Master's thesis

NTNU
Norwegian University of Science and Technology
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An Evaluation of the Activation Frequency of Detention Solutions in Current and Future Climate

Master's thesis in Civil and Environmental Enginnering
Supervisor: Tone Merete Muthanna
Co-supervisor: Thea Ingeborg Skrede
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Trondheim, June 11, 2022

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Abstract

The pressure on stormwater management systems has increased due to climate change, urbanization, and increased use of impervious surfaces. Traditionally stormwater has been managed by combined sewer systems, but the ageing infrastructure is inadequate in handling the increased frequency of precipitation, rainfall intensity and subsequent stormwater volumes. Today, stormwater management approaches commonly aim to imitate natural hydrology to address the uncertainties of climate change and urbanization.

In Norway the 3-Step approach to stormwater management is the industry standard, focusing on three steps of respectively infiltration, detention, and safe stormwater conveyance. The aim of this study is to evaluate the capacity of step 1, retention in the form of infiltration, and the activation frequency of step 2, detention. A mass balance model was built to explore the relationship between rainfall patterns and system performance. A full-scale combined infiltration and detention solution was developed in 2019 in the city centre of Trondheim, Norway was used as a case in this study.

The activation frequency of the detention solution was found to be a result of the precipitation pattern, and it increased for the future scenario when climate change induced more frequent, higher peak flows. The performance of the second step as part of an overall stormwater strategy was found to be dependent on site specific properties and design guidelines, and this study argues to make local strategies that to a larger extent emphasize on local conditions, for cities to develop sustainable stormwater managements.

Keywords – 3-Step approach to stormwater management, Activation frequency, Detention, Mass balance model, Sustainability

Sammendrag

Presset på dagens overvannshåndtering har økt på grunn av klimaendringer, urbanisering og økt bruk av tette overflater. Tradisjonelt har overvann blitt håndtert ved bruk av lukkede rørsystemer, men som ikke lenger er dimensjoner for å håndtere den økte nedbørsfrekvensen, -intensiteten og den påfølgende økte avrenningen. Overvannshåndteringen i dag har som mål å imitere den naturlige hydrologien for å møte utfordringene knyttet til klimaendringer og fortetting.

I Norge er 3-trinns strategien den gjeldende overvannstrategien, delt opp i tre trinn med fokus på henholdsvis infiltrasjon, fordrøyning og sikre flomveier. Målet med denne studien er å evaluere kapasiteten til trinn 1, infiltrasjon, og aktiveringsfrekvensen til steg 2, fordrøyning. En massebalansemodell ble bygget for å se på forholdet mellom nedbørsmønstre og systemytelse. Ef fullskala kombinert infiltrasjons- og fordrøyningsanlegg ble bygget i Trondheim sentrum i 2019, og er brukt som en case i denne studien.

Aktiveringsfrekvensen til fordrøyningssystemet var avhengig av nedbørsmønsteret, og den økte i fremtiden når klimaforandringer bidro til hyppigere og høyere avrenningstopper. Studien viser at ytelsen til steg 2, som en del av en overordnet overvannsstrategi, er avhengig av stedsegenskaper og dimensjoneringskrav, og vil derfor argumentere for at det bør lages lokale strategier som i enda større grad baserer seg på lokale forhold, for at byer skal utvikle en bærekraftig overvannshåndtering.

Thesis Structure

This thesis is written as a manuscript based on the requirements and structure of a research article, unlike a traditional Master's thesis at NTNU. It is to the author's wish to promote and make available the research work carried out for this thesis. As of that it is also written in English, with a Norwegian summary. Description of the research format used as a guide, is found in appendix A1. This thesis is deliberately more extensive than a traditional research article, to cover the work done for this Master's in a detailed manner for censorship. The appendix in this thesis presents parts of the study that are not included in the final manuscript, and provides a more detailed illustration of some of the presented figures and graphs.

Contents

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List of Abbreviations

BMP Best Management Practice

IDF Intensity-Duration-Frequency

LID Low Impact Development

S3SA 3-step approach to stormwater management

SUDS Sustainable Urban Drainage Systems

SUWM Sustainable Urban Water Management

1 Introduction

The hydrological characteristics in cities are changing, where climate change and urbanization are the two most influential drivers (Arnone et al., 2018). Climate change has a significant impact on the precipitation pattern, and there is an increase in frequency and intensity of heavy precipitation (IPCC, 2021). Urbanization gives denser built and populated cities, and an increased use of impervious surfaces (Sörensen et al., 2016). As a consequence, cities experience reduction of infiltration, increased runoff volumes, and higher peak flows (Kourtis and Tsihrintzis, 2021), and they remain vulnerable to short-duration, intense rainfall events, known as cloudbursts, that can cause pluvial flooding (Rosenzweig et al., 2019). The cities characteristics such as climate, topography and land use, combined with their stormwater management practices, decide on the outcome of an intense rainfall event (Westra et al., 2014). Since the 20th century the traditional approach has been to construct larger capacity pipe systems and subsurface storage facilities, to collect and transport stormwater runoff from urban areas as quickly as possible to receiving water bodies (Eaton, 2018). Haghighatafshar et al. (2020) would argue that this approach intrinsically is causing the challenges of flooding. The traditional approach has usually been designed with low probability of failure, oversized to handle unforeseen threats and risk management for infrastructure adaptation to climate change (Kim et al., 2017). With respect to operational, environmental, social and economical constraints there has been a growing concern with the traditional approach, which has called for a shift towards sustainable urban water management (SUWM) (Marlow et al., 2013).

For cities to manage the uncertainties of climate change and urbanization there is a need for them to enhance their resilience (Rosenzweig et al., 2019). Park et al. (2013) suggested a transition from a traditional, risk-based design paradigm to a resilience based. The traditional design paradigm is typically based on rainfall intensity which derives from statistical analysis of rainfall records, also known as intensity-duration-frequency (IDF) curves (Yazdanfar and Sharma, 2015). In addition, the design often include a safety factor that consider capacity and function failure (Kim et al., 2017), or future change in climate (Paus et al., 2015). This design paradigm is challenged, as not even the best models have the precision to estimate future extreme weather, population growth, social demographics or urban form (Shortridge et al., 2017). Kim et al. (2017) described the approach as a "fail-safe"-strategy and advocated for the resilience based "safe-to-fail"-strategy. A "safe-to-fail" strategy centers the design decision on urban resilience which is the urban systems ability to respond to disturbance and experience minimal disruptions and rapidly return to desired functions (Meerow et al., 2016). This is done through minimizing the consequences of the extreme event rather than minimizing the probability of damages (Park et al., 2013). In the "fail-safe"-strategy Qin et al. (2013) found that upsizing the existing drainage system to relieve the increasing pressure from climate change and urbanization has proved to be costly, impractical, and unsustainable, especially for urbanized areas.

The change to enhance sustainable development SUWM aims to restore the natural hydrological cycle, and is known in other terms as Best Management Practices – BMPs; Low Impact Development – LID; and Sustainable Urban Drainage Systems – SUDS (Fletcher et al., 2015). In this thesis, the term SUDS is selected to describe the use for practices and systems that use and enhance natural processes. In the Scandinavian cities, as in other cities, there have been a shift from the traditional pipe network towards implementation of SUDS. In Malmö they solve design rain problems, through open solutions and implementation of SUDS in the suburbs, while Copenhagen focus on extreme rain, by allowing 10 cm of flooding above ground level in a 100-year storm and aims to quickly export the stormwater to the sea (Haghighatafshar et al., 2014). The different implementations, are supported by Dawson et al. (2020) who found that to ensure interoperability between different infrastructure systems, different actions were required.

In Norway, SUDS is applied through the standard stormwater management approach, the 3-Step approach to stormwater management (S3SA). It is highly accepted as the industry standard for climate adapted stormwater management, and for its adaption to climate change and urbanization impacts (Skrede et al., 2020). The S3SA was established by Lindholm et al. (2008), and the approach was classified into three steps, motivated by creating a better urban environment and reducing the use of the pipeline networks through: (1) infiltration for light rainfall; (2) retention and detention for moderate rainfall and subsequent infiltration and slow release downstream; and (3) conveyance of stormwater in safe floodways for heavy rainfall. The goal for each of the steps are (1) to maintain the water balance; (2) reduce damage on the pipe network; (3) reduce damage on the surface. Each of the steps has a clear purpose and design criteria, but to the author's knowledge there is little research on the intersection between the steps, and on the steps as a coherent system. Norsk Vann (Norwegian Water) (2020) have commented on the need to have a coordinated strategy from within the catchment area to the recipient.

The second step in the S3SA is built on-site, and due to the little research on connection between the steps, the contribution of the second step to the performance of the overall stormwater management is unclear. In the second step, SUDS can be underground detention or retention storages, retention and detention ponds, rain gardens and green roofs among others. Among the Norwegian municipalities the second step is commonly designed for a rainfall event with a return period of 20- to 25-years. Despite potential green solutions, in practice the second step is often implemented as large, underground concrete basins which is cost, CO_2, area and terrain demanding. There is emerging a shift in the approach to the second step within the Norwegian municipalities. Early in 2022 the Municipality of Bodø approved a stormwater plan where they considered the second step in the S3SA to be less important than the first and third step, due to local infiltration limitations in the ground as well as close proximity to the ocean (Bodø Kommune (The Municipality of Bodø), 2022). Today, Norwegian municipalities are facing an increased annual precipitation, more frequent and intense heavier rainfalls and more frequent and larger pluvial floods due to the expected change in climate (Hanssen-Bauer et al., 2017); an increased need for investments in the urban drainage infrastructure (Mikkelsen, 2021); and stormwater management approaches, both traditional and SUDS, that has a large carbon footprint due to high use of concrete, in pipes and basins (Wang et al., 2017).

As part of a shift towards resilient stormwater management, this study will specifically evaluate the role of step 2, detention, and a case study is used to study the function of the second step in the S3SA. The selected case is a full-scale combined infiltration and detention pilot, built in 2019 in Trondheim, Norway. Specifically the objectives of this thesis are formulated in the following points:

1. How can a combined infiltration and detention solution be modelled using a mass balance approach?

2. What is the activation frequency of the detention step for current and future climate scenarios?

3. Critically evaluate the need for the second step in the S3SA.

There are some limitations to this thesis: it only considers the quantity and not the quality of the stormwater runoff, and it will not be carried out calculations to quantify the results, as a Life Cycle Analysis or carbon footprint, but already existing literature on the subject will be used for the discussion.

2 Method

The overall goal of this study is to evaluate the capacity of step 1, retention in the form of infiltration, and the activation frequency of step 2, detention, in the S3SA. To do this, a mass balance model was built, and calibrated to a full-scale pilot in Trondheim. From the pilot, information on system performance of the infiltration pipes was lacking, hence a laboratory experiment on the exfiltration properties was carried out as part of this study. The model was applied to a selection of scenarios to demonstrate the applicability of the model, and to further evaluate the activation frequency of detention basins. Figure 2.1 illustrates the method in a flowchart, where subsection 2.1 elaborates on the site and pilot, subsection 2.2 describes the experimental set-up and elaborates on the exfiltration functions, subsection 2.3 describes the mass balance model, while subsection 2.4 elaborates on the selection and data for the scenarios applied to the model.

