3.4 Implementation of underground detention basins in MIKE URBAN

To determine the effect of underground detention basins, 90 basins were constructed and connected to each sub-catchment in the model based on the guidelines from the municipality of Trondheim. An overview of the defined dimensions of each basin was calculated and obtained in Excel before the implementation of the basins in MIKE URBAN.

The rational formula, given in equation 2.1, was used to determine the discharge from each sub-catchment, where an average runoff coefficient was used. This coefficient was based on the automatic generated imperviousness percentage in MIKE URBAN, as well as the runoff coefficients for different surfaces listed in the guidelines. A runoff coefficient of 0,9 was used for the impervious surfaces, while a runoff coefficient of 0,4 was used for the remaining surfaces which were assumed consist of mostly green areas. A safety factor (k) of 1,2 was added to take climate change projections into account. A return period of 20 years was chosen, which is recommended for populated areas. The rainfall intensity was then found by using IDF-curves from Trondheim (Voll – Moholt – Tyholt) and a rainfall duration of 10 minutes. This led to a rainfall intensity of 124 l/s*ha.

The reduced area (m^2) was calculated using equation 2.2. From these calculations, the necessary detention (mm/m^2 reduced area) and the flow control (l/s) was obtained using figures from the guidelines of Trondheim municipality (Trondheim Municipality, 2015). With the use of the guidelines, the necessary detention volume was found for each basin with its respective outlet discharge in l/s. A standard dimension of the wetted-cross area perpendicular to the main flow was obtained, in order to simplify the implementation of detention basins in MIKE URBAN. The required basin geometry with its required parameters is explained in chapter 2.5.2. A standard width and depth of 1,2 meters were decided for all basins, leading to a defined Ac of 1,44 m^2 when the basin is full. In periods of minimal precipitation causing empty basins, the Ac is defined to equal zero. With the calculated detention volume and the area Ac, the required length and the following area of As could be obtained. This leads to a full definition of the required parameters of the basin geometry in MIKE URBAN. One basin geometry was obtained per volume defined. With specifying the first H value as the bottom level of the node, the same geometry for each volume could be used several places in the model, which made the work with implementing the basins more time-efficient.

The flow control and the installation of a link regulation are necessary to avoid capacity problems from the detention basins to the connected sewer system. These controls were implemented in the model through the settings for regulation, where a selected link and control node A was chosen, as explained in chapter 2.5.2. To secure that the Q defined by the control function is within a safe range of discharges to avoid numerical instabilities, H_{min} and H_{max} were set to 0 and 100 m.

The standard and normal cover type concerning surface flooding were chosen in order to create a realistic situation when the water level reaches the ground level in the model. In these situations, water spills on the ground surface and is stored in an artificial basin, to be returned into the sewer system, as shown in Figure 5.

3.5 Implementation of bioretention cells in MIKE URBAN

Two different methods exist for the implementation of LID controls in MIKE URBAN, as explained in chapter 2.5.3. The given model from DHI includes runoff model A, which consist of the time-area method. To be able to use the first option of LID modelling at the screening level, runoff model B must be used, which consist of the kinematic wave method. A combination of these different models for individual catchments in one runoff computation is not possible (DHI, 2017c). The second option for implementation of LID controls in MIKE URBAN was therefore chosen as a method in this thesis, which is a network-based approach and are available for the given surface runoff model.

3.5.1 Calculation of the necessary basin geometry

A bioretention cell was designed for each catchment using the soakaway function. This function requires a basin geometry same as for the construction of underground detention basins. A standard dimension for Ac, the wetted cross-section area perpendicular to the main flow direction was decided, hence simplified the work with the implementation of the bioretention cells in MIKE URBAN. A standard depth of 0,6 meters for the bioretention media was chosen based on the recommended design values from Braskerud & Paus (2014), as mentioned in the theory chapter 2.4. Same background theory was used for the definition of h_{max}, which was set to 0,2 meters.

Because of the restrictions in the model, the total depth of the structure was set to the sum of the bioretention media and the maximum water level at the cell structure, h_{max}. This led to a total depth of 0,8 meters, which was used for the calculation of the area of Ac. A standard width of 2,5 meters was chosen. These values led to a design value of 2,0 m^2 for Ac, which was used as a standard dimension for each bioretention cell in the model.

Table 1 was used for determining the amount of precipitation, P. This included use of a rainfall duration (tr) of 1 hour. For the effectiveness of the bioretention cell, a rainfall designed to take care of 99% of the annual precipitation was chosen as a design criterion. Furthermore, this leads to a rainfall duration of 5,5 mm with the use of the given values in Table 1.

All sub-catchment except one consists of an area less than 0,8 ha, which is recommended in the literature (Braskerud & Paus, 2014). Furthermore, with the use of the mentioned values above and an average runoff coefficient, the area of the bioretention cells was computed using equation 2.3.

3.5.2 Necessary input parameters

Required parameter inputs for the soakaway in MIKE URBAN, is related to the porosity, infiltration rate, ground level and initial water level. The initial water level is set to be equal to the ground level. This is done to ensure the soakaway to be gradually filled up from the bottom of the structure during rainfall events.

The up level of the connected service pipe is placed at the top of the soakaway, hence equal to the ground level of the soakaway. This is done to avoid requirements of link regulation, which were mentioned in chapter 2.5.3. Figure 10 shows the geometry of the soakaway and illustrates the connected service pipe, which is defined as Q_{out} in the figure below.

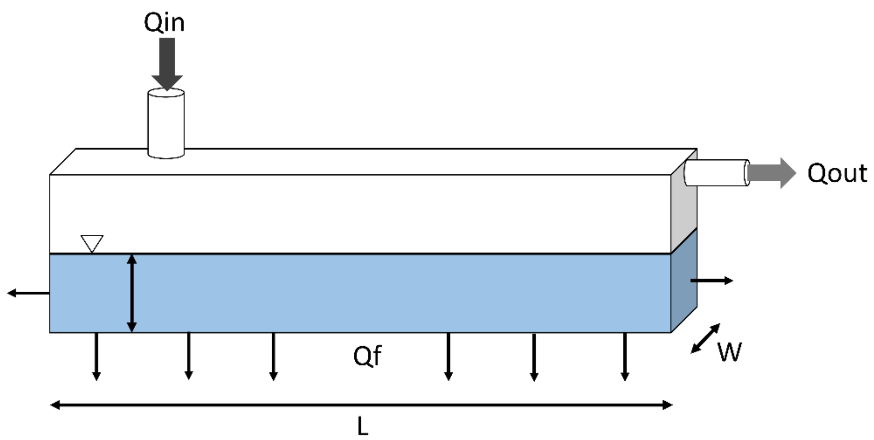


Figure 10: Schematic of the soakaway function in MIKE URBAN (DHI, 2017c)

Q_f defined in the figure above is equal to the infiltration rate. This is set to 0,10 m/h, which is recommended for the design of bioretention cells for a Nordic climate (Braskerud & Paus, 2014). Assuming a high content of sand in addition to some leaf compost and topsoil, the porosity of the bioretention media was set to 0,35. The bioretention media does normally consist of 50-60% sand (Ødegaard, 2014).

Because of the mentioned restrictions of the soakaway function in MIKE URBAN, where layers cannot be defined, an average porosity was calculated using equation 3.6:

$$p_{average} = \frac{(D_{bio} \cdot p_{bio}) + (h_{max} \cdot p_{surface})}{D_{bio} + h_{max}} \tag{3.6}$$

D_{bio}	=	depth of the bioretention media (m)
H_{max}	=	the maximum water level at the cell surface (m)
$P_{surface}$	=	the porosity of the surface layer (-)
P_{bio}	=	the porosity of the bioretention media (-)

The porosity of the surface layer is set to equal 1,0, assuming only water storage at the cell surface. Furthermore, this results in an average porous coefficient of 0,51, with the implementation of the defined variables in the equation above.

4 Results and discussion

The results of the simulations of the different scenarios are in this chapter presented and discussed. This is summarised into three sections, consisting of the following:

- The effects of CSO discharge for the three scenarios.
- Evaluation of the capacity of the sewer system for each scenario.
- Evaluation of the design criteria of the detention basins.

4.1 CSO discharge

Simulations were done for an average year of precipitation data and for an extreme rainfall event with a return period of 20 years, including a factor of 1,2 for climate projections. Maximum CSO discharges and volumes are found for both events, in addition to the discharge and volume of sewer led to the wastewater treatment plant (WWTP).

4.1.1 Extreme event

A maximum rainfall intensity of 148,8 l/s · ha is obtained with the use of IDF curves from the area of Voll, Moholt and Tyholt in Trondheim for a 20 years rainfall event, including a climate factor of 1,2. Table 4 illustrates the results related to CSO discharge for the different scenarios. A general reduction of CSO discharge and accumulated volume can be seen with comparing the results of implemented solutions with today's situation, as shown in Table 5. An interesting aspect of the data in this table is the small increase of CSO volume between scenario Basin50 and Basin100. There is a 20,5% reduction of the maximum CSO discharge between the two scenarios with implemented detention basins. However, this results in a small increase of CSO discharge.

Data from these tables can be compared with the graphs shown in Figure 11 and Figure 12. A further comparison of the graphs of scenario Basin50 and Basin100, the maximum peak reduction is illustrated. However, a higher duration of the CSO event is shown, which explains the increase of the accumulated CSO volume from 50% to 100% implementation of detention basins.

Scenario	Max CSO discharge (m^3/s)	Max discharge to WWTP (m^3/s)	Accumulated CSO discharge (m^3)	Accumulated discharge to WWTP (m^3)
0.	0,51	0,0394	827,1	161,3
Basin50	0,48	0,0390	784,4	204,8
Basin100	0,38	0,0372	790,6	207,7
LID50	0,39	0,0315	400,1	134,4
LID100	0,06	0,0263	51,8	99,2

Table 4: Results of CSO discharge for the simulations of the extreme rainfall event.

Changes in relation to scenario 0				
Scenario	Max CSO discharge	Max discharge to WWTP	Accumulated CSO discharge	Accumulated discharge to WWTP
Basin50	-5,5 %	-1,0 %	-5,2 %	27,0 %
Basin100	-24,8 %	-5,6 %	-4,4 %	28,8 %
LID50	-22,8 %	-20,1 %	-51,6 %	-16,7 %
LID100	-88,1 %	-33,3 %	-93,7 %	-38,5 %

Table 5: Changes in percentage in relation to scenario 0 for the extreme rainfall event.

A significant reduction in CSO discharge and accumulated volume for the scenarios with implemented LID controls, is illustrated in Figure 11. What especially stands out from these results, is the large decrease of CSO discharge for scenario LID100 with 100% implemented bioretention cells. Comparing this scenario with today's situation, results show a reduction in 88% of the maximum weir discharge and a corresponding reduction in 94% in volume reduction. When comparing these results with findings from other studies on LID practices performances, the obtained results seem unreliable. Several studies show LID stormwater controls to be most effective at controlling the shorter return period events. The study of Eckart et al. (2017) summarises the knowledge of LID as a stormwater technique and the current research state of this topic. Their research showed LID stormwater controls to be more effective at controlling the hydrological impacts of shorter return period and concluded a combination of LID strategies and traditional stormwater management practices to be the best strategy for larger events. These results are also supported by the study of Damodaram, et al. (2010), which simulated a combination of combined best management practices and LID development. The obtained results from the study showed a significant stormwater control for small events and less control for flood events. Hence, the use of the soakaway function for LID implementation may be a limited tool for the design of bioretention cells in MIKE URBAN.

The soakaway function uses some simplifications to illustrate the hydrological functions of a bioretention cell. Moreover, the restrictions for implementation of different soil layers with its related parameters affect the functions ability to adapt the functionality of a bioretention cell in a realistic way. The use of the same geometry as for detention basins with the porosity, infiltration rate and initial water level as input parameters, leads to a highly simplified solution. Compared to the design of detention basins, the difference between the two solutions is the reduction in volume as a result of the porosity, in addition to the infiltration rate which leads to a reduce in transported water from the cell to the connected sewer system. Overall, this leads to a solution which is very similar to the function implemented for the detention basins. Because of its availability to infiltrate some of the water, the reduction of CSO discharge for the scenarios of LID controls shown in Figure 11, is large in comparison to the other scenarios. The functionalities of the soakaway are similar to the functions of detention basins including crushed stone materials. Such basins have a porosity of around 30% and are capable of infiltrating water. As illustrated in Figure 2 for the recommended design of a bioretention cell, a drain pipe should be implemented at the drainage layer placed at the bottom of the bioretention cell. In the construction of the bioretention cell in this master thesis, the drainage pipe was placed at the top of the function due to restrictions explained in chapter 3.5.2. This leads to an overall simplified solution, where the possibilities of accumulated water on the cells surface is not included in the soakaway function.