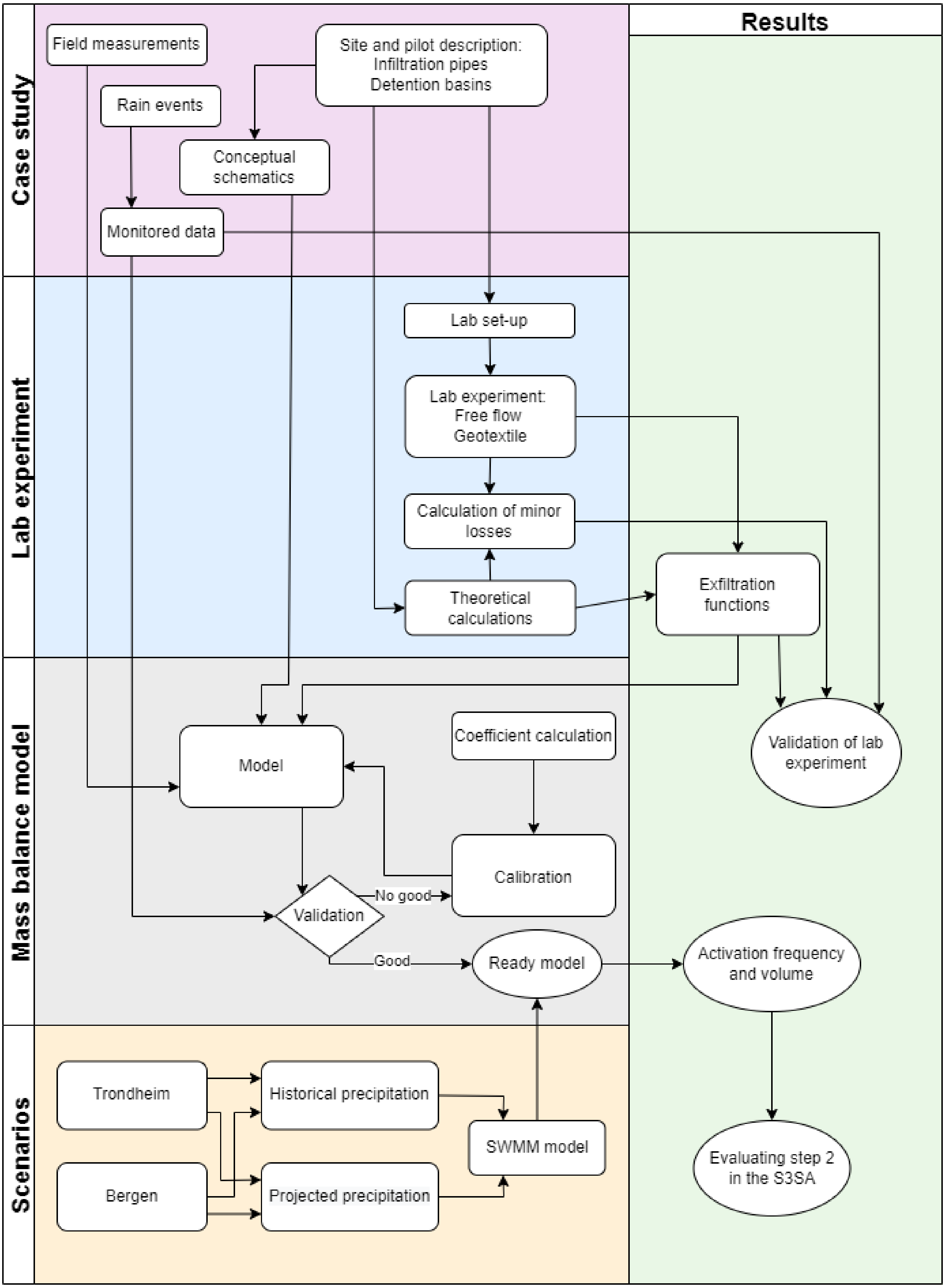


Figure 2.1: An overview of the method illustrated in a flowchart.

2.1 Case description

The full-scale combined infiltration and detention solution was built in 2019, and is located in the city centre of Trondheim. Trondheim is one of Norway's larger cities, located in mid-Norway with a coastal climate. The pilot was designed with a 20-year return period rainfall, with an additional safety factor of 1.2 to take future climate change into consideration, according to the water and sanitary norm for the Municipality of Trondheim. The size of the detention basins was reduced to 72 % of the original size as it was assumed that in combination with infiltration the need for storage volume would be reduced. One of the goals for the pilot was to relieve the already existing pipe network, when a new town square was built. Downstream, the pilot is connected to the existing combined sewer network, while the area upstream is isolated from the already existing pipe network. In other words, it is only the precipitation within the catchment area of the pilot that contributes to the pilot. The catchment area is to a large extent delimited by the town square, with a total area of 0.63 ha. Most of the city is situated on clay soils and marine depositions (NGU, 2022), but prior core drillings on site by (Sagli, 2020) conclude on the soil likely to consists of homogeneous materials of sand and silt, and backfill materials. An infiltration test carried out by (Multiconsult, 2018) assumed an infiltration rate of 0.001 m/s. Geographical information is provided in figure 2.2.

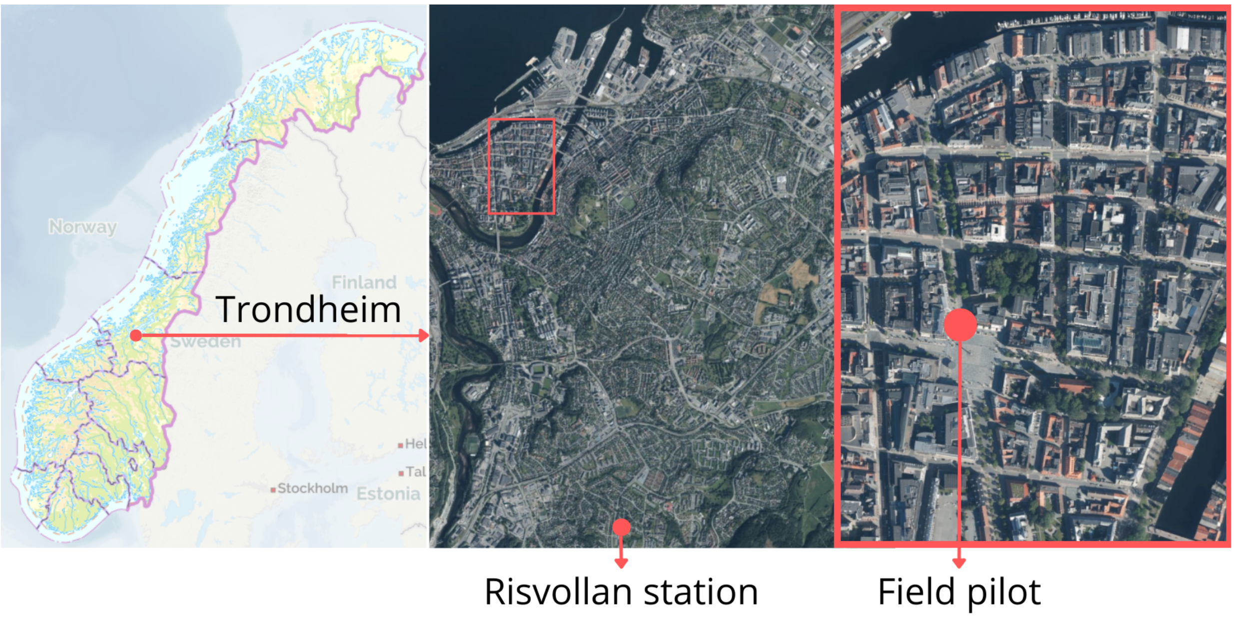


Figure 2.2: Geographical information on the site of the field pilot. Maps retrieved from Norgeskart (2022).

The full-scale pilot was built as a hybrid solution combining the processes of infiltration and detention. The concept is to first activate the infiltration system during a precipitation event, and later to activate the detention basins if the infiltration system is insufficient in managing the incoming runoff volumes. In the infiltration system, the stormwater is led through infiltration pipes, exfiltrated to the trench by the pressure head in the system and then infiltrated to the ground. Figure 2.3 illustrates the conceptual schematics of the pilot. In this thesis, the term infiltration pipes is used to address the pipes, their exfiltration properties and the following infiltration to the ground. The pilot was constructed of four infiltration pipes with a diameter of 160 mm. For the infiltration pipes, PP SN16 was selected, instead of traditional drainage pipes with a lower ring stiffness. The infiltration pipes were to face a higher load at the town square from the surrounding trench and traffic, than the traditional pipes could endure. Perforations where then drilled to the infiltration pipes, 300 perforations per meter with a diameter of 8 mm evenly distributed 12, 3, 6 and 9 o'clock as shown in the lab set-up in figure 2.4b. The lengths of the infiltration pipes are respectively 16 and 24 m, and they were placed with an inclination of 10 ‰. The pilot was built with three detention basins with a circular cross-section of 2000 mm with sloped sides assumed to be cords in the lower half, with a length of 11.85 m, and an inclination of 8.4 ‰. Data measured by Sagli (2020), indicated a constant water table in the detention basins of 1.5 cm, supported by a visual inspection that shows stormwater in the basins even after extended dry periods. The basins are activated when water flows into the basins, and subsequently water will flow out of the basins and a changing water level will be monitored in the downstream manhole. For a more detailed description on the pilot see (Sagli, 2020). Appendix A2 illustrates the detailed constructional design of the full-scale pilot.

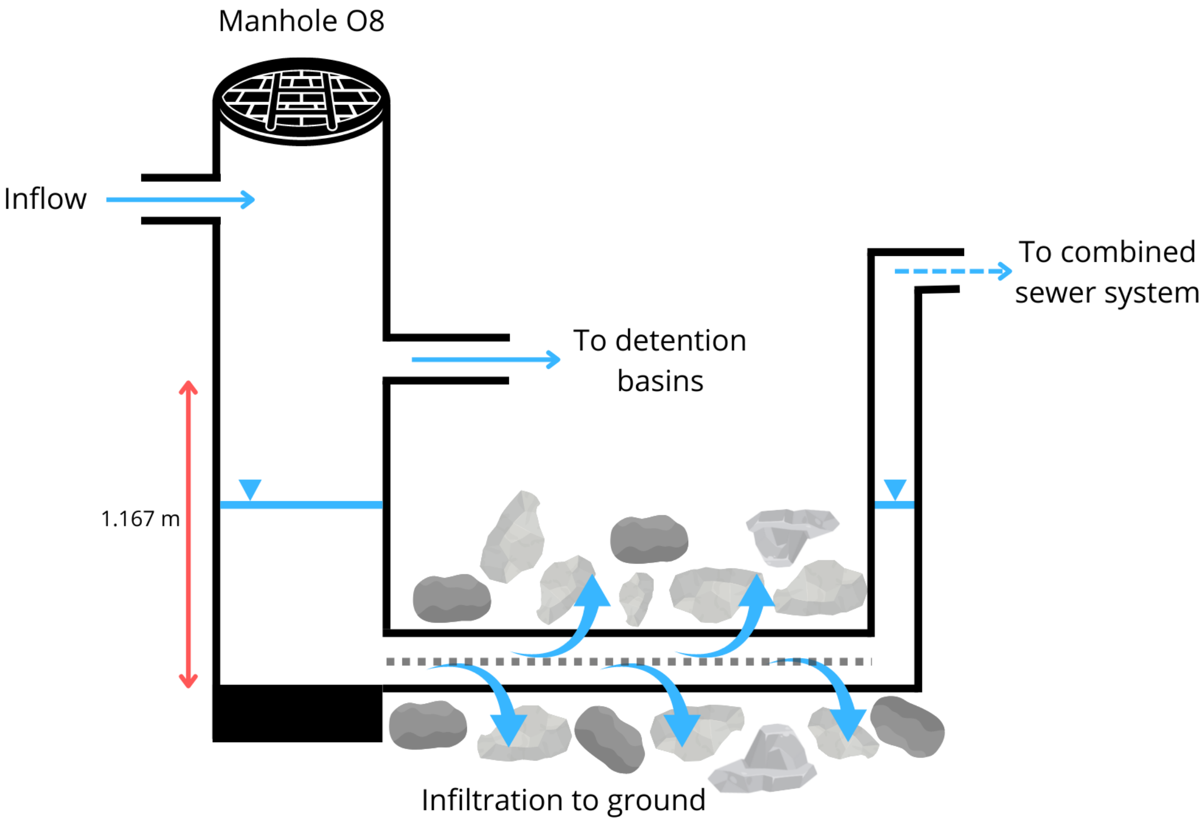


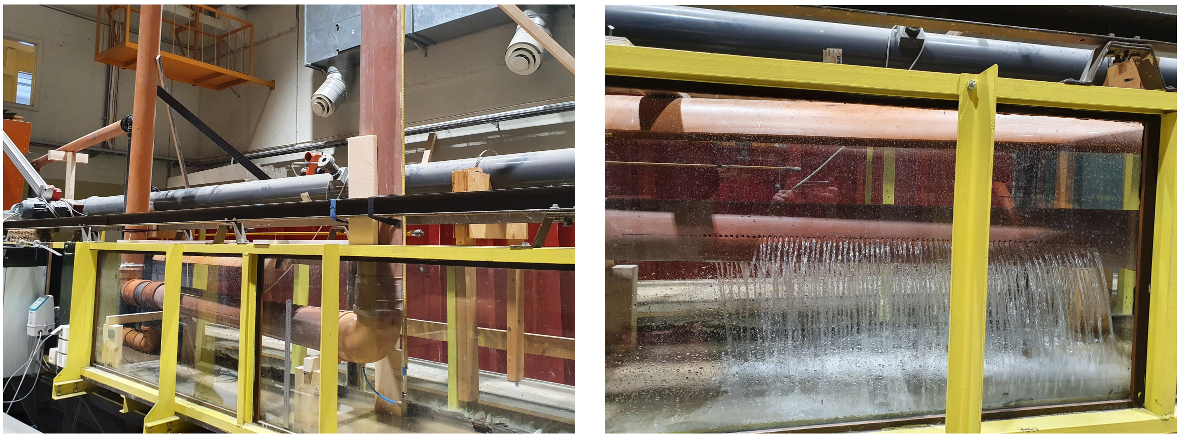
Figure 2.3: A conceptual illustration of how the combined infiltration and detention system works. The stormwater is sent to the combined sewer system if the detention basins go full.

To verify relevant measurements of the pilot, the height difference between the inlet to the infiltration pipes and detention basins was measured on site by a laser meter, Leica DISTO D1. The measured height difference was 1.167 m compared to 1.19 m stated in the the constructional design. The groundwater table was 1.4 m below the infiltration pipes, at 3.8 meters above sea level.

The system has been monitored since early 2020. Measures relevant for this study were the water flow in to the system measured in manhole O8 upstream of the pilot, the groundwater level, and the water level measured in the manhole downstream the pilot, where outflow from the detention basins is passing. Since the start of monitoring, the detention basins have only been activated twice: the 18th of July and 8th of August 2020. The 18th of July it rained 10 mm in 30 minutes corresponding to a 2-year rainfall event, following some previous days with rain. The 8th of August it fell 12 mm in 20 minutes approximately corresponding to a 10-year rainfall event, which followed a dry period with fewer, minor rain events.

2.2 Exfiltration functions

A hydraulic model was built to validate the exfiltration properties of the infiltration pipes in the full-scale, field pilot. The aim of the lab experiment was to produce exfiltration functions from a flow vs. head curve, by measuring heads at different flows through the pipe in the lab, and then to make a polynomial regression based on the measured, averaged values. Figure 2.4 shows photos from the experimental set-up.



(a) Lab set-up **(b)** Free flow when Q = 4.46 L/s

Figure 2.4: Photos from the lab

For the experiment, a smaller section of the field pilot was replicated with some adjustments for lab purposes. The infiltration pipes in the field pilot are respectively 16 and 24 m long, but 1 m of the infiltration pipe was considered sufficient for replication to reduce the impact of head loss and for practical purposes. 2 m of a PP SN16 pipe was used, and 300 holes were manually bored 1 m in the centre of the pipe, in the same patterns as in the field pilot. The infiltration pipe was placed in a chute, supported by two benches, with adjusted height and distance to each other to give an inclination of 10 ‰. Downstream the infiltration pipe, a bend attached to a vertical pipe of 160 mm was assembled. A hole was bored in the bottom of the bend, and then a tube connect to a clear perimeter was installed. Alongside the perimeter a measuring tape was attached, for the purpose of reading of the water level manually. Upstream the infiltration pipe, a T-bend was assembled, where a vertical pipe of 160 mm in diameter was attached on top, while at the bottom the lab pilot was connected to the already existing pipe network connected to a water pump, DESMI NSL 150-265 A12. The vertical pipe upstream replaced the manhole with a diameter of 1600 mm in the field pilot, as the vertical pipes function solely were to build head pressure and not considered for storage purposes. The reason for supplying the water from the underside was to reduce potential turbulence in the system, which differs from the field set-up where the water flow from the top. The water flow was measured by a flowmeter, Siemens Sitrans FM MAG 5000.

To prepare a run, first the measure tape was adjusted to level zero equal to the water level in the perimeter when the system was emptied. Then the systems was run with a high water flow, with the purpose of emptying the water supply pipes of air to secure a continuous water flow when the measurements were done. The measuring points were done at different water flows, in increments dependent on the height interval of the perforations: every 0.5 L/s in the interval 0-0.08 m, every 1 L/s in the interval 0.08-0.16 m and every 1.5 L/s from 0.16 m. To adjust the water flow, both counterpressure to the water pump and capacity were adjusted. The water pump provided a water flow in the range of approximately 1.0 L/s to 35 L/s. The values given by the water flow meter and the observed water level were not exact, hence an average value of the observed data was chosen.

Two cases were tested and repeated three times each: free flow without a geotextile and flow with a geotextile, class NG3. The selected geotextile was the same as the one used in the field pilot. For the purpose of this study the geotextile was wrapped around the pipe two times to replicate possible counterpressure from the surrounding soil in the pilot. The exfiltration capacity of the pipes was assumed to be the limiting factor compared to the infiltration capacity in the soil. This is based on the observations that the detention basins have only been activated two times in the past two years, and that the groundwater level follow seasonal variations and not specific rain events.

To verify the results from the lab, theoretical calculations were carried out by using the Orifice equation for each perforation:

$$Q = Ca\sqrt{2gh} \tag{2.1}$$

where; Q = flow (m^3/s); C = coefficient of discharge; a = cross-sectional area (m^2); g = gravitational acceleration (m/s^2); and h = total head (m).

In this method Smith's Coefficient of Discharge for circular orifices was used, based on experiments on sharp edged orifices (Smith Jr., 1886). The coefficient varies for both diameter and head, but the differences are negligible and in these calculations C was given a set value of 0.63. Calculations were carried out for one meter of the pipe with 300 perforations.

For the theoretical function with 300 perforations, and adjusted theoretical function was calculated to take minor losses into account. The minor loss coefficient was found using formula:

$$h = K \cdot \frac{V^2}{2g} \tag{2.2}$$

where h = head loss (m); K = minor loss coefficient; V = velocity in pipe (m/s); and g = gravitational acceleration (m/s^2).

The head loss was given as the difference between the theoretical height and the observed height from the free flow function, at the same flow. The velocity was found using Manning's formula, where Manning's roughness coefficient was set to the default-value of 0.01 as the pipes are around two years old and it is assumed low sedimentation.

All the exfiltration functions were expected make a jump when the number of activated perforations increased, therefor one function was made for each scenario; when the bottom perforations were activated, when the sides were activated as well and when all perforations were activated.

2.3 Mass balance model

A mass balance model was built based on the design and concept of the full-scale combined infiltration and detention solution in Trondheim. The system boundary of the model is illustrated in figure 2.5.

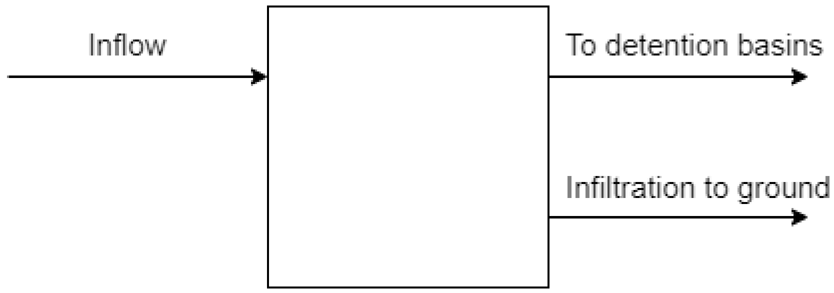


Figure 2.5: System boundary for the mass balance model

The mass balance was done in manhole O8 in the field pilot, where the stormwater in to the system is sent to either infiltration to the ground or to the detention basins. The model assumes no water storage in the area below the infiltration pipes in manhole O8 for sand trap purposes or in the infiltration pipes themselves, and there is not set an upper height limit in the manhole.

The model can be calibrated using to two criteria: the time of activation and volume detained or the activation frequency of the basins. For the first criterion, the model was calibrated to the event specific, monitored data from the two rainfall events that have activated the detention basins, comparing the time of activation and the corresponding water level, given a 1-minute resolution. Calibration was done by trial and error at random of the different exfiltration functions and change of coefficients. To compare the output from the model to the monitored data, a part to calculate the water level was added to the code, based on the same geometry assumptions that were made by (Vartdal, 2021). These calculations do not consider the swirl chamber, that regulates the outflow from the detention basins in the field pilot.

For the second criterion, the model was calibrated to the monitored data on the activation frequency of the detention basins using the continuous, monitored data-series of inflow. To calibrate the model coefficient adjustments where done to the different exfiltration functions. The results from calibrating the model to the two criteria where compared, and formed the model to use for other scenarios. The code for the model can be found in appendix A5.

2.4 Scenarios

To further evaluate the applicability of the model, a selection of scenarios was run in the model. Current and future, projected precipitations in the cities of Trondheim and Bergen were selected. Trondheim was selected as it is the location for the studied field pilot, and Bergen was selected to compare cities with the same stormwater management approach, but with a different rainfall pattern. Bergen is also one of Norway's larger cities, and is located on the west coast. Compared to Trondheim, Bergen has more than the doubled amount of annual precipitation but nearly the same amount of days with precipitation (Rydock et al., 2005). From this it follows that the rainfall pattern in Bergen is more intense and heavier. The scenarios were based on the following data:

- **Scenario 1:** Current precipitation data for Trondheim in the period of 1988-2019
- **Scenario 2:** Projected precipitation data for Trondheim in the period of 2071-2099.
- **Scenario 3:** Historical precipitation data for Bergen in the period of 1990-2019
- **Scenario 4:** Projected precipitation data for Bergen in the period of 2071-2099

The data series for the historical and projected precipitation in Trondheim and Bergen was available from the study done by Pons et al. (2021). The series were based on precipitation measured at Risvollan station in Trondheim and Sandsli station in Bergen. As the mass balance model was built using inflow to a point, both the historical and projected time series of precipitation was applied to the SWMM model developed by (Vartdal, 2021) to generate inflow to the same point under the same conditions as for the monitored data. The projected time series were given in a 6-minute resolution, but as the mass balance model was built on a 1-minute resolution, the flow from the SWMM model was generated in a 1-minute resolution. All four scenarios were run in the model, which gave a list of water flow activating the detention basins. For each scenario the activation frequency was manually counted, and divided on the number of years of data before the scenarios were compared to each other.