Figure 11 shows the discharge of transported water to the treatment plant for each scenario. The ratio between the accumulated CSO discharge and the accumulated discharge to the WWTP is increasing for each scenario implemented, which are illustrated in the given values in Table 4. The highest ratio is obtained for scenario LID100 for 100% implementation of bioretention cells, which corresponds to the large decrease of CSO discharge.

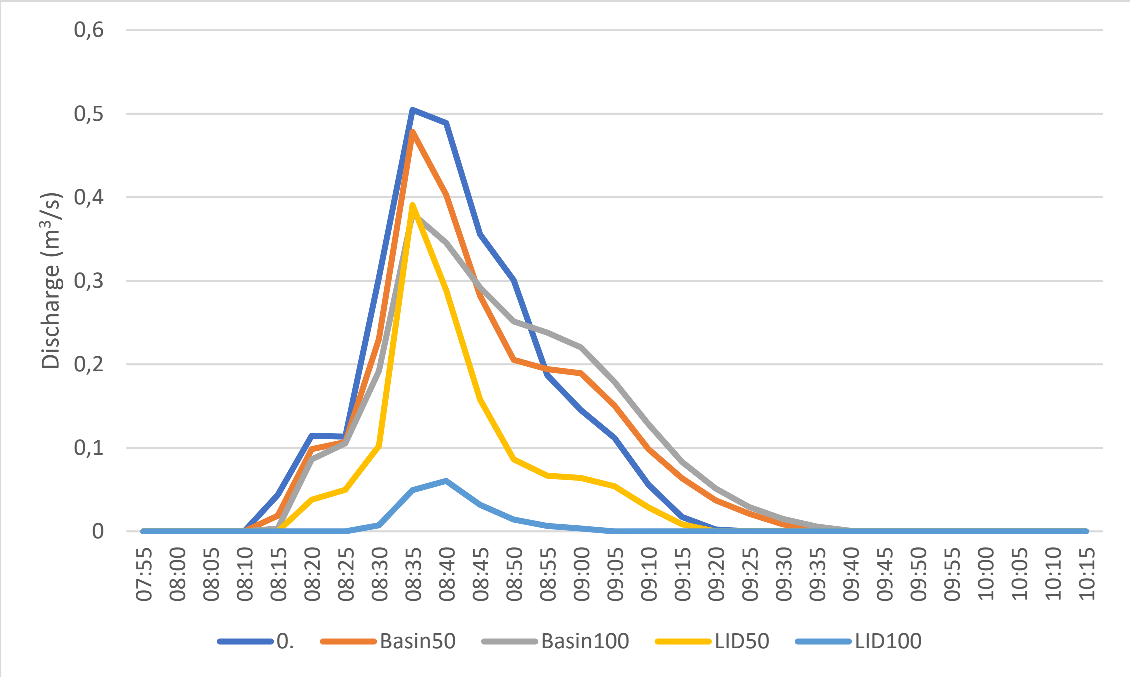


Figure 11: CSO discharges for the scenarios during an extreme rainfall event.

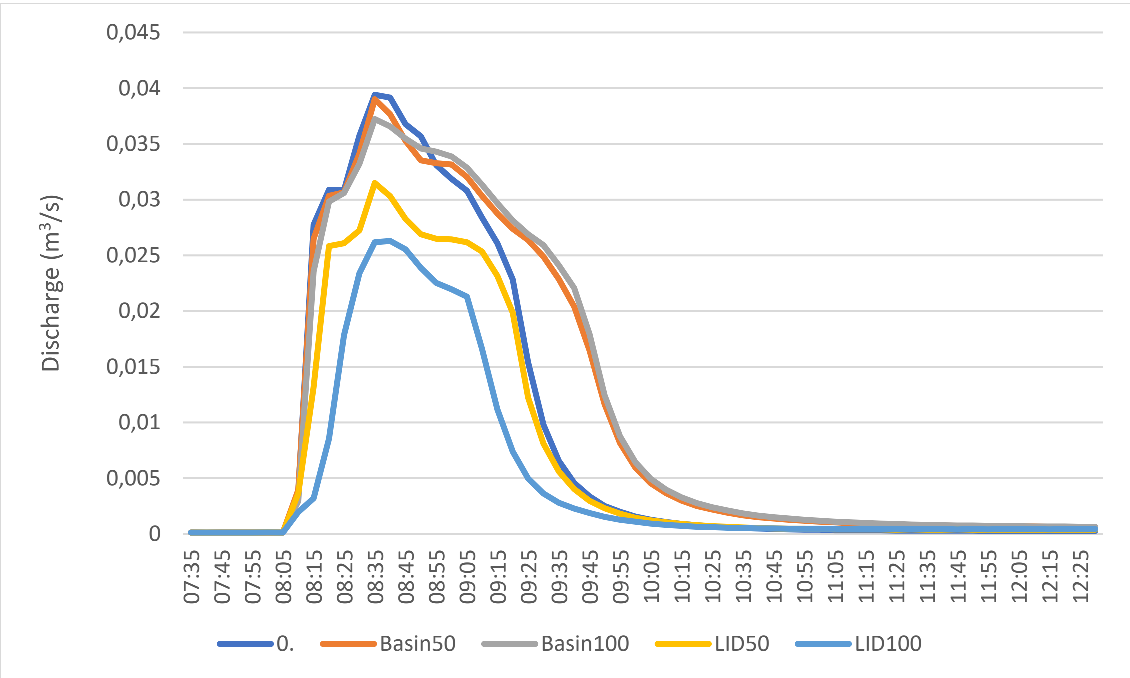


Figure 12: Discharge transported to the connected WWTP during an extreme rainfall event.

4.1.2 Average year (2011)

The results from the simulations using rainfall data from an average year, is shown in Table 6. A reduction of CSO discharges is shown, in addition to a reduction of the duration and the number of CSO events. There is a large variation in the given results compared to the numbers of registered CSO discharge and discharge to the treatment plant of Fagerheimsbukta, which were shown in chapter 3.1. For the scenario of today's situation, the obtained accumulated CSO discharge is 31,6% lower than the average CSO volume obtained from the period 2009-2018. The same pattern is shown for the obtained accumulated discharge to the wastewater treatment plant, which is 56,2% lower than the volume of treated discharge observed at the treatment plant. Likewise, the duration of CSO events is lower for the obtained results in this study, with a duration of 227 hours compared to the registered 319 hours at the treatment plant. There have been reconstructions at the overflow weir at Fagerheimsbukta (König, 2019), which affected the geometry of the weir. The model used in this thesis does not include these changes, which explains the differences in the obtained results and the registered numbers from the treatment plant.

According to the targets for the duration of CSO events with disposal connected to the fjord, the obtained results are within an acceptable range. The municipality of Trondheim has defined a limit of 400 hours concerning CSO discharge (Trondheim Municipality, 2013b). Moreover, the reduce of CSO duration with the implementation of detention basins are as shown in Table 6, minimal. However, the number of CSO events related to detention basins are reduced with 11,4% for 50% implementation and 15,8% for 100% implementation, as shown in Table 7. LID implementation reduces the number of CSO events even more, with 56,8% and 98,1% for the scenarios of 50% and 100% implementation, respectively. As explained before, restrictions in the model affect the functions ability to reflect reality, and uncertainty should be taken into account related to the result of the LID simulations.

The reduce of accumulated CSO discharge is relatively small for the scenarios of implemented detention basins, which corresponds to the obtained results from the extreme event simulations. Hence, a reduction of 6,2% and 8,1% of the CSO volume is obtained for the respective scenarios with 50% and 100% implemented detention basins. This is affected by the minimal reduction in duration and CSO discharge. The maximum CSO discharge is reduced with 11% and 27% for the scenarios of detention basins, which leads to the reduction of the number of CSO events. However, these results show a small reduce of accumulated CSO discharge for the scenarios of implemented detention basins, which indicates detention basins to be relatively inefficiently for the average rainfall events.

Scenario	Duration (h)	Number of CSO events	Max CSO discharge (m^3/s)	Accumulated CSO discharge (m^3)	Accumulated discharge to WWTP (m^3)
0.	227,3	361	0,50	13 996	97 298
Basin50	224,4	320	0,44	13 134	99 710
Basin100	223,1	304	0,37	12 865	99 913
LID50	81,8	156	0,33	3481	64 028
LID100	2,8	7	0,12	251	34 817

Table 6: Results of CSO discharge from simulations of an average year of precipitation data.

Changes in relation to scenario 0					
Scenario	Duration	Number of CSO events	Max CSO discharge	Accumulated CSO discharge	Accumulated discharge to WWTP
Basin50	-1,3 %	-11,4 %	-11,4 %	-6,2 %	2,5 %
Basin100	-1,8 %	-15,8 %	-26,6 %	-8,1 %	2,7 %
LID50	-64,0 %	-56,8 %	-34,8 %	-75,1 %	-34,2 %
LID100	-98,8 %	-98,1 %	-76,4 %	-98,2 %	-64,2 %

Table 7: Changes in percentage in relation to scenario 0 for the simulations of an average year of precipitation data.

4.2 Evaluation of the capacity

4.2.1 Basement flooding

Another aspect of the evaluation of different stormwater solutions, is their effects on the capacity of the sewer system. High rainfall intensities may cause flooding in basements as a result of capacity problems of the sewer system. This may cause large damages with respect to the impacts on the affected property, as well as economical consequences. The catchment of Fagerheimsbukta is evaluated with respect to the critical water level in manholes concerning basement flooding. The municipalities require a height of 0,9 meters from the pipe top in order to avoid flooding in basements, as mentioned in chapter 2.2. Illustrations with respect of this critical water level are made in MIKE URBAN to evaluate the risk of basement flooding and for the evaluation of the effects of implemented stormwater measures during an extreme event with a return period of 20 years. The results layers illustrate the maximum node depth of the manhole, which is calculated as the node water level minus the node invert level (DHI, 2017a).

Figure 13 shows the situation as it is today for scenario 0. For a better visualisation, is the implemented manholes connected to each sub-catchment, deleted. An evaluation of the water level in the detention basins can be seen in chapter 4.3. As shown in the figure below, the nodes coloured with dark blue represent a water level which is non-critical concerning basement flooding. The other manholes consist of a water level higher than 0,9, which is critical.

There is a total amount of 159 manholes of the total catchment area of Fagerheimsbukta, where 61 of these manholes has a critical water level for a 20 years rain event. Moreover, this relates to about 38,4% of the total amount of manholes, where a total overview of the simulations from each scenario is shown in Appendix B. For the evaluation of the effects of the stormwater solutions, the same evaluation is done for the scenarios with implemented stormwater measures, which are shown in Figure 14.