To evaluate the consequences if the field pilot was built solely using infiltration pipes and not detention basins, it was assumed that the stormwater sent to the detention basins was distributed and detained in the catchment area of the field pilot instead. It was not considered how this is solved in practice, and the stormwater sent to infiltration will still be infiltrated. The rainfall event with the highest accumulated stormwater volume sent to detention was manually selected for each scenario. The volume was calculated assuming a constant water flow in each time-step, and then accumulated over the period the detention basins were activated. The accumulated stormwater volume was divided on the catchment area of the field pilot, to get a distributed water level. These calculations do only consider volume, and not the effects of inclination on water flow within the catchment area. The catchment area covers almost the whole town square, and will be addressed as the town square further in this thesis.

For the selected rainfall event in each scenario, the corresponding return period for the rainfall was found. In this thesis, the return period was found by first identifying the start and end of the rainfall event activating the system. As the mass balance model do not consider saturation in the ground, it is considered memoryless, and the cut was done considering the timeframe with a continuous series of precipitation including the rainfall event itself. Then the precipitation was accumulated, and compared to the IDF-curves for Risvollan (Trondheim) and Sandsli (Bergen). For all the scenarios the current IDF-curves were used, without an additional climate factor.

The activation frequency, the water level in the town square, and return period of the rainfall event were used to illustrate some of the literature used to evaluate the second step in the S3SA.

3 Results and Discussion

The results and discussion section is divided into four parts: subsection 3.1 validation of the lab experiment; subsection 3.2 presents the activation frequencies for the different scenarios; subsection 3.3 evaluates the second step in the S3SA using the results from the previous subsection and relevant literature; and subsection 3.4 discuss further work based on this study.

3.1 Validation of lab experiment

The exfiltration functions made from the lab experiment and theoretical calculations are presented in 3.1. One exfiltration function was found for each of the three height intervals. The geotextile functions were second-degree polynomials, with an $R^2 > 0.999$, the free flow functions were third-degree polynomials, with an $R^2 > 0.999$, and the theoretical function in the first interval was a second-degree polynomial, while the two other intervals were third-degree polynomials, all with an $R^2 > 0.999$.

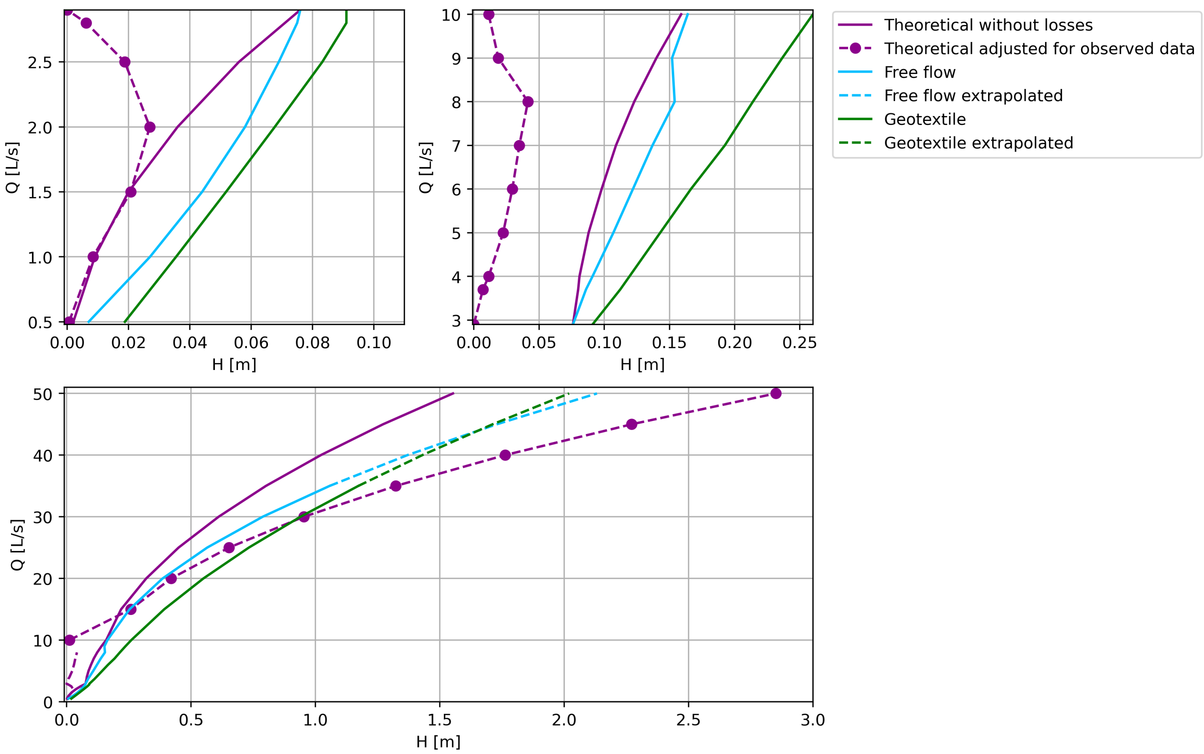


Figure 3.1: Comparison of the exfiltration functions from the theoretical calculations with and without minor losses, and the experiments with free flow and geotextile. The dashed lines for the free flow and geotextile are extrapolated from the exfiltration functions.

In the figure, the instability in the height intervals is illustrated, and in the interval of 0.08 - 0.16 m, the gradients are steeper than for the other intervals as the number of newly activated perforations is doubled. Overall, the plotted results follow the expected hydrodynamical pattern: Given equal heads, the run with the lowest resistance, has the highest flow. This hydrodynamical principle is contradicted at two points: when the theoretical and free flow graphs meet at H = 0.076 m; and when the extrapolated free flow and geotextile graphs intersect at H = 1.7 m. For the functions derived from the lab experiment, the irregularities could be due to uncertainties in the measurements, which could be corrected by additional runs. The results were also limited by the pump capacity, as the water flow was limiting to the pressure head in the vertical pipes. This will give some uncertainties when running the mass balance model at pressure heads within the extrapolated area. The hydrodynamical principle is also contradicted, where the extrapolated free flow function intersects with the theoretical function, as seen in figure 3.2, as the the free flow includes minor losses and should give a lower flow rate than for the theoretical function. This could be corrected using a larger capacity pump, to get a sufficient water flow for all the heights in the manhole. Hence, this would give a more fitting exfiltration function for the water levels above 1-1.2 m, as the current free flow function give a higher water flow than what is hydrodynamical possible for water levels in the extrapolated area. In the lab experiment the pressure head was measured downstream the inclined pipe, which gave a higher head downstream than upstream. This was supported by observations from the lab experiment, that showed not all perforations were activated when the pressure head downstream would indicate it, seen in figure 2.4b. This contributes to unstable measurements and irregularities around the height when new perforations are activated, at 0.08 and 0.16 m.

The irregularities in the free flow function, influences the theoretical function adjusted for monitored data, as the minor loss coefficient K, is correlated to the relationship between the theoretical and free flow function. This relationship is graphically illustrated in appendix A4. K varies in the range between 0.01 and 1.84, while the height relationship between the theoretical and free flow function range from 1.03 to 3.01. The irregularities reflect the instability up to Q = 10 L/s, the point at which all the perforations are activated. From the following stabilisation of measurements from Q = 15 L/s, K has a steeper gradient as the minor loss is a function of height difference and velocity which gives a higher friction as more water is pushed through the pipe due to the static head. The minor losses could be induced by the rough edges of the manually drilled perforations, and other factors such as roughness of inner surface, and ratio of the orifice diameter to the dimensions of the tank wall.

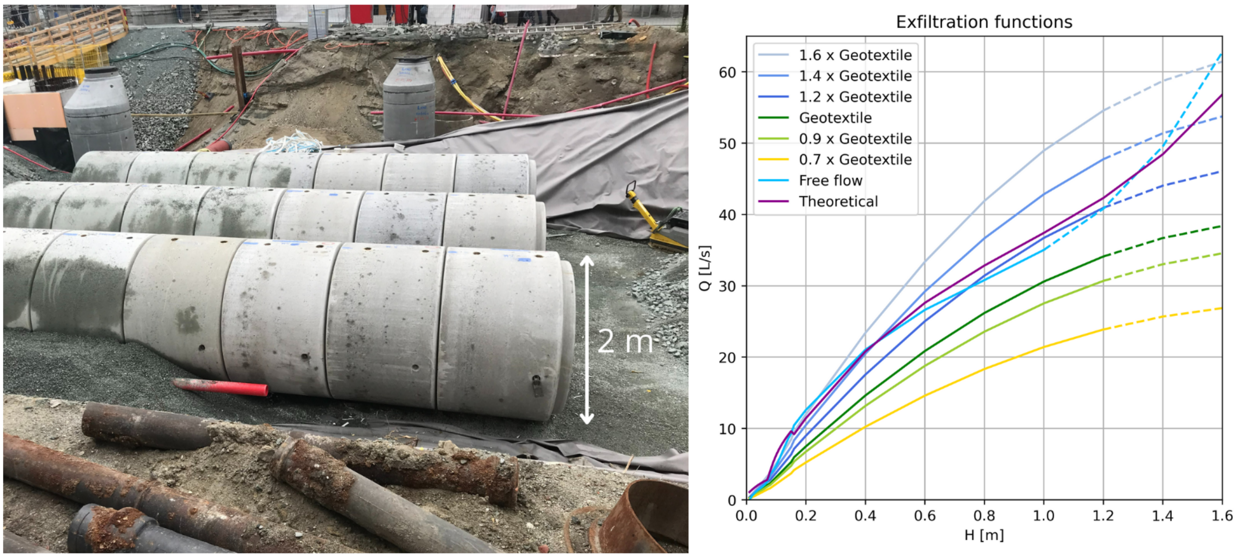


Figure 3.2: Left photo: From the construction site of the field pilot. The red pipe in front of the photo is the infiltration pipe. Credits: Edvard Siversten (SINTEF). Right graph: Comparison of exfiltration functions adjusted for calibration coefficients. The dashed lines are extrapolated, and the theoretical line is without adjustments for minor losses.

Figure 3.2 illustrates the relationships and the effect of the different coefficients for the geotextile exfiltration function, and the free flow and theoretical functions without adjustments. The geotextile function gives more counterpressure to the infiltration pipe than in the field pilot. As seen in the photo in figure 3.2 the geotextile is not wrapped around the infiltration pipe, but instead it is separating the infiltration pipe embedded in crushed stones from the native soil. The crushed stones will give less counterpressure than the geotextile, and the field exfiltration rate is expected be somewhere between the free flow and geotextile function. Figure 3.2 shows that a calibration using coefficients do not consider what is physical possible. The different, calibrated geotextile functions with a higher output than the theoretical and free flow cannot physically perform better than the two more ideal situations without counterpressure and minor losses, as the performance of the geotextile function is limited by those. Even though these functions are not physical possible, they are used for calibration to better understand the model.

The model was validated to two different criteria for each of the monitored rainfall events that activated the detention basins, the 18th of July and 8th of August 2020, by adjusting a calibration coefficient for each of the exfiltration functions. First the model was calibrated using time of activation and detained volume, and the results from the calibration process are illustrated in figure 3.3. For the event the 18th of July the model was calibrated to 90 % of the geotextile function, where it matched both activation time of the detention basins and water level. From the point of activation, the graphs deviates; the calibrated graph accumulates stormwater, while for the monitored graph, the swirl chamber is activated and gradually releases the accumulated stormwater.

In comparison, the event the 8th of August was calibrated to 70 %, 90 %, and 160 % of the geotextile function. For the first two calibrations, the activation frequency was 5 minutes earlier compared to the monitored activation time for the field pilot, while it had to be calibrated to a 160 % to give an activation frequency closest to the monitored at 1 minute too early. The calibrated graphs flatten when the exfiltration rate is higher than the inflow, while the monitored graph continuously increases and consequently accumulates stormwater despite the activation of the swirl chamber. The deviating behaviour between the monitored and calibrated graphs at 70 % and 90 % when the stormwater is accumulated, can be explained by an even lower exfiltration coefficient, that also compensate for the activation of the swirl chamber. These observations, in addition to the knowledge on the dry weather previous to the event, it is probable to assume that there is a high level of storage capacity in the trench and for the soil to be unsaturated when the event starts. Hence, the first flush of stormwater will rapidly exfiltrate to the ground, and fill the storage volume in the trench, before infiltrating to the ground. But, as the trench fills up and the soil becomes saturated, the exfiltration rate will decrease and the infiltration pipes will get some counter pressure. The stormwater will start to accumulate in the manhole before it is sent to the detention basins.

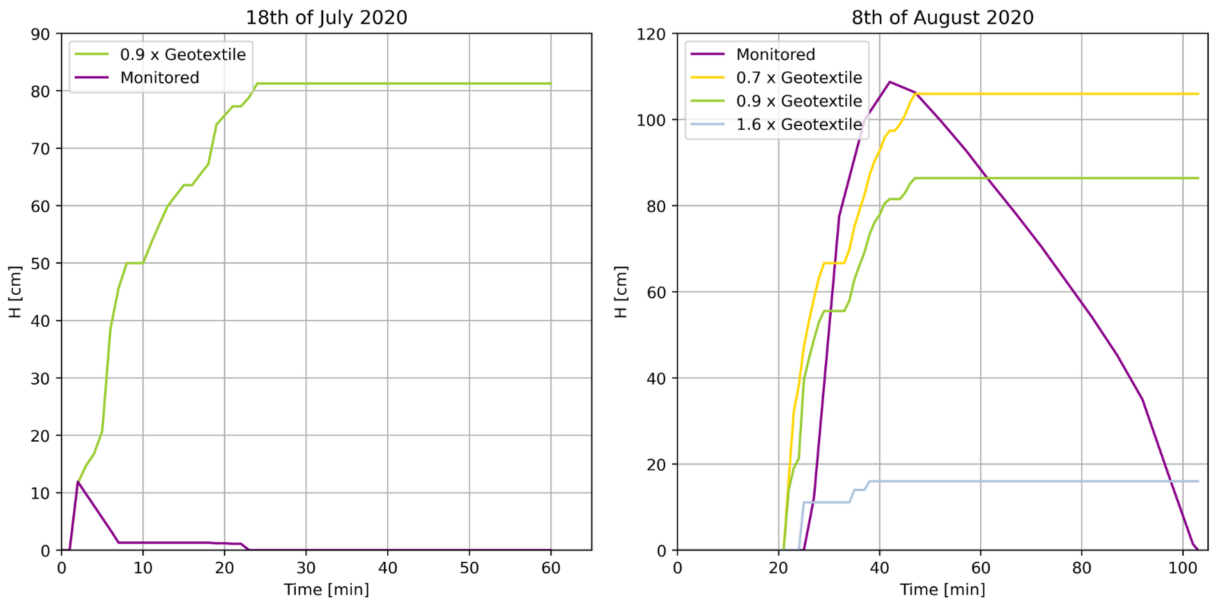


Figure 3.3: Calibration of the model using time of activation and detained volume. Height is given above detention basin bottom.

For the second validation, the model was also calibrated to the monitored, activation frequency given the whole data series of monitored inflow at the field pilot. To separate the rainfall events the 8th of July and 18th of August 2020 as the only events activating the detention basins since the start of monitoring, the geotextile function had to be calibrated to a 160 %. In addition to the two expected events, a third event the 3rd of July 2020 activated the basins. This event had the highest measured inflow to the field pilot in a minute, at 90 L/s. Looking at precipitation data, inflow pattern, and after talking with Trondheim Municipality which operates the field pilot, it is reasonable to assume that the system was flushed manually.

As 160 % of the geotextile function is not physical possible, the calibrations that use this function indicate that there are other factors that influence the output of the exfiltration rate in the model. This could be due to the model solely looking at what happens within a timestep of 1 minute, and do not consider the continuous changes in water level in the manhole and detention basins through time. For further use in this thesis, the model will use the geotextile function without additional adjustments. It is selected, as the calibrations showed that the exfiltration rate is within the range of 70 % to 160 %. As the geotextile function is considered to have a lower exfiltration rate compared to the field pilot, a more conservative function is considered acceptable when the infiltration properties and possible saturation are not accounted for in the model.

3.2 Activation frequency

The activation frequencies of the detention basins, for current and future climate scenarios in Trondheim and Bergen, are presented in table 3.1.

In between the different scenarios the results are as expected. Bergen has a heavier precipitation pattern than Trondheim, hence the detention basins were activated more frequently. For Trondheim the future activation frequency is close to unchanged, which is supported by Hanssen-Bauer et al. (2017), who found that climate change will have a relatively small effect on the precipitation pattern in Trondheim. The deviation in between the current and future scenario for Trondheim, could be a result of the limited time series of approximately 30 years, as the current view on return periods today, typically includes a 200-year period. Bergen, on the other hand, is expected to have an increased rainfall intensity and number of days with heavy rain, and as the result the future activation frequency increases by a factor of 2.7.

Table 3.1: Activation frequency of detention basins derived from the mass balance model

Scenario	**Act.freq. [times]**	**Act.freq. [times/year]**
1: Trondheim current	64	2.0
2: Trondheim future	58	1.8
3: Bergen current	211	7.0
4: Bergen future	547	18.9

With the current design, under perfect conditions, stormwater will start to accumulate on the surface when there is a design failure to the second step, which corresponds to once in every 20 or 25 years. The third step could also be activated due to structural failure of the second step, hence an expected activation frequency of the the third step could be more often than a 20- to 25-year return period. Beside discussing an increased activation frequency in terms of resilience, there is an aspect of social acceptance: to accept visible stormwater and reduced operability more frequently. This aspect will not be further discussed in this thesis, but is of relevance when discussing an acceptable level of activation frequency.

In figure 3.4 the event with the highest flow activating the detention basins for each scenario, is imagined detained in the town square, instead of in the subsurface basins for the whole rain event. For each of the scenarios, it corresponds to a rainfall event with a return period of: 5 years for Trondheim current; 10-20 years for Trondheim future; 10-20 years for Bergen current; and more than 200 years for Bergen future. The current events in Trondheim and Bergen, have the same accumulation pattern, but the difference in return periods shows how the model is triggered by a peak flow, as a result of the length of the rainfall event. The event in Bergen future stands out from the others, being an extreme event where it rained 180 mm in 13 hours. In this period of time, the surface accumulated stormwater for 5.5 hours.

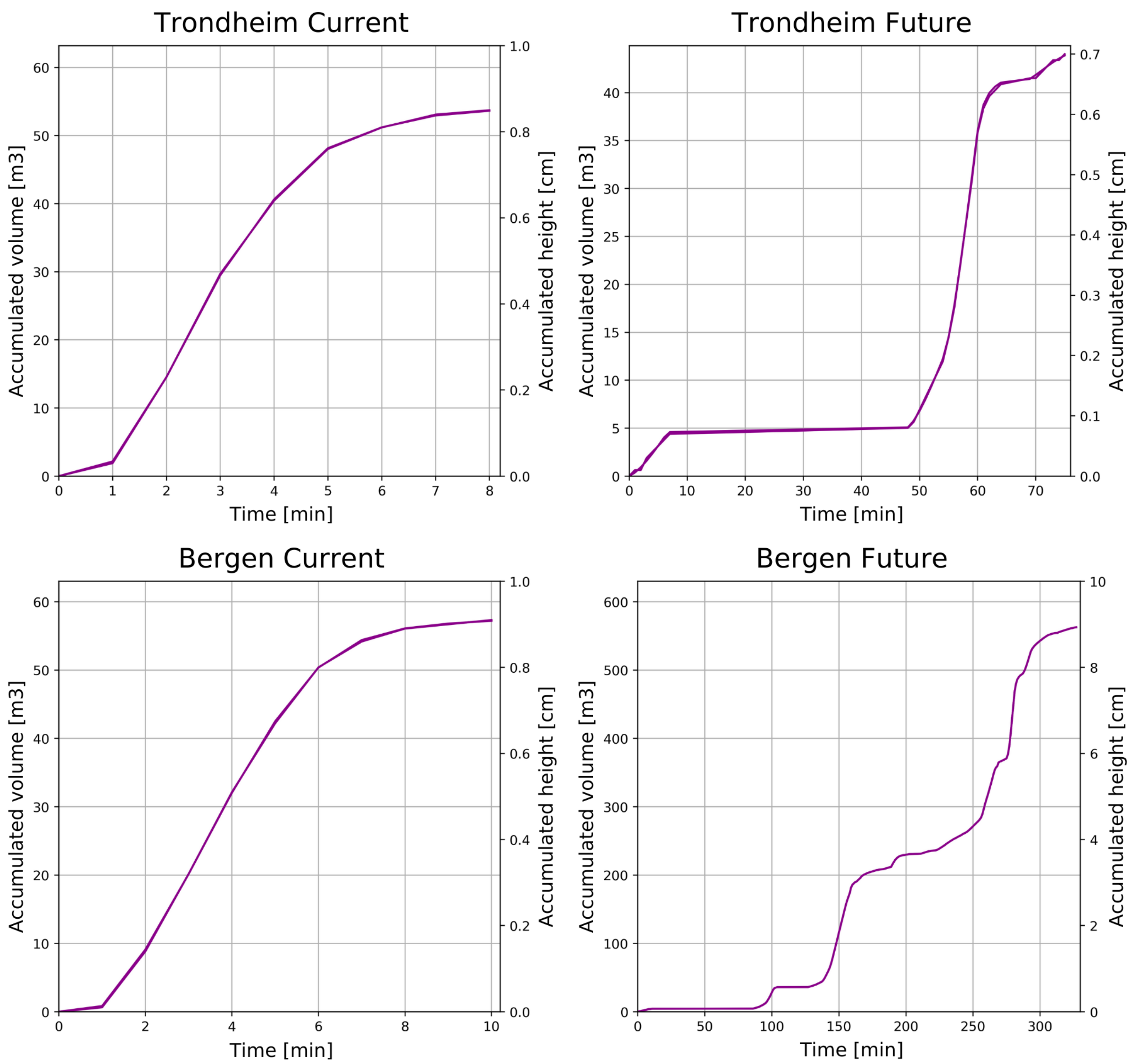


Figure 3.4: Considering the rain event with the highest flow sent to detention for each scenario, and the effects on the town square if the field pilot did not have detention basins.