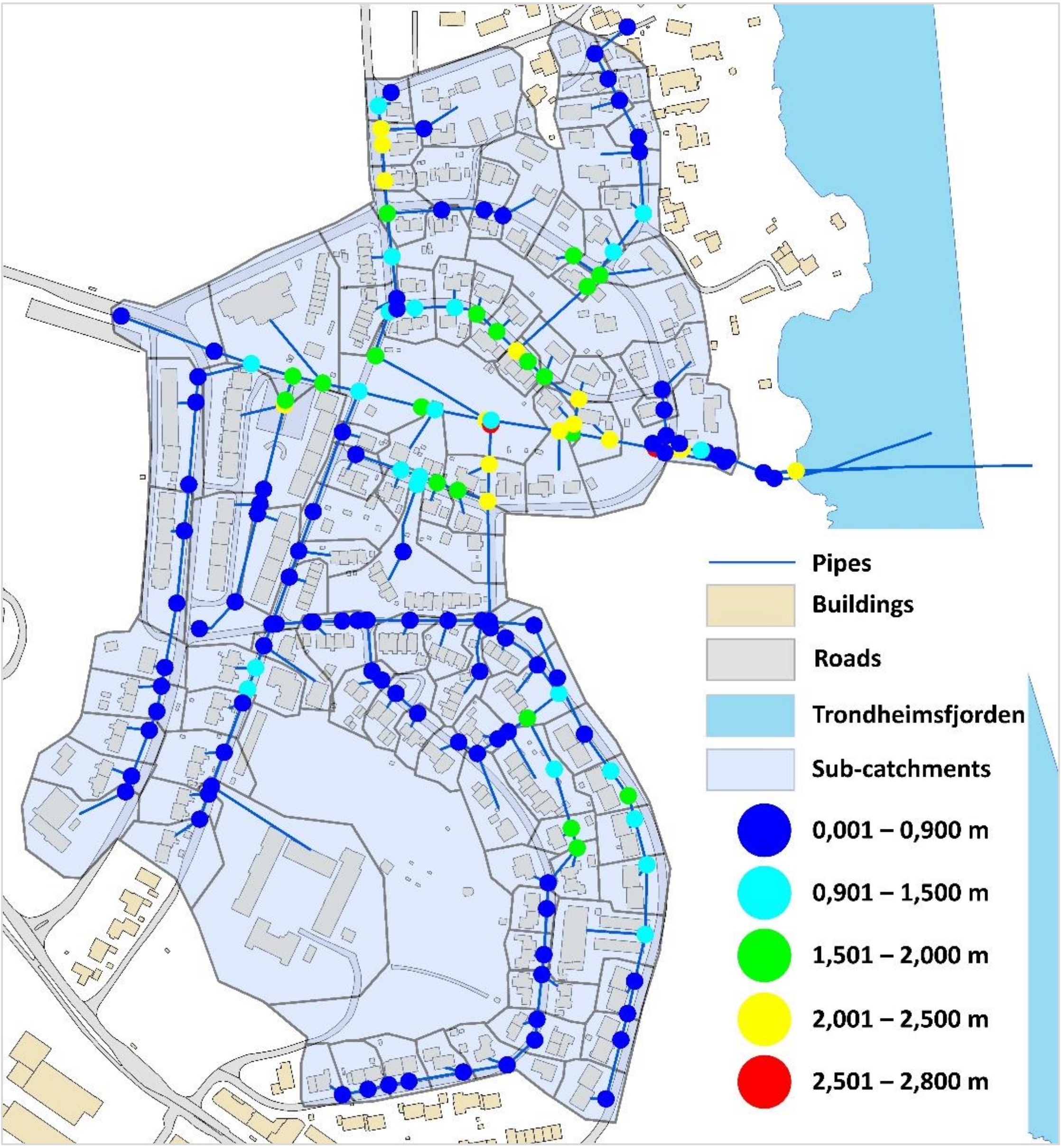


Figure 13: Results of the maximum water level in the manholes for scenario 0, obtained from the simulations of the extreme event.

By comparison of scenario Basin50 and Basin100 with today's situation, a reduce in critical water level is illustrated. Implementation of 50% detention basins corresponds to a reduce of critical nodes, where 23,3% of the manholes contains a water level higher than 0,9 meters. Scenario Basin100 with 100% implemented detention basins reduces the critical nodes even more, with a total of 4,4% critical nodes.

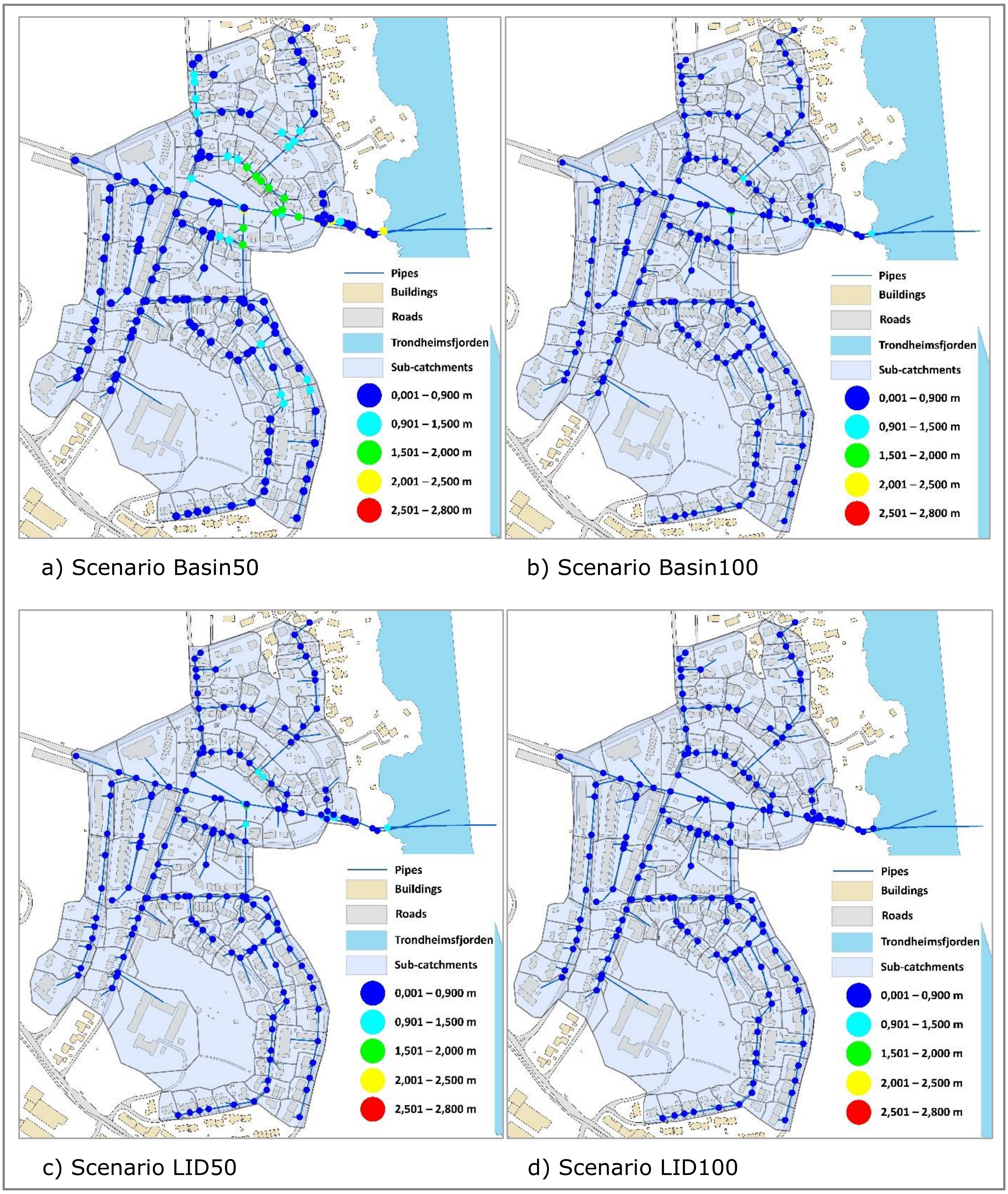


Figure 14: Results of maximum water level in the manholes for implemented stormwater solutions.

For the scenarios of implemented bioretention cells, is the effects concerning critical water level even higher, as illustrated in Figure 14. This figure shows only a few critical levels for the 50% implementation and none for 100% LID implementation. These results correspond to the obtained results of CSO discharge in the previous chapter, where the effects for the scenarios with implemented bioretention cells were greater than for the scenarios with implemented detention basins. As previously discussed, these results do not illustrate the functions of the bioretention cells satisfactorily with its simplified design.

Overall, these results indicate the effects of stormwater measures related to critical water level, where the risk of basement flooding decreases with the proportion of implemented stormwater measures. This is summarized in Figure 15, where the percentage of each interval concerning the water level is illustrated.

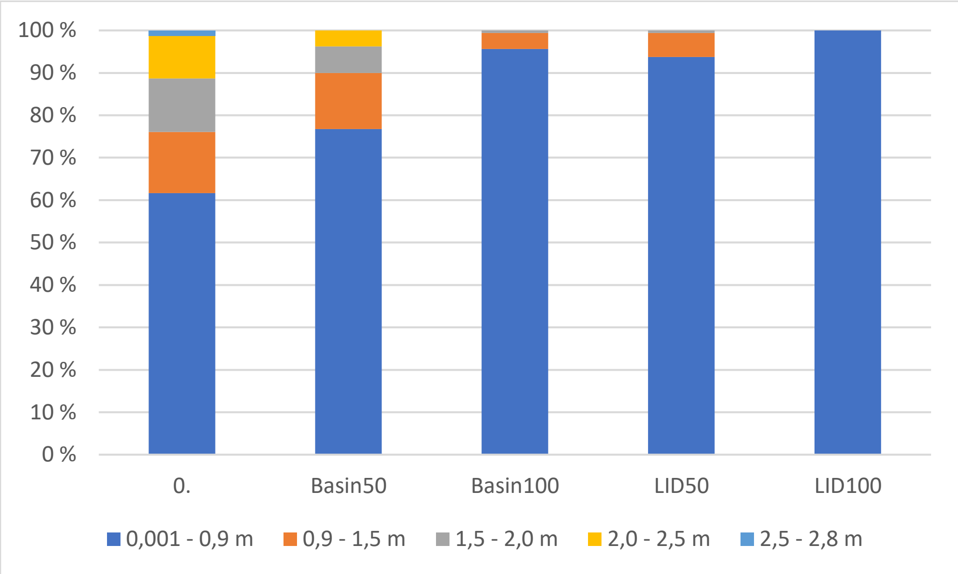


Figure 15: Critical water level in relation to basement flooding in the percentage of the total number of manholes.

4.2.2 Surface flooding

In order to evaluate the effect of the different scenarios in terms of surface flooding for a 20 years return period, a similar analysis is done with respect to the water level above ground level. Moreover, this is in MIKE URBAN illustrated as the node flood, which is calculated as the node water level minus the node ground level (DHI, 2017a). The results obtained from the simulation of scenario 0, shows nine critical manholes concerning surface flooding, which is illustrated in red in Figure 16. With the implementation of 50% detention basins, this is reduced to one critical manhole which has a water level above ground level, shown in Figure 17. For the remaining scenarios, all the critical nodes are replaced with nodes that contain water level within an acceptable range for the risk of surface flooding.

As shown in Figure 16 for the results of today’s situation, most of the critical nodes are placed near the overflow weir in the catchment. Furthermore, this is also the case in the evaluation of basement flooding and may be the result of insufficient capacity of the upstream sewer system, which leads to high discharges and pressure to the respective manholes and causes risk of surface flooding.