The consequences of the stormwater accumulated on the town square is dependent on the next step in the stormwater management approach. There are two approaches for the next step: It could either be detained and stored, and later be sent to infiltration in the field pilot when it has capacity; or it could gradually during the time of accumulation, be sent to a recipient through urban floodways. The first option would give an extended period of stormwater accumulated on the surface at max water level. For the second option, the time of accumulation would be the same, but the water level would not reach its peak as it is diverted to a recipient.

3.3 Evaluating step 2 in the S3SA

The purpose of the second step in the S3SA is to detain the stormwater that cannot infiltrate to the ground, and to maintain the peak flow to the same as the pre-urbanized conditions. It is designed for moderate rainfall, which in most municipal guidelines corresponds to a 20- to 25-year rainfall.

Today the second step in the S3SA must handle the increased stormwater volumes induced by urbanizing an area, on-site. Norsk Vann (Norwegian Water) (2020) indicated that optimizing the stormwater management on-site could lead to negative effects over time off-site. Changes in runoff pattern and increased stormwater volumes downstream are consequences seen in connection with the second step, e.g. potential downstream erosion. Kim et al. (2017) suggested to look at the stormwater management approach as a system and not at a component level. The results on activation frequency indicate that a combined system, compared to only detention, has reduced the level of combined sewer overflows due to the favourable infiltration properties. Dawson et al. (2020) argue that to manage stormwater the interoperability between systems must be consider, such as the infiltration component in the pilot with existing infrastructure such as roads, land use and buildings.

Norsk Vann (Norwegian Water) (2020) propose that the Norwegian guidelines follow the ones from Copenhagen, and that the surrounding infrastructure should be secured towards a rainfall event with a return period of a 100 years if the stormwater on the surface accumulate to more than 10 cm. This is in addition to the already existing demand, to design floodways for a minimum return period of a 100 years. From the scenarios, the highest accumulated water level is just below 10 cm for a 200-year rainfall event in

Bergen. If changes in the guidelines are carried out, this would mean that the surrounding infrastructure to the town square should be prepared and secured to meet this extreme event. The town square is relatively large compared to other open spaces in urban areas, and given the same conditions except the a smaller area, the accumulated water level would get higher. As a result for the future scenario in Bergen this would lead to an amount above suggested design practice and is in many municipalities also above the current requirements for floodways. For the other three scenarios the increase in water level would still be manageable within the proposed guidelines for a smaller area. When the floodways are designed to make them robust to handle a 100-year event, it also indicate a capacity to handle rain events up to a return period of a 100 years. Thus floodways do already take the expenses to manage the moderate rainfall into account, and the costs of designing a measure to manage the same rain in the second step is contradicting. A cost-benefit analysis could be carried out, to go in detail on how the cost to manage moderate rainfall with and without the second step is distributed, and the effects of the current and proposed guidelines (Mikkelsen, 2021).

The demand from the municipalities to handle a 20-year rainfall event on site, increases the pressure on developers as urbanization limits available area for both stormwater measures and other infrastructure. While the current paradigm provides clear and measurable criteria for dimensioning based on a predefined level of protection, it is criticised by Haghighatafshar et al. (2020) for using specific design rains with changing recurrence intervals which do not consider the complexity of urban system. The design criteria is based on statistical analysis, a climate factor, and as the development of cities also leads to high exposure of people and assets to flooding (Rosenzweig et al., 2019), safety factors are integrated in the return period thresholds dependent on the consequences. This gives large, oversized infrastructures largely as a result of a design based on different uncertain parameters. Instead of basing the guidelines on predicted future measures that are uncertain, Kim et al. (2017) suggested to develop a resilient design that rather is based on how to handle an uncertain future. The comparison of the current scenarios in Bergen and Trondheim, indicated that different return periods could have the same effect on the system, dependent on the peak flow. The detention basins in the field pilot were reduced to 72 % of the original design, as the needed storage volume was considered reduced in the combination with infiltration. The 10-year rainfall event the 8th of August 2020, activated the detention basins and reached a max water level at around 110 cm. This was a cloudburst event, and a similar cloudburst event with a return-period of 20-years is not expected to reach full height in the basins. Hence, the field basin are considered overdimensioned compared to the requirements based on the current design practice. The corresponding consequences of overdimensioned concrete basins, are high CO_2 emissions, energy consumption and costs (Wang et al., 2017).

If the second step could be removed, will be dependent on different factors such as topography, proximity to recipient, soil conditions and climate. To evaluate the applicability of the surface above ground to detain and transport water, it is dependent on the velocity and hazard potential (Skrede et al., 2020). In the case of the field pilot in Trondheim town square, a removal of the detention system, would require to activate the connecting streets as temporary floodways more often. The city centre of Trondheim is flat and in close proximity to the fjord, directly or through the Nidelva river, and both street Munkegata north of the pilot and street Kongens gate, east of the town square, are the potential floodways. On the other hand, Bergen is a city with steep, populated hills, which would be inadequate in including the surrounding infrastructure in managing stormwater above ground due to high velocities and hazard potential even at lower return periods (Skrede et al., 2020). Compared to Bodø, they share some of the same properties, the soil conditions are inadequate for infiltration, but in Bodø the hills are not populated, making it possible to make use of the proximity to a recipient.

3.4 Further work

For future research based on this study, it is relevant to develop the combined infiltration and detention solution as a stormwater management measure. The infiltration pipes alone, are not common as part of the current stormwater management, but have been proven through this study to be highly effective in areas with ground conditions suitable for infiltration. This study could also be a part of a further evaluation of detention, as part of the urban stormwater management. It is in the interest of consultants, municipalities and other stakeholders to acquire more knowledge on these subjects.

Continued lab experiments

Further lab experiments on the infiltration pipes are highly relevant for a deeper understanding of the system performance. For future work the lab set-up should be reassembled to a set-up closer to the field pilot. As this study found limitations to the extrapolated exfiltration rate, a new set-up should have a pump with the capacity to give pressure heads equal to the available head. The set-up should embed the infiltration pipe in the soil from the field, to evaluate the exfiltration rate given counter pressure from the surroundings, and also for the purpose of monitoring how the surrounding soil influence the exfiltration rate over time with respect to saturation. Further experiments could also evaluate the effects of sediments in the stormwater, on the exfiltration rate. If the infiltration pipes are to be implemented in other places, alone or as part of a combined solution, a lab experiment to understand the the site specific exfiltration properties should be carried out before dimensioning a solution.

Further development and use of the mass balance model

The mass balance model can be further improved by adding other conditions, such as adjust for limitations to the performance due to infiltration and saturation, or to adjust for the effect from the swirl chamber on the volume in the detention basins. Adding the swirl chamber to the model will give a better understanding on how the volume in the detention basins are used. The additions could make the model suitable as a tool when dimensioning a solution like the pilot, or a system of infiltration pipes alone. The exfiltration rate is dependent on the pressure head, and the possible head in the system is dependent on the distance between the depth of the manhole, the inlet to the infiltration pipe, and the detention basins. Through an iterative process the model could be used to find the optimal design regarding the heights. It is also interesting to run current and future precipitation series, for more cities, e.g. Oslo and Bodø, and additional series for Bergen and Trondheim for further evaluations on activation frequency and to comprehend how local conditions influence the performance.

Further evaluation of detention as part of the Norwegian stormwater approach, S3SA

In this study the detained stormwater in the town square, has to be further managed, and possible solutions are discussed. As the stormwater is accumulated on the surface, it becomes a matter of urban flooding. Urban flooding is an active field of experimental and numerical studies, and there is research on single street intersections, surface-sewer exchange, obstacles and quasi-realistic urban (Mignot et al., 2019). A suggestion for further work is to look at the intersection between stormwater management measures in the first step and the third step in the S3SA, and how other aspects than the technical affects a change in stormwater management practice, such as social, operational and economical aspects.

4 Conclusion

This study is discussing detention as a SUDS to meet some of the challenges cities are facing due to urbanization and climate change: increased frequency of precipitation, rainfall intensity and subsequent stormwater volumes. On the contrary, this study also calls attention to the use of detention as SUDS, such as subsurface detention basins, as these also are a part of the challenges induced by climate change and urbanization: large, concrete basins that contribute to CO_2 emissions, and area demanding solutions in already dense cities.

A mass balance model was built based on a combined infiltration and detention system in Trondheim, Norway. It considered the exfiltration properties of the model to be the driving force within the system, and demonstrated the importance of good infiltration rate and storage capacity in the soil, to reach the potential of infiltration pipes as a SUDS.

The changes of activation frequency depending on location, and current and future climate, demonstrated how the precipitation pattern largely influences the performance of the system: during cloudbursts the system does not have the capacity to infiltrate all the stormwater compared to a longer event with more heavier rain. Besides the extreme events, cloudbursts are expected to cause the majority of the activation frequencies. Hence, the effects of alternative detention solutions on the surface will be relative small water levels for a short duration of time.

Through this study, the need and the performance of the second step in the S3SA, detention, are shown to be dependent on site specific conditions and current design guidelines. In cities, as Bergen, a replacement of the second step is inadequate. There is a need to change the current design approach to limit the disadvantages in the future, by changing the design criteria within a framework of uncertainty: from a design based on the need to manage a combination of uncertainties to handling an uncertain future. While in cities as Trondheim, where the site conditions are beneficial to have alternative solutions for the second step, the city will be forced to have a more holistic approach and hence gain in different aspects, where the second step today is overlapping with the two other steps. As a concluding remark, the study has revealed that the stormwater management approach in Norway to a larger extent should be based on local conditions, as is starting to emerge within the municipalities, as an implementation of the S3SA is to general, and hence limiting, for the cities to develop sustainable.