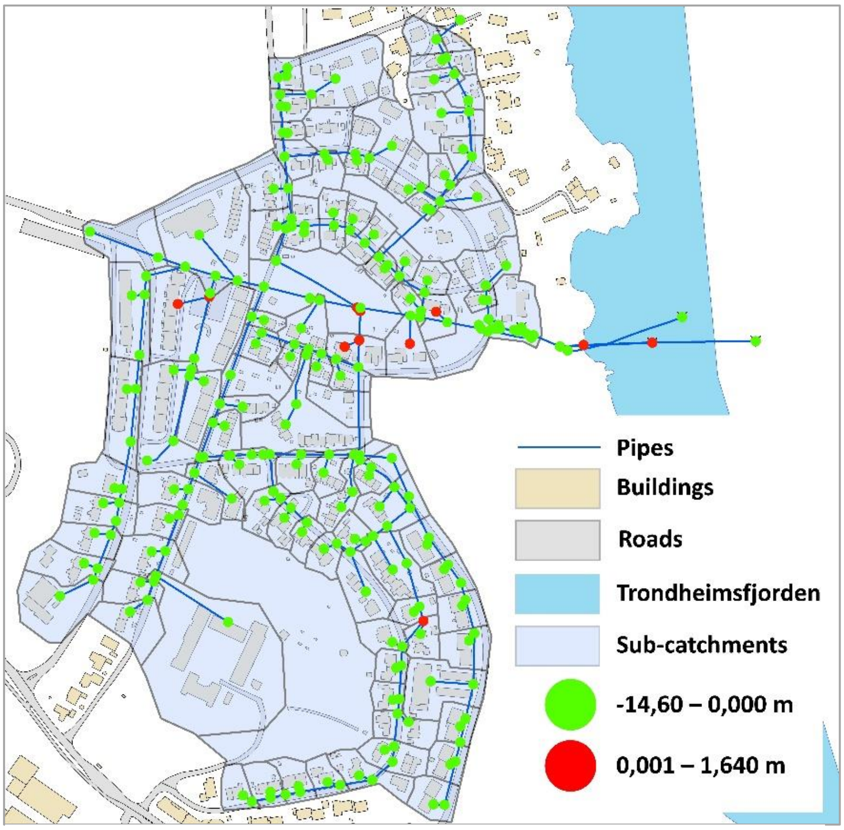


Figure 16: The results of maximum node flood for scenario 0

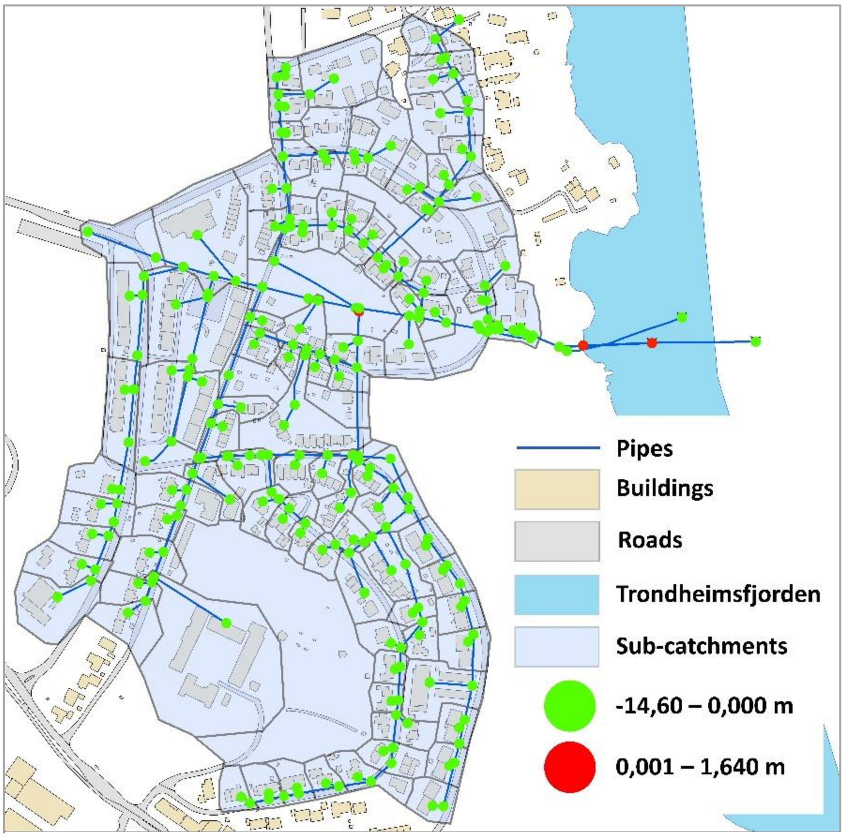


Figure 17: The results of maximum node flood for scenario Basin50

4.2.3 The capacity of the sewer system

The capacity of the sewer system concerning maximum pipe discharges during a 20 years rainfall event is in this chapter evaluated. The theoretical maximum capacity of the sewer system today is automatically calculated in MIKE URBAN, where the maximum discharge is computed with the Manning formula for the full flowing pipe. The Manning formula uses the cross-sectional average velocity, the Gauckler-Manning coefficient, the hydraulic radius and the slope as input parameters (DHI, 2017a). Hence, the theoretical maximum capacity of the sewer system should be equal for every scenario. Nevertheless, while simulating the scenarios with implemented detention basins, backflow was experienced in the basins as a result of the capacity in the connected sewer system. To be able to evaluate the water depth in the detention basins, which is done in chapter 4.3, the slope of the implemented service pipes was changed by adding 2 meters elevation from downstream to upstream of the service pipes. Furthermore, this led to an increase in the capacity of the service pipes, and the backflow effects were significantly reduced, which is further explained in the following chapter. Moreover, because of the changed slope in the service pipes, the maximum theoretical discharge varies slightly in the scenarios with implemented detention basins compared to the other scenarios. Figure 18 shows an overview of the theoretical maximum capacity of today's sewer system for Scenario 0, LID50 and LID100. An overview of the maximum capacity for the scenarios with implemented detention basins are shown in Appendix C. The increase of the capacity of the implemented service pipes for scenario Basin50 and Basin100 because of a higher elevation, are illustrated in Table 8.

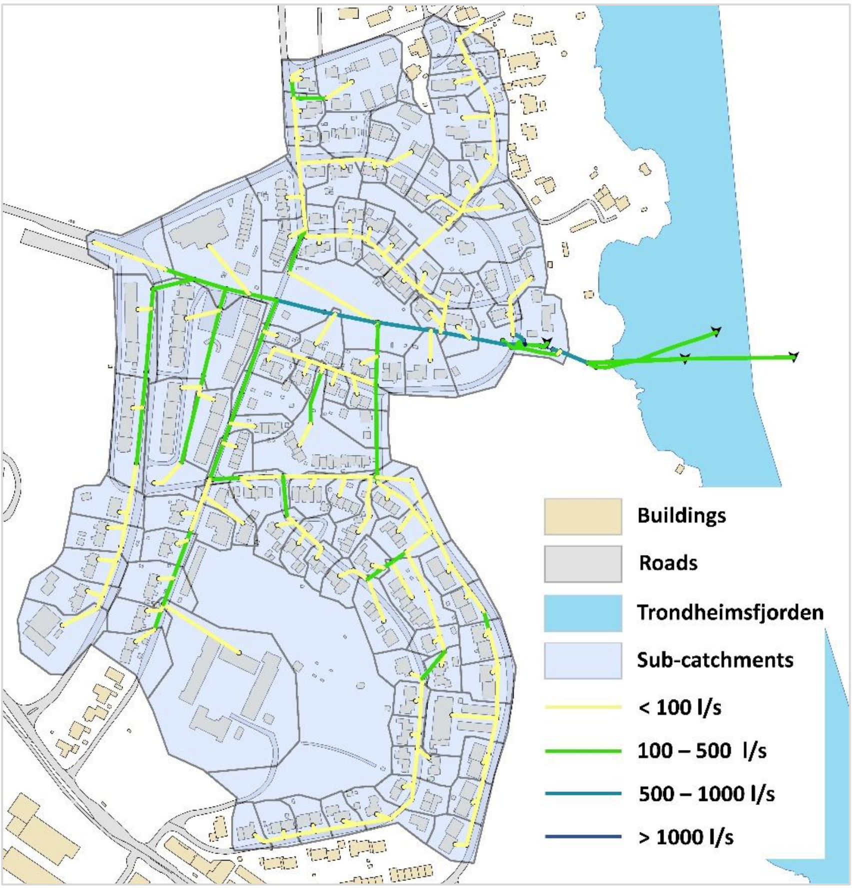


Figure 18: The theoretical maximum capacity (l/s) of today's sewer system, for scenario 0, LID50 and LID100.

	Number of pipes	
Interval (l/s)	0, LID50, LID100	Basin50, Basin100
< 100	187	107
100 - 500	53	133
500 - 1000	11	11
> 1000	1	1
Sum	252	252

Table 8: The theoretical maximum capacity (l/s) of the sewer system for different scenarios.

Figure 19 shows an overview of the maximum discharge in l/s for each pipeline for the simulations of scenario 0 during the extreme rainfall event. An overview of the maximum discharge for all scenarios is shown in Table 9. The results obtained show a general decrease in the maximum discharge in the sewer system for the scenarios with implemented stormwater measures.

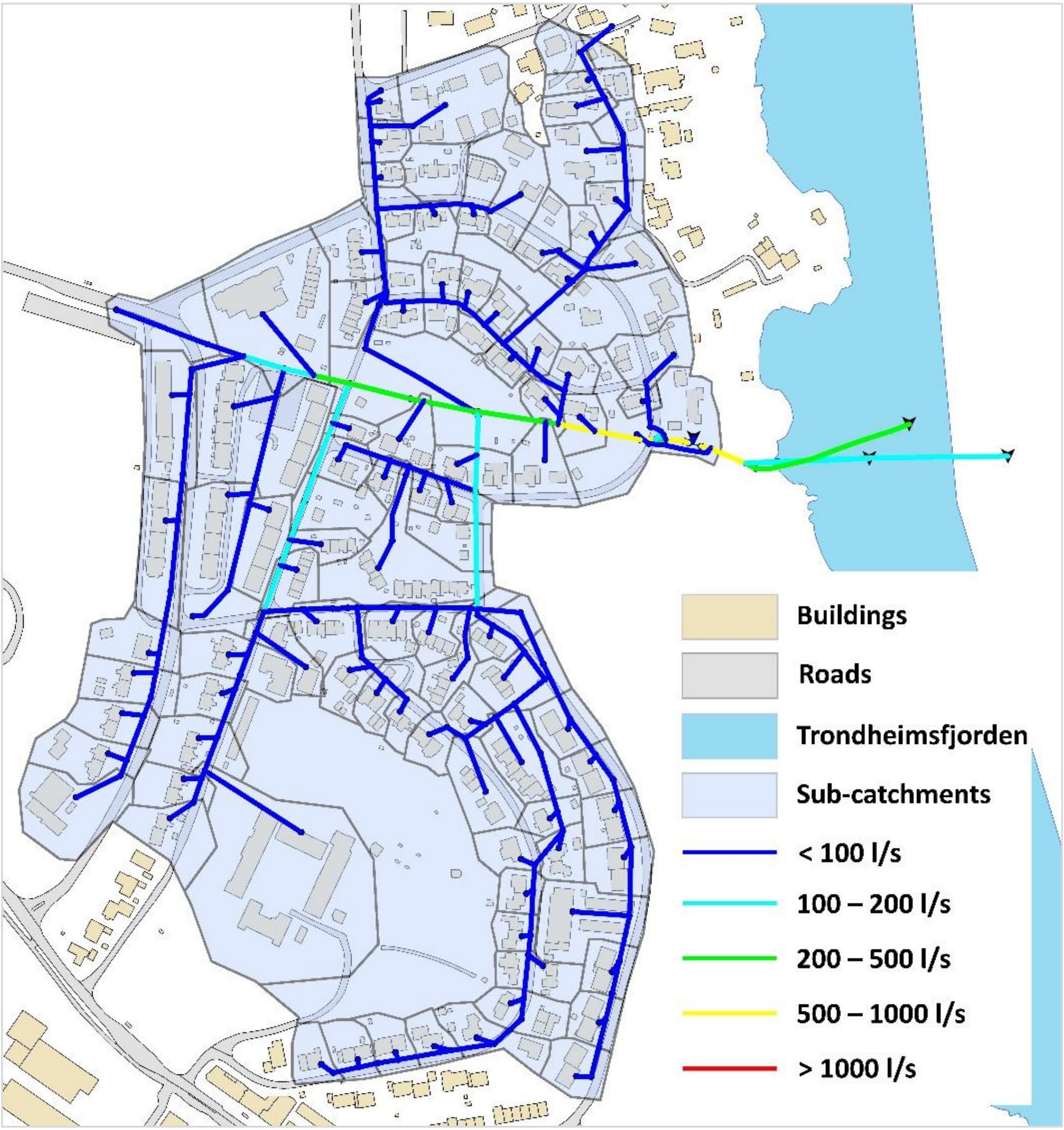


Figure 19: Maximum discharge (l/s) for the 20 years rainfall event

	Number of pipes				
Interval (l/s)	0	Basin50	Basin100	LID50	LID100
< 100	219	225	225	225	252
100 - 200	15	10	15	15	0
200 - 500	11	15	12	12	0
500 - 1000	7	2	0	0	0
> 1000	0	0	0	0	0
Sum	252	252	252	252	252

Table 9: Maximum discharge (l/s) for the different scenarios during a 20 years rainfall event.