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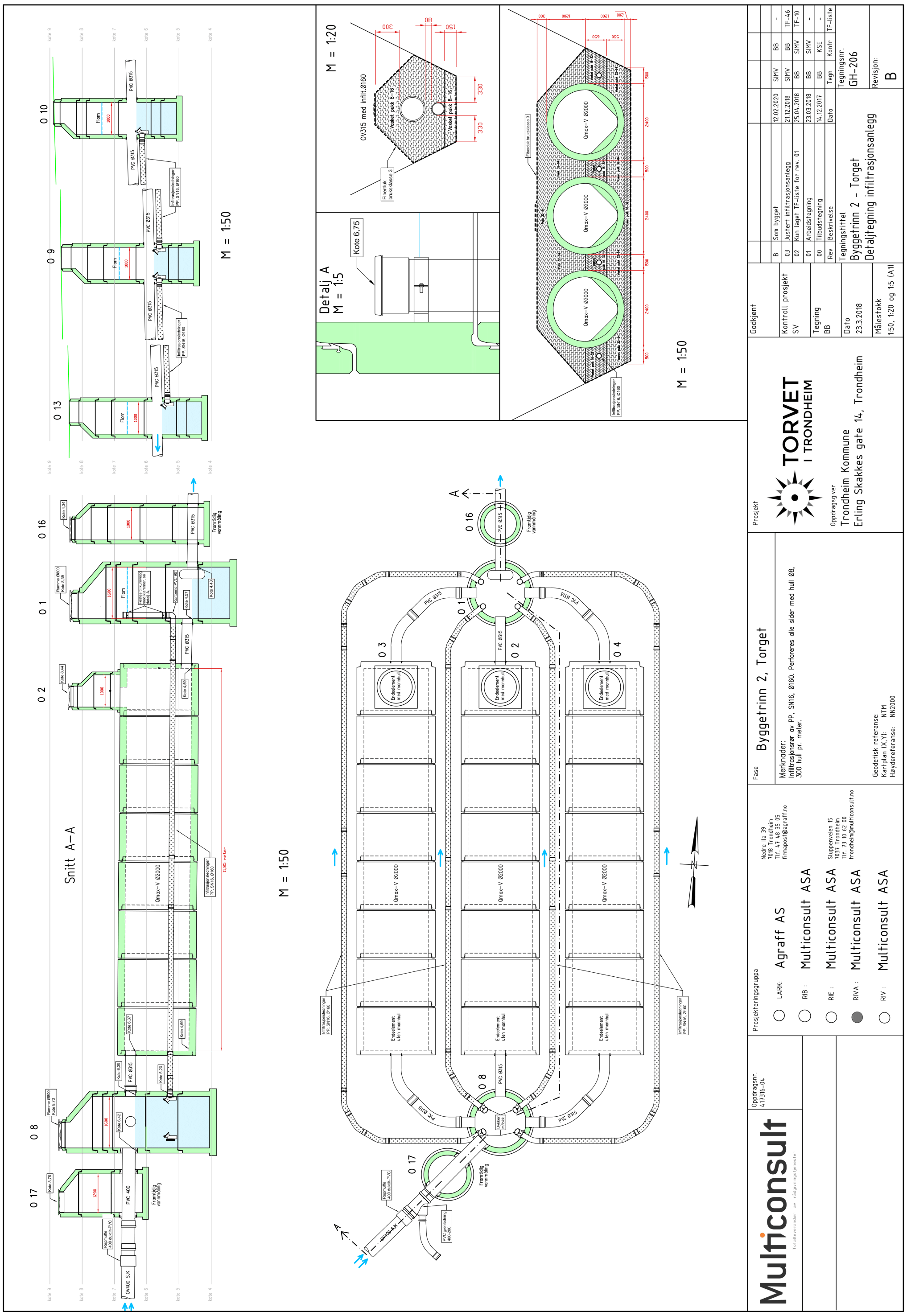
Appendix

A1 Framework for Article

Article structure: The maximum acceptable length of a Research Paper is 10.000 words, minus 350 words for each normal-sized figure or table you include. Papers should include:

- **Title**
- **Short title of no more than 80 characters including spaces**
- **Author name(s)**, full postal addresses for each author.Include the e-mail address for the corresponding author only
- **Abstract:** no more than 200 words briefly specifying the aim of the work, the main results obtained, and the conclusion drawn.
- **Keywords:** up to 6 keywords(in alphabetic order)
- **Main text:** clarity this should be subdivided into: **Introduction**; **Study area and data:** should describe the location, size, geographical and relevant climatic data and other conditions of the region. It should clearly describe all the data used and their sources,including data periods, temporal resolution, limitations, quality, etc. Use of tables is encouraged where appropriate; **Methods:** a brief description of the methods/techniques used; **Results and Discussion:** a clear presentation of experimental results obtained, highlighting and trends or points of interest and a brief explanation of the significance and implications of the work reported; **Conclusion:** a brief statement of what was undertaken in the study (one or two sentences) followed by what was established relative to the stated aims and objectives.
- **References:** Note that a paper is at risk of rejection if there are too few (<10) or too many references, or if a disproportionate share of the references cited are your own.
- **Supplementary Material:** Appendices and other Supplementary Material are permitted, and will be published online only.

A2 Detailed constructional design of the combined infiltration and detention solution



A3 Photos from the field pilot and lab

Figure A3.1: Inside manhole O8 in the field pilot. The infiltration pipes are in the bottom, the pipe south-west is the outlet pipe from the catchment area, and the last three are inlet pipes to the detention basins. Photo credits: Ida Hovland Haveraaen

Figure A3.2: The manhole downstream the detention basins in the field pilot. The four vertical pipes are connected to the infiltration pipes as an outlet if the detention basins go full, and the swirl chamber regulates the outflow from the detention basins. Photo credits: Ida Hovland Haveraaen

Figure A3.3: From the construction site of the field pilot. The red pipe is the infiltration pipe. The three concrete constructions are the detention basins. Credits: Edvard Siversten (SINTEF)

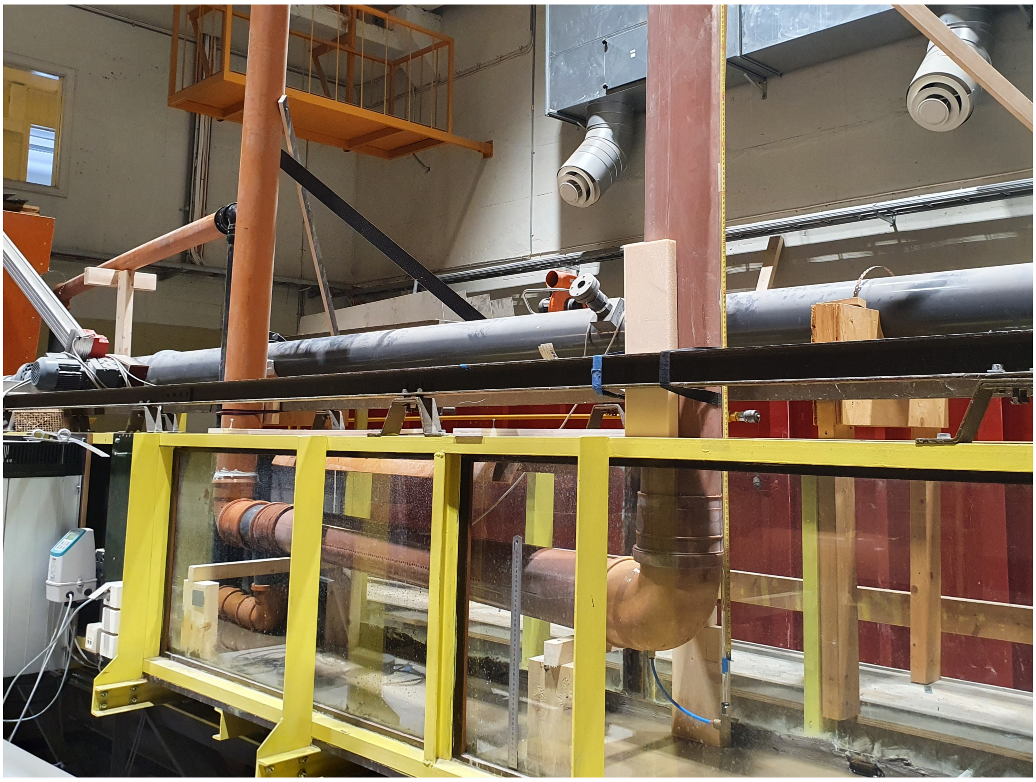


Figure A3.4: The lab set-up. Photo credits: Ida Hovland Haveraaen

Figure A3.5: A run of the free flow. Photo credits: Ida Hovland Haveraaen

Figure A3.6: A run with the geotextile. Seen from above. Photo credits: Ida Hovland Haveraaen

A4 Graphs

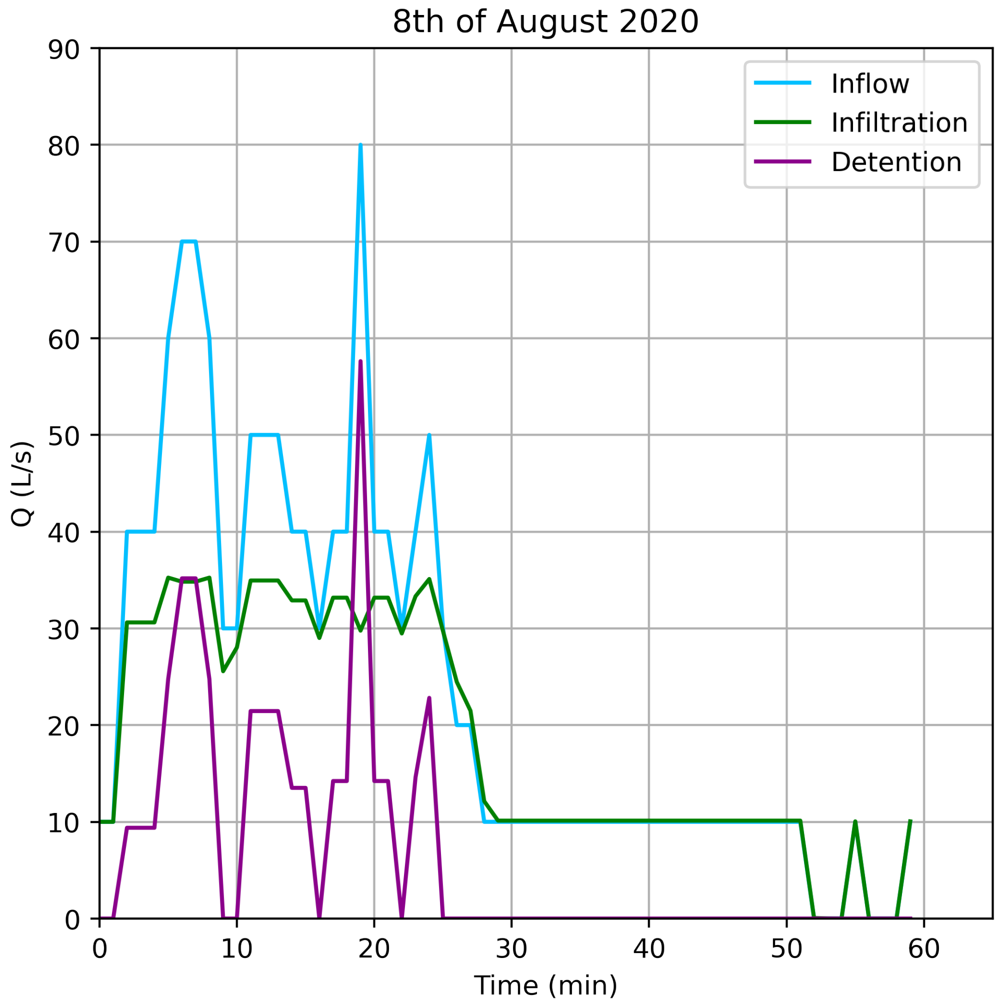


Figure A4.1: A look at the process within the mass balance model the 18th of July when the detention basin were activated. Monitored inflow is used.