Furthermore, the capacity utilization is investigated based on the results concerning the theoretical maximum capacity and the maximum discharge of the sewer network during an extreme rainfall event. MIKE URBAN uses these results and calculates the capacity utilization automatically for each pipeline. Moreover, this is done by computing the ratio of the theoretical maximum capacity and the maximum discharge, which MIKE URBAN defines as $Q_{actual}/Q_{Manning}$ (DHI, 2017a). The capacity utilization of the pipe network is illustrated in percentage, where the results of scenario 0 are shown in Figure 20. Table 10 shows the results for all scenarios, where the pipes containing a maximum discharge larger than the capacity of the sewer system is reduced for the scenarios of implemented stormwater management solutions.

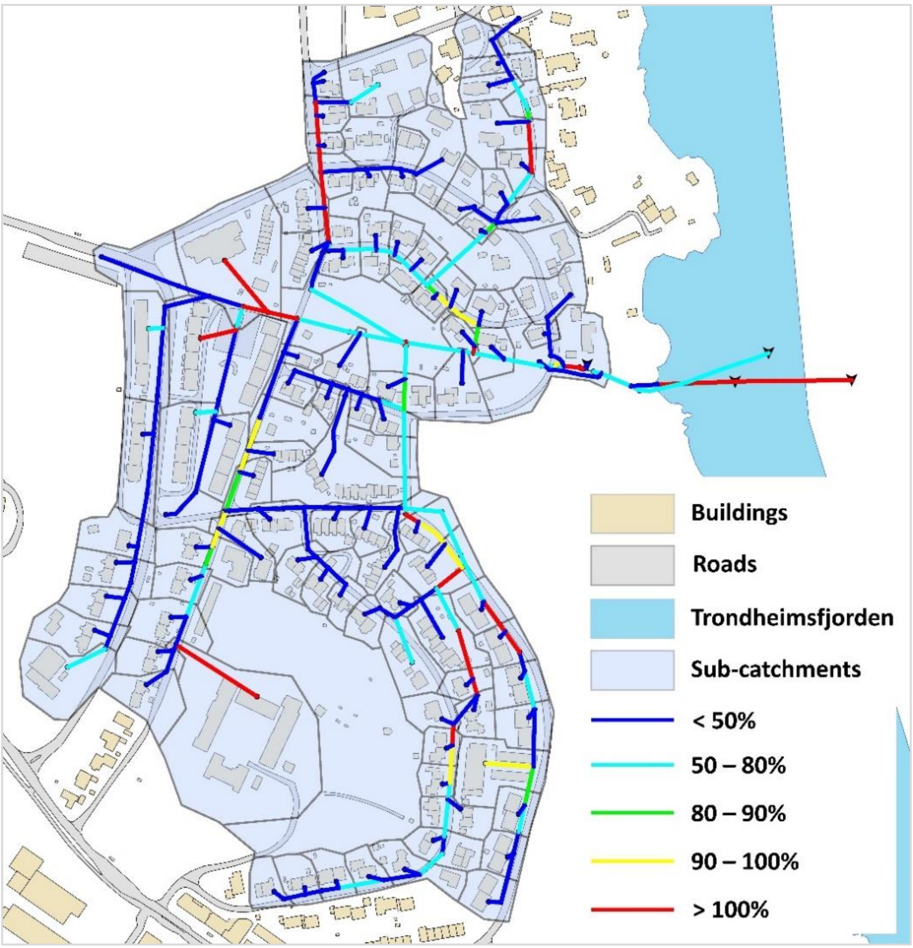


Figure 20: The capacity utilization of the network for scenario 0 for the simulations of the extreme rainfall event.

	Number of pipes				
Interval	0	Basin50	Basin100	LID50	LID100
< 50%	162	195	216	212	250
50 - 80%	46	32	17	21	0
80 - 90%	8	4	3	5	0
90 - 100%	11	4	3	3	0
> 100%	25	17	13	11	2
Sum	252	252	252	252	252

Table 10: The maximum capacity utilization of the network for the simulations of the extreme rainfall event.

As illustrated in the results above, is the capacity of several pipelines in the sewer system not reached during the simulations of the 20 years rainfall event. However, the results illustrating the risk of basement flooding from chapter 4.2.1, showed a critical water level in several manholes of the pipe system. Moreover, for the illustration of this situation, is the longitudinal profile plotted for a selection of pipelines, shown in Figure 21. This profile contains pipelines where the capacity is not reached and where the risk of basement flooding and surface flooding is obtained in the connected manholes. The pressure lines from the water level in the sewer system are illustrated with their respective colours. Moreover, the pressure line for the water level from the manholes has a steeper slope compared to the pressure line for the water level in the pipes, as illustrated in the figure below. Furthermore, this leads to backflow effects in the manholes and a critical level with respect to basement flooding or surface flooding is reached. Hence, the water discharge cannot reach the theoretical capacity of the sewer system, which was illustrated in Figure 20. The reason for the backflow effects in the manholes is related to energy losses. These energy losses may be related to losses due to changes in the flow direction, changes in elevation, and head losses at the inlet or the outlet from the manhole. In addition, does the water flow from different directions lead to losses in the kinetic energy (DHI, 2017d). The implementation of service pipes from the implemented stormwater practices to the connected sewer system increases this effect.

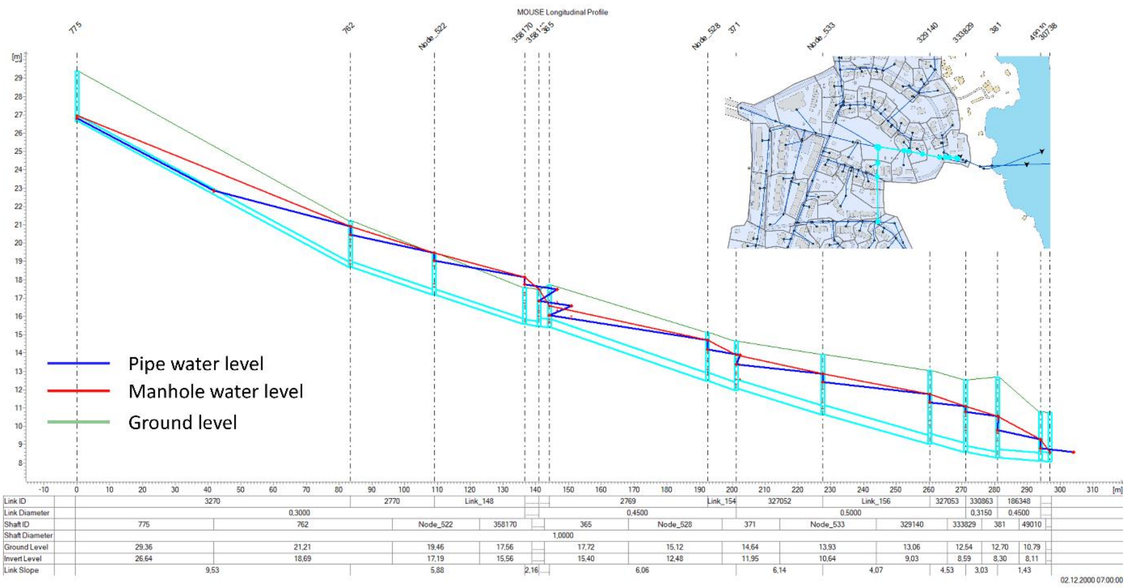
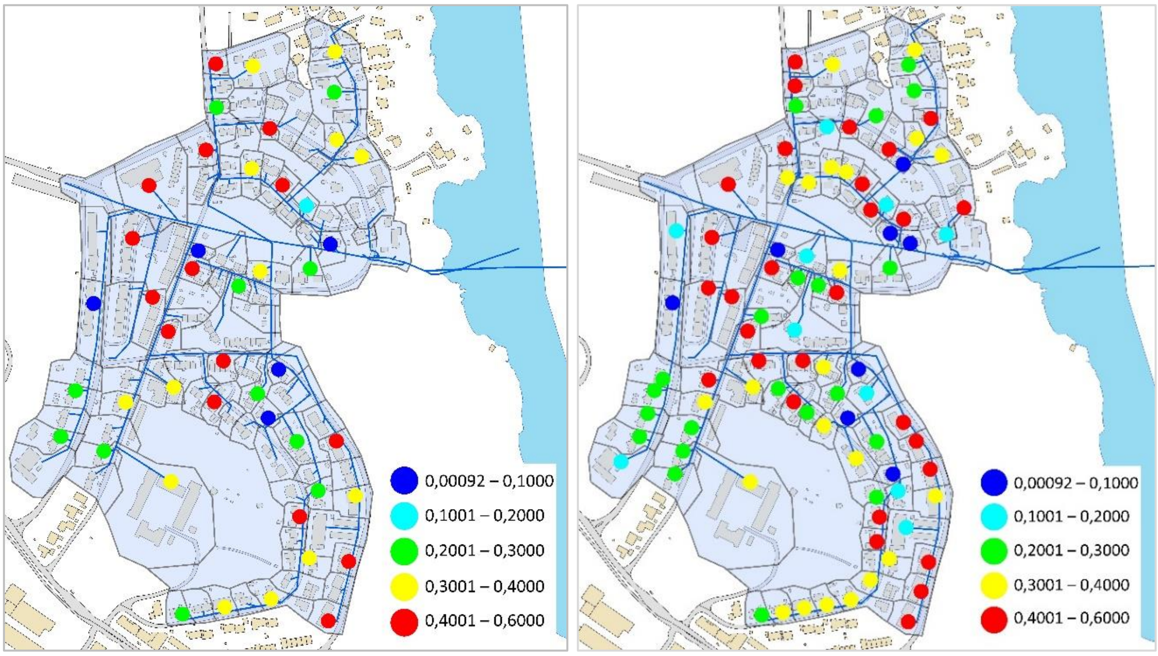


Figure 21: Longitudinal profile over a selection of manholes and pipelines, illustrating the different pressure lines.

4.3 Evaluation of design criteria and the effects of detention basins

In order to evaluate the design criteria of today's guidelines of stormwater management in Trondheim municipality, the water depth in the detention basins during a 20 years rainfall event is evaluated. The results for the simulations of the scenarios with 50% and 100% implemented detention basins are shown in Figure 22, where the remaining manholes of the sewer system are deleted for a better visualisation. As shown in these figures, the water depth varies between around 0 to 0,6 meters. Overall, these results indicate that the detention basins during a 20 years rainfall event are maximum half filled, as all of the detention basins are designed with a standard height of 1,2 meters.

Backflow in the detention basins was experienced during simulations with a standard elevation of 0,2 meters from the upstream to the downstream node of the service pipes. Moreover, this led to different results for the water depth in the detention basins for the scenarios of 50% and 100% implemented basins, which may be related to capacity problems of the connected sewer system. Lower water levels were experienced in the Basin100 scenario, which is a result of higher capacity of the connected sewer system due to the increase of detention basins compared to the Basin50 scenario. The evaluation of the service pipes was increased with 2 meters from the upstream to the downstream level of the connected node, in order to avoid backflow effects for easier evaluation of the water depth in the detention basins. Furthermore, this resulted in an increase of the capacity of the sewer system and an equal water depth for the detention basins in both scenarios, which is shown in Figure 22.



a) Scenario Basin50 b) Scenario Basin100

Figure 22: The maximum water depth (m) obtained in the detention basins for the simulations of the extreme rainfall event.

However, after the increase of the elevation of the service pipes, there is still a large variation in the water depth of the detention basins, which are further illustrated in Table 11. The results show a maximum water depth of 0,59 meters and a minimum water depth of 0,09 meters, for both scenarios. An overview of the distribution of the detention basins in relation to the water depth, are shown in the table below. Dimensions of the basins are done with the use of today's guidelines for stormwater management in Trondheim municipality. These guidelines for detention basins, which were further explained in chapter 2.3, use the imperviousness and the size of the catchment area as design criteria for the computations of the necessary detention volume. Based on this, the water depth was expected to be equal for each sub-catchment. With avoiding the backwater effects to happen, could this no longer be the reason for the difference in water depths. Furthermore, may the reason be related to the automatically computed imperviousness of the sub-catchments, which were further used for the calculations of the volume of the basins. Uncertainties related to these automatically tools should be taken into account. There is in addition some uncertainty concerning the constructions of the detention basins, which were designed with a rounded value. Moreover, this was done to ensure easier implementation of the detention basins in MIKE URBAN, which were manually done in the model. Furthermore, this may lead to wrong dimensioning for some of the detention basins. However, there is unlikely that this would lead to such a large difference related to the water depth in the detention basins, but some uncertainty should be taken into account related to the dimensioning of the basins.

Interval (m)	Number of basins (-)	Percentage compared to the total amount of basins (%)
0,00092-0,1	8	8,9
0,1001-0,2	10	11,1
0,2001-0,3	21	23,3
0,3001-0,4	22	24,4
0,4001-0,6	29	32,2
Sum	90	100

Table 11: Overview of the water depth in detention basins for scenario Basin100 for the extreme rainfall event simulations.

To be able to evaluate today's guidelines for stormwater management in the municipality of Trondheim, an overview of the water depth in the basins and their respective flow control and the reduced area was made, which are shown in Figure 23 and Figure 24. This was done for the scenario of 100% implemented detention basins and by simulations of the extreme rainfall event. The main purpose of this analysis was to investigate if there were any correlation between the water depth and some of the design criteria. A theory of smaller catchments with its corresponding low flow control leading to a low water level in the detention basins were investigated. Such an investigation could be helpful for the understanding of the obtained water depth, and to analyse the optimum size of the catchment area leading to a preferable performance concerning stormwater detention. However, the results illustrated in Figure 23 and Figure 24 shows no correlation between the water depth and the corresponding flow control or the reduced area. The non-existing correlation between the water depth in the detention basins and the design criteria may be related to the uncertainties in the model, which were explained above. To be able to evaluate the performance of the detention basins, more research should be done. Considering the results obtained in this thesis is the capacity utilization of the detention basins during an extreme rainfall event, not satisfactory.

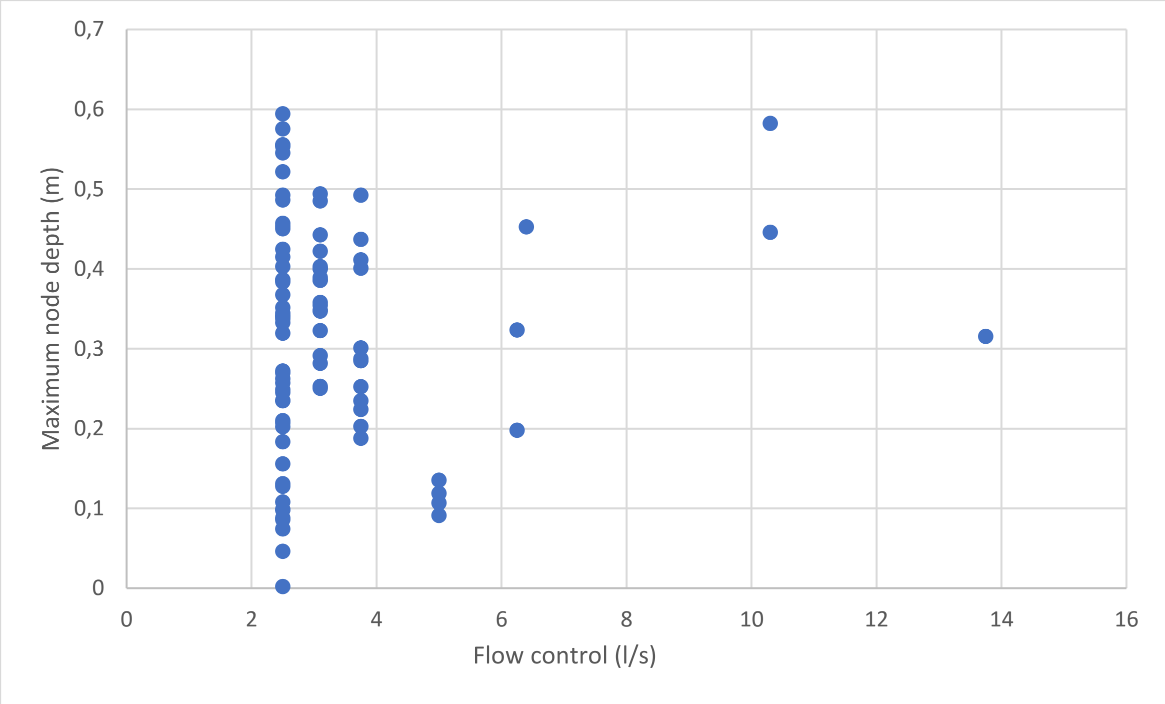


Figure 23: The distribution of the water depth in the detention basins in relation to the respective flow control for the extreme rainfall event.

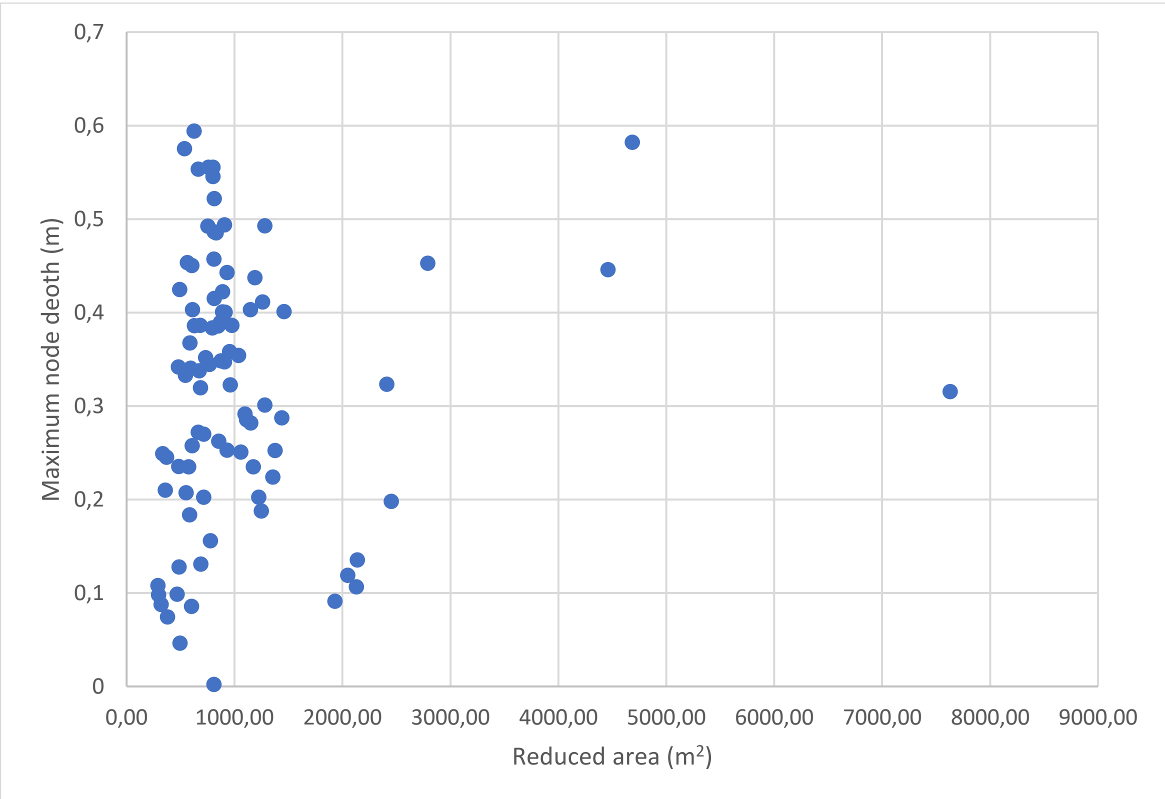


Figure 24: The distribution of the water depth in the detention basins in relation to the respective reduced area for the extreme rainfall event.

A maximum water depth leading to 50% of its capacity, indicates an over-dimensioning of the detention basins based on today’s guidelines for a 20 years rainfall event. A stricter flow control out of the detention basins should be considered to obtain a better optimisation of the volume of the detention basin and the capacity of the sewer system. In relation to an economical perspective, focusing on other solutions in order to optimise the capacity of today's sewer system, should be considered. Improvements of today's sewer system by implementing separate systems, would lead to a better capacity of the sewer system and reduce the risk of flooding. Moreover, this could be an option for the instalment of detention basins, and the two solutions should be further investigated in order to develop an optimal solution in an economical and hydrological long-time perspective.

5 Conclusion

In this study, the effects of different stormwater solutions with respect to CSO discharge and risk of flooding were evaluated using a model in MIKE URBAN, which was already calibrated and given from DHI in Trondheim. The second aim of the study was to investigate the effects of the detention basins and evaluate the design criteria of the guidelines for stormwater management in Trondheim municipality.

The results indicate a decrease in CSO discharges for the simulations of both the extreme rainfall event and the average year. The maximum CSO discharge is reduced with 5,5% and 24,8% for the respective scenarios of Basin50 and Basin100 for the simulations of the extreme rainfall event. Furthermore, this led to a CSO volume reduction of 5,2% and 4,4% for the two scenarios with implemented detention basins. Moreover, the relatively small reduction of the accumulated volume of CSO discharges is a result of an increase in the duration of the extreme event. The results from the simulations of the scenarios with implemented bioretention cells showed a significant reduction of the CSO discharge for both rain events. However, limitations related to the input parameters for the design of the bioretention cells in MIKE URBAN, affect the results for the simulations of the scenarios with LID as an implemented stormwater solution. Furthermore, this may question the hydrological performance of the implemented LID function to replicate the physical reality.

Another important evaluation in relation to the performance of the stormwater management solutions is their effects on the risk of flooding, which were evaluated in this study. For the situation of today’s scenario, 38,4% of the total amount of manholes in the model consists of a critical water level concerning basement flooding for a 20 years rainfall event. Implementation of stormwater practices led to a decrease in the risk of flooding, for both basement flooding and surface flooding. However, results show several pipelines in the sewer system where the water discharge cannot reach the theoretical capacity of the sewer system. This is related to energy losses in the manholes, which causes backflow effects and risk of flooding.

A further investigation of the performance of the detention basins was done in order to evaluate today's guidelines of stormwater management in Trondheim municipality. The results from the simulation of the extreme rainfall event show a maximum water depth of approximately 0,6 meters in the basins. This water depth corresponds to 50% of the maximum capacity of the basins. In comparison, a minimum water depth of 0,02 meter was obtained by the same simulation. There was not found any correlation between the water depth in the detention basins and the design criteria related to the reduced area and flow control.

Taken together, detention basins have some effects on the CSO discharge, although small effects for the average year with a reduction of 6,2% and 8,1% for the respective scenarios of Basin50 and Basin100. However, the implementation of detention basins decreases the risk of flooding during an extreme rainfall event. Concerning the general low water depth obtained in the basins, further investigation should be done in order to evaluate the performances of detention basins. Moreover, other solutions should in addition be evaluated in a long-term economical and hydrological perspective.

6 Further work

To be able to evaluate the effects of LID practices, other methods should be investigated for the implementation of the solutions in MIKE URBAN. The use of the catchment-based method should be considered for the implementation of LID controls if the kinematic wave runoff model is supported. A division of the different layers in the bioretention cell, which are available in the catchment-based approach, may give results that illustrate the physical reality more satisfactorily.

The evaluation of the performances of detention basins should be further investigated and seen in relation to other solutions for an increase in the capacity of the sewer system. Solutions including separation of the sewer system or an implementation of traditional stormwater solutions combined with LID controls should be investigated. An evaluation of the average year performances of the basins should be further analysed in terms of the time of use and grade of capacity utilization. An overview of the number of events where the detention basins are in use during a year could be helpful for the evaluation of their performances during an average year.

The low differences of CSO reduction for the Basin50 and Basin100 scenarios for an extreme storm event should be further investigated. A greater focus on the placement of the detention basins could produce interesting findings related to the capacity of the sewer system and the change of CSO discharges. The increase of the volume of CSO discharge from 50% to 100% implementation of detention basins, should be further investigated. A comparison of implementing detention basins far away from the overflow weir and close to the overflow weir could give interesting results on their performances concerning the duration of the events, which in this study has shown to have a direct effect on the volume of CSO discharge.

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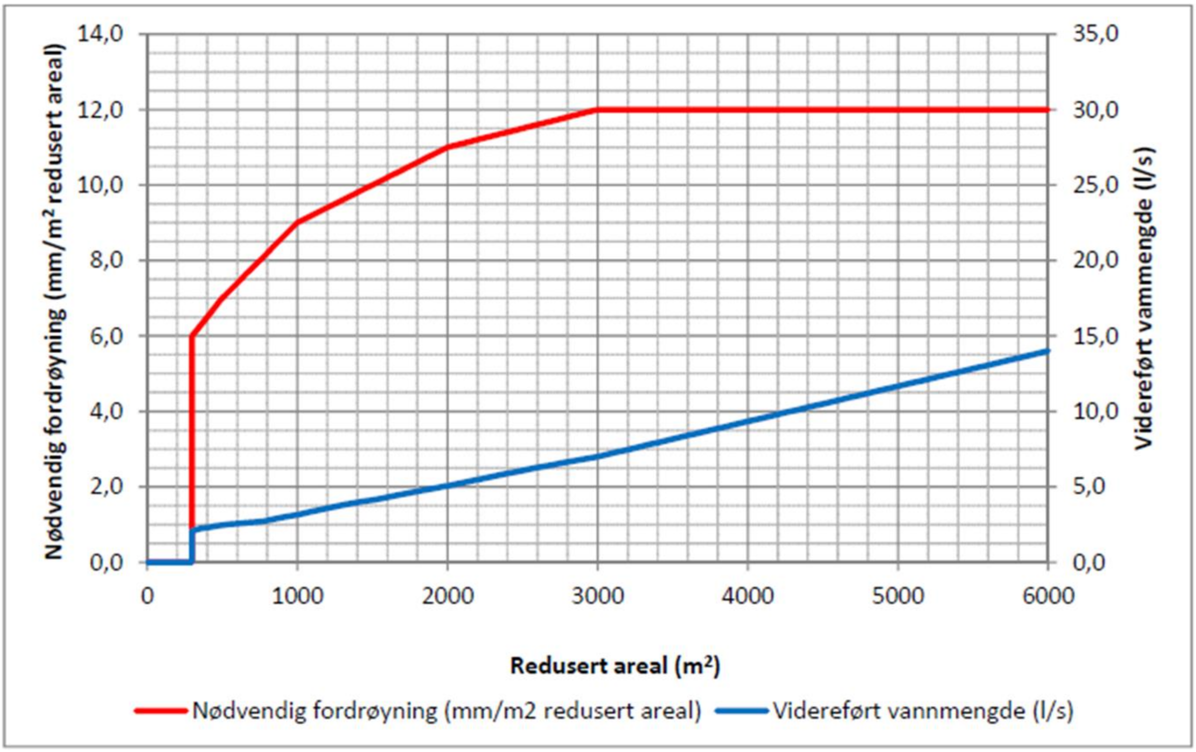
Appendix A – Guidelines, Trondheim municipality



Minimum requirements for detention and maximum flow control for combined sewer systems.

Returperioder(år); Nedbørintensitet(l/s*ha)
68862 TRONDHEIM - VOLL MOHOLT TYHOLT
Periode: 1967 - 2009
Antall sesonger: 39

År	1 min.	2 min.	3 min.	5 min.	10 min.	15 min.	20 min.	30 min.	45 min.	60 min.	90 min.	120 min.	180 min.	360 min.	720 min.	1440 min.
2	163,1	130,5	115,8	94,5	69,1	56,4	47,8	37,2	29,2	24,5	19,0	16,2	13,1	9,3	6,3	4,0
5	224,3	187,3	167,7	135,8	93,0	72,5	60,2	45,7	36,3	30,9	23,9	21,1	17,0	12,0	8,0	5,0
10	264,8	225,0	202,1	162,3	108,8	83,2	68,4	51,3	41,0	35,1	27,2	24,4	19,6	13,8	9,1	5,6
20	303,7	261,1	235,1	188,2	124,0	93,5	76,3	56,7	45,5	39,2	30,4	27,5	22,1	15,5	10,1	6,3
25	316,0	272,5	245,6	196,4	128,8	96,8	78,8	58,4	47,0	40,5	31,4	28,5	22,9	16,0	10,5	6,5
50	354,0	307,8	277,8	221,7	143,7	106,8	86,5	63,6	51,4	44,4	34,5	31,6	25,4	17,7	11,5	7,1
100	391,7	342,8	309,8	246,9	158,4	116,7	94,1	68,8	55,8	48,4	37,5	34,6	27,8	19,3	12,5	7,7
200	429,3	377,8	341,8	272,0	173,1	126,7	101,7	74,0	60,2	52,3	40,6	37,7	30,2	21,0	13,5	8,3

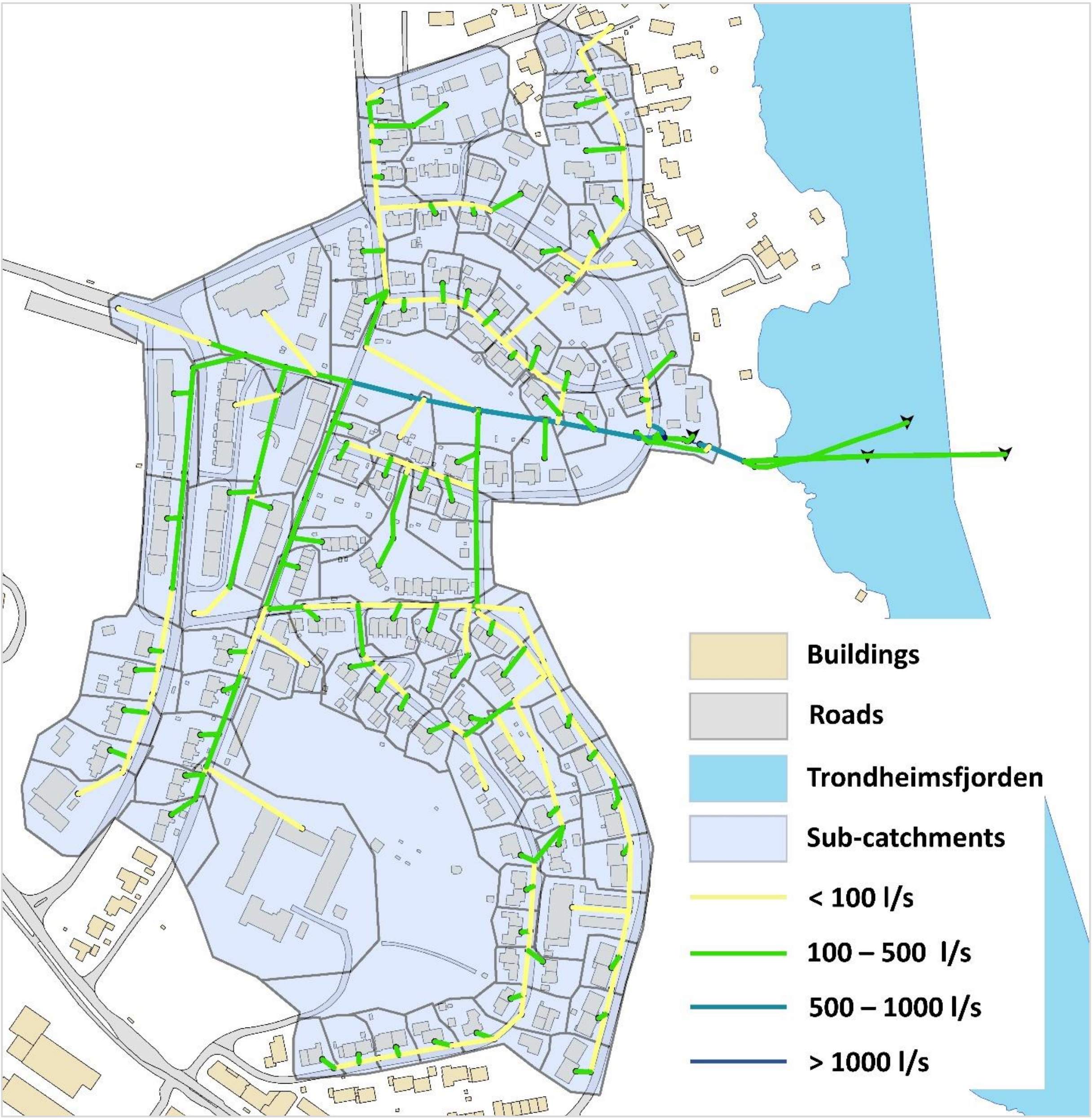
IDF curve for the area of Voll – Moholt – Tyholt, Trondheim.

Appendix B – Risk of basement flooding

	Scenario				
Interval (m)	0.	Basin50	Basin100	LID50	LID100
0,001 - 0,9	98	122	152	149	159
0,9 - 1,5	23	21	6	9	0
1,5 - 2,0	20	10	1	1	0
2,0 - 2,5	16	6	0	0	0
2,5 - 2,8	2	0	0	0	0
Number of critical nodes	61	37	7	10	0
Percentage of critical nodes	38,4%	23,3%	4,4%	6,3%	0,0%

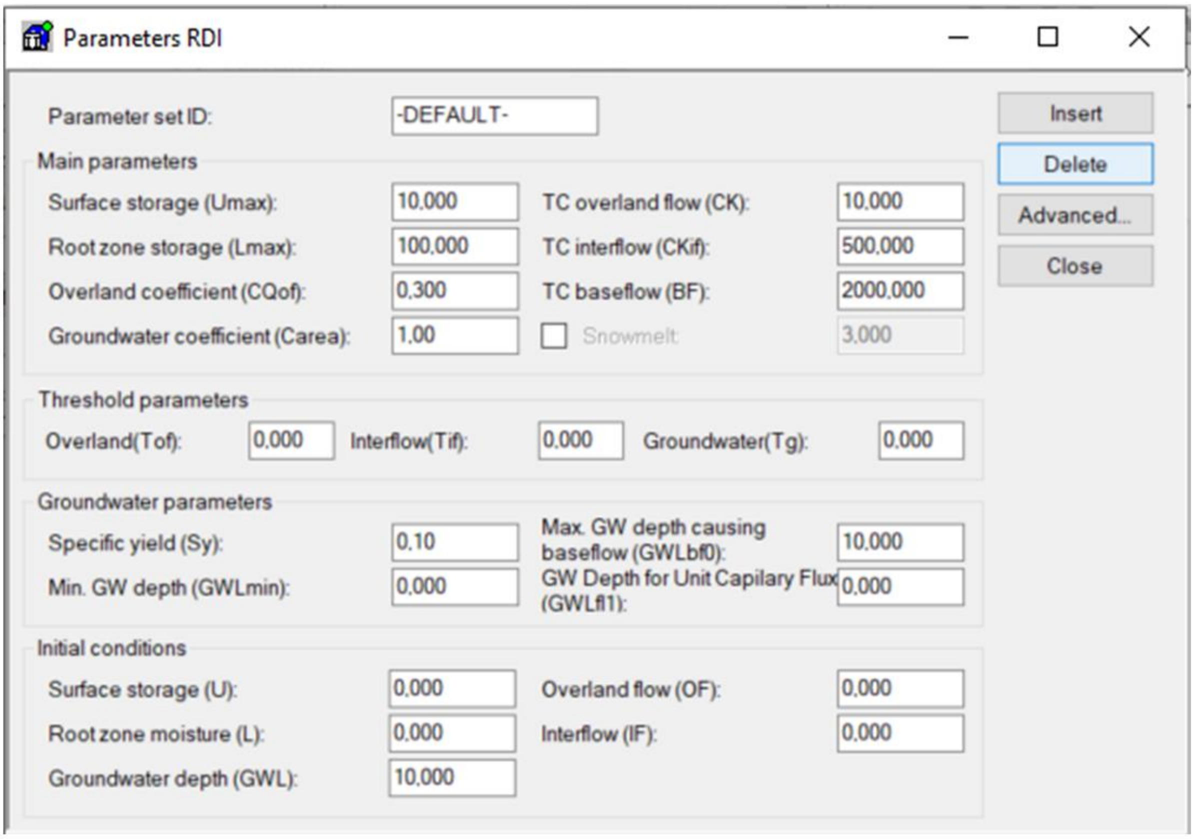
Results from the simulation of the extreme event with a return period of 20 years, including a factor of 1,2 for climate projections.

Appendix C – $Q_{manning}$, Basin50 and Basin100

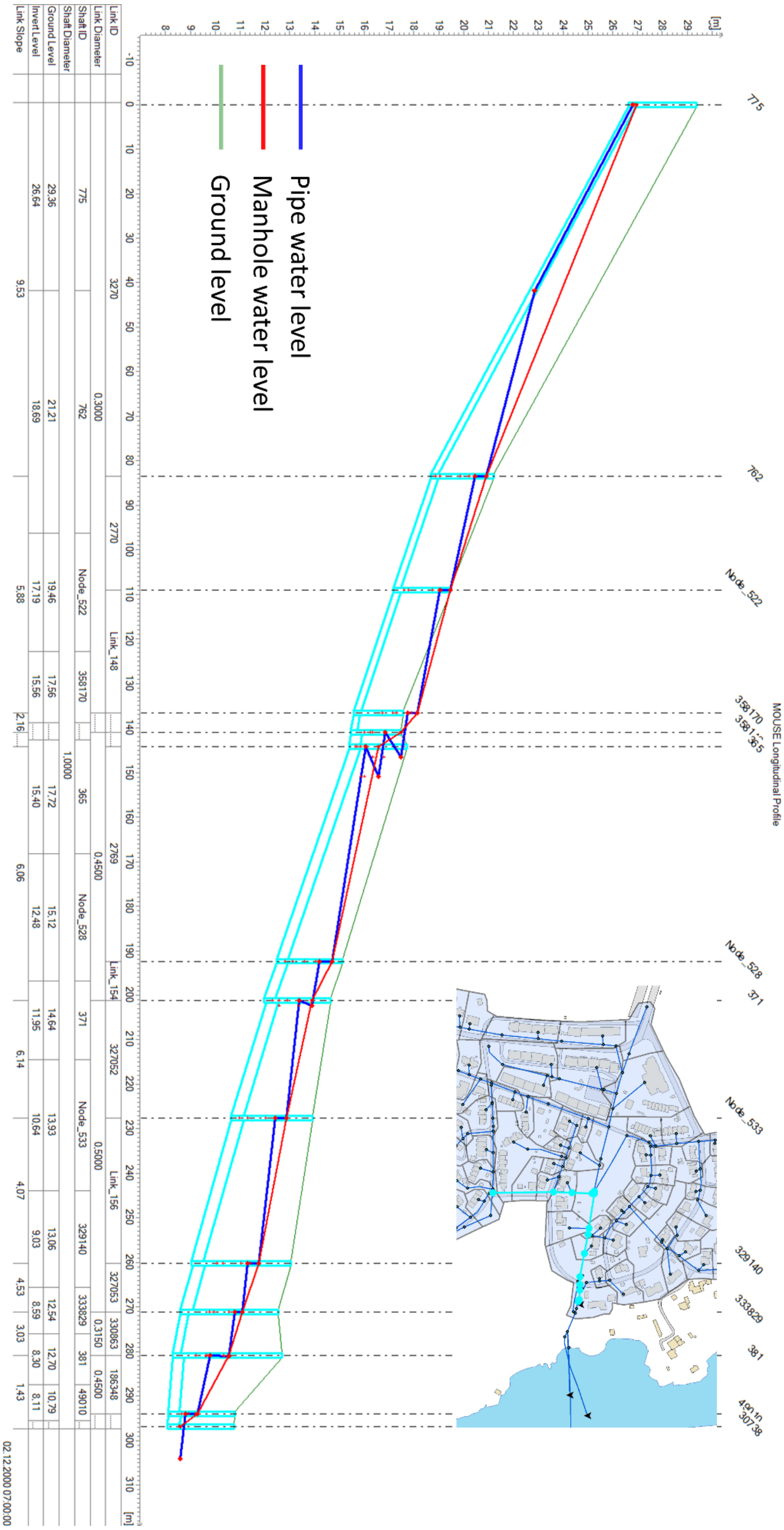


The theoretical maximum capacity (l/s) for the scenarios with implemented detention basins.

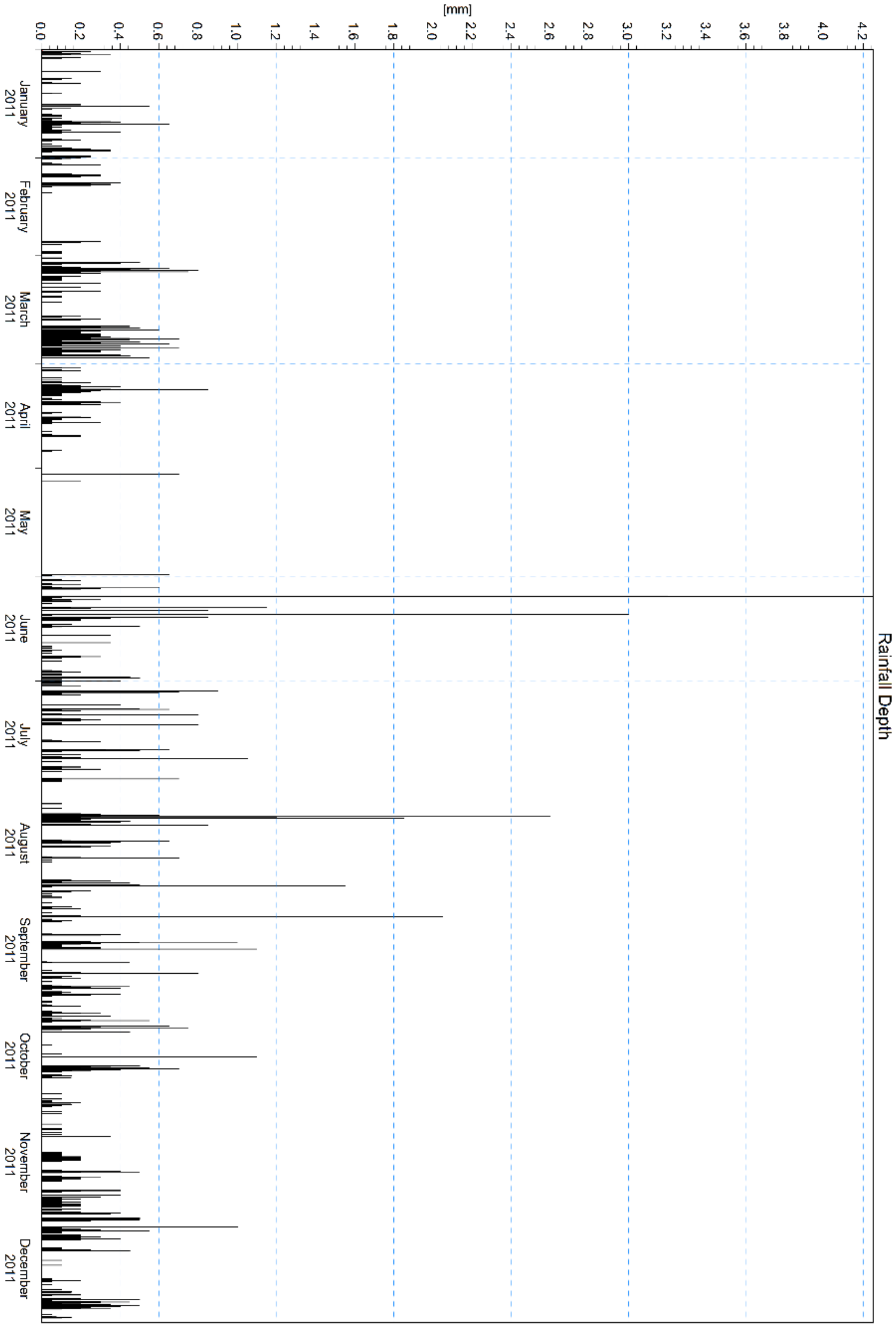
Appendix D – RDI data



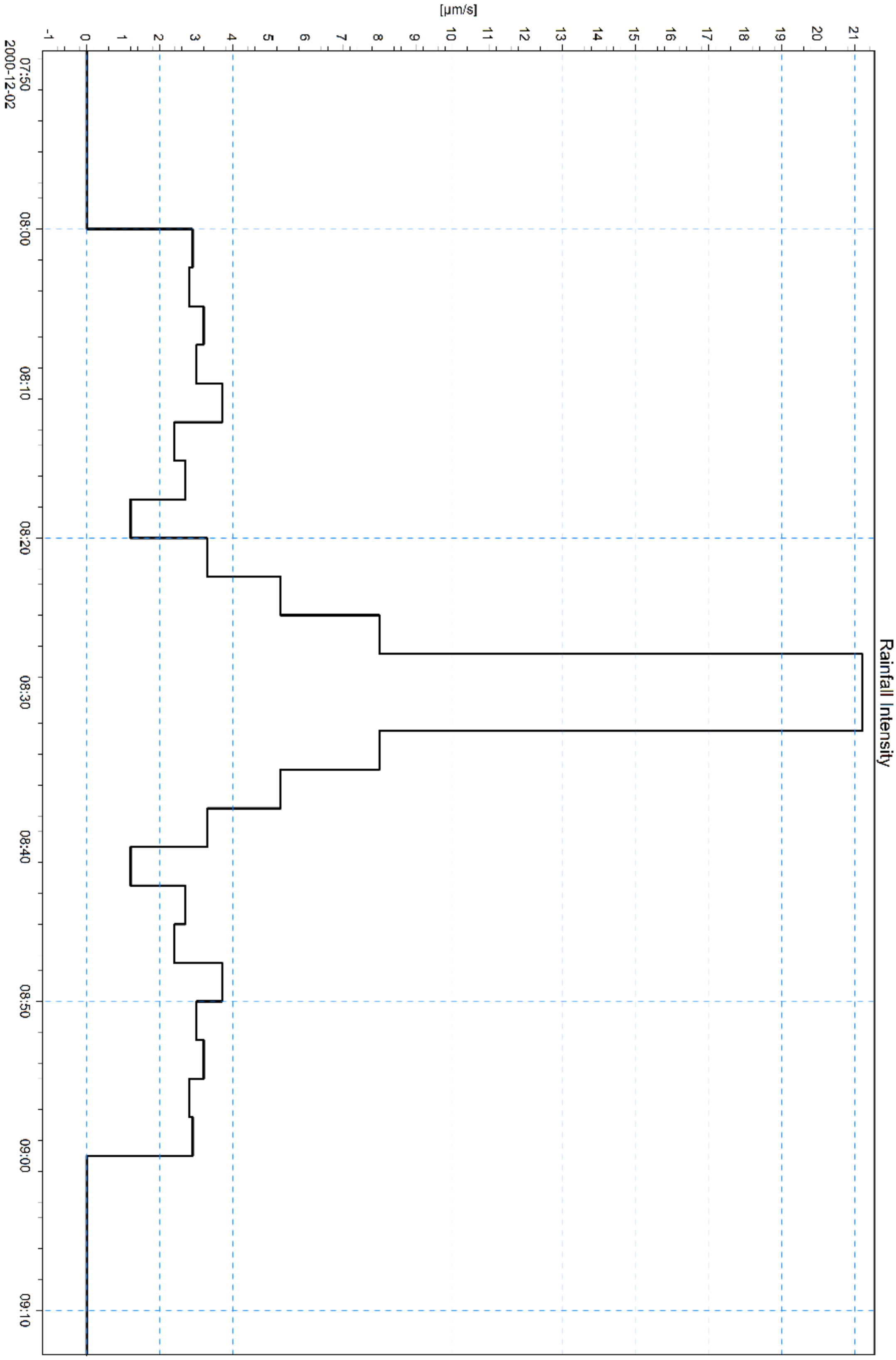
Appendix E –Longitudinal profile



Appendix F – Rainfall data, 2011



Appendix G – Rainfall data, extreme event



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