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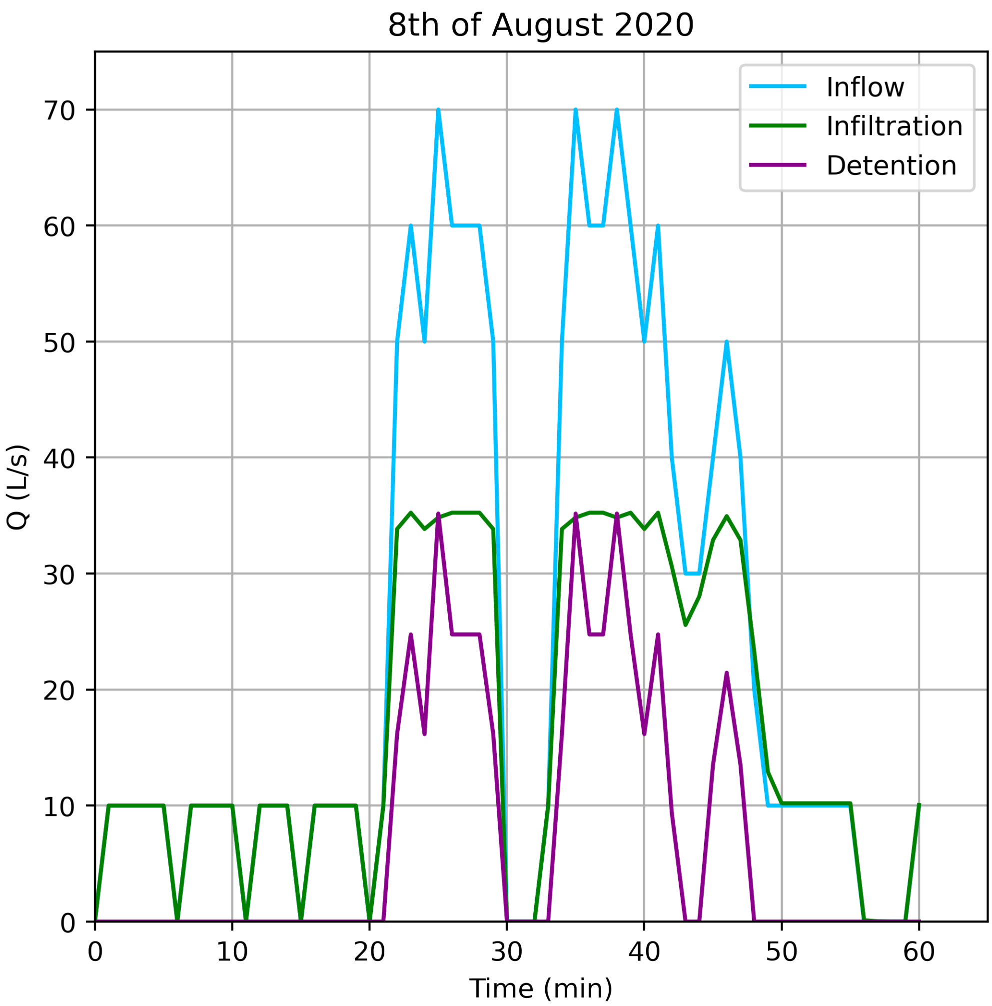

Figure A4.2: A look at the process within the mass balance model the 8th of August when the detention basins were activated. Monitored inflow is used.

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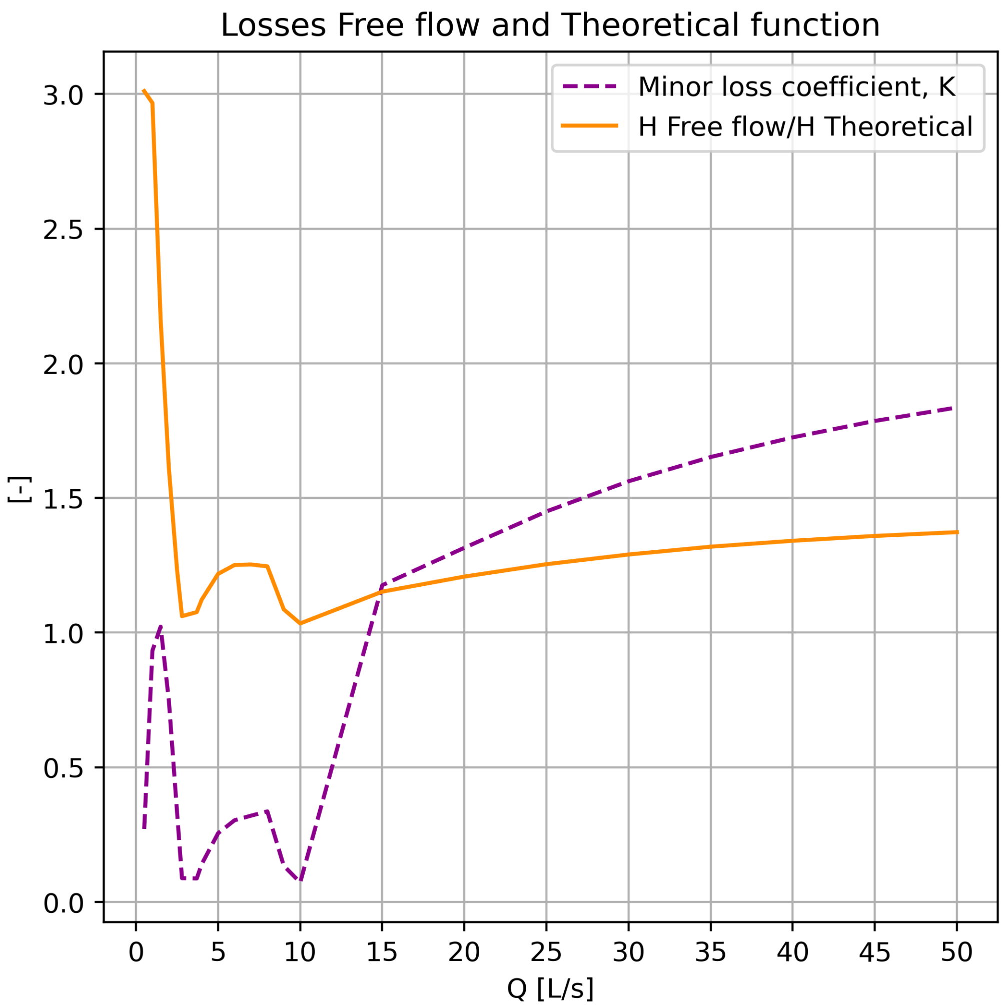

Figure A4.3: Comparison of coefficients induced by minor losses. The solid line is the height relationship between the free flow and theoretical function. The dashed line is the change in minor loss coefficient, calculated using function 2.2

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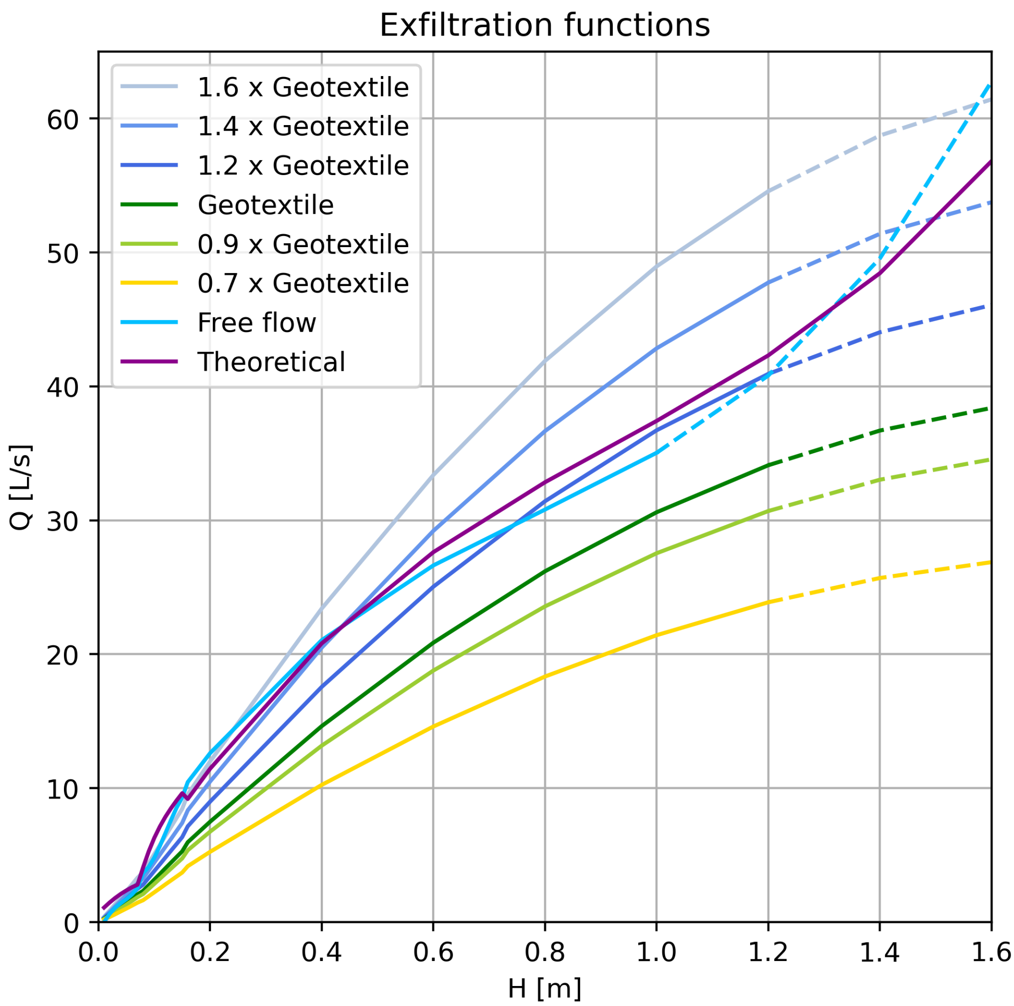

Figure A4.4: Comparison of exfiltration functions adjusted for calibration coefficients. The dashed lines are extrapolated, and the theoretical line is without adjustments for minor losses.

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A5 Mass balance code

Script of the mass balance mode, where: .

Q_{in} is inflow to the system.

Q_{exf} is the exfiltration rate.

Q_{det} is the flow sent to the detention basins.

Inf is the flow sent to infiltration.

```
import pandas as pd
import numpy as np
import matplotlib.pyplot as plt
import sympy as sp
from sympy import Symbol
```

```
#Exfiltration functions
A=0.9 #Calibration coefficient

def exf008(x): #Interval 0<H<0.8
    #return (-179.38*x**2+42.83*x+0.6718)*A #Theoretical
    #return (13429*x**3-1804.3*x**2+110.4*x-0.9842)*A #Free flow
    return (23.157*x**2+28.488*x-0.0413)*A #Geotextile

def exf016(x): #Interval 0.8<H<0.16
    #return (4962.5*x**3-2250.2*x**2+393.77*x-15.562)*A #Theoretical
    #return (-13467*x**3+4913*x**2-498.69*x+19.011)*A #Free flow
    return (0.5371*x**2+42.223*x-1.0514)*A #Geotextile

def exf120(x): #Interval 0.16<H<
    #return (19.821*x**3-55.592*x**2+74.59*x+1.3999)*A #Theoretical
    #return (30.474*x**3-72.346*x**2+77.062*x-0.1495)*0.98 #Free  flow
    return (-11.35*x**2+42.501*x-0.5517)*A #Geotextile
```

```
#Mass balance model
#Input file is called TA_flow. This was for the monitored data,
#but smaller changes were done when

h=0
H_max = 1.167 # Measured manually at site 1.167
time_step = 1
dt = time_step*60
area_08 = (3.14/4)*1.6**2
Inf = 0
dQ = 0

detention = []
detention2 = []
infiltration = []
time2 = []
height = []
Event_time = []
Event_Qdet = []


for i in range(0,len(TA_flow['Flow in (m3/s)'])): #Go through Flow_in values
    timeValue = i

    Q = (TA_flow['Flow in (m3/s)'][i])
    H = h + Q * dt / area_08 # Height in manhole 08,
                             #also consider potential extra height
                             #from previous step

    Q_in = H / dt * area_08 * 1000  # Calculating new flow dependent
                                    #on current height in manhole
    height.append(H)

    if H < 0.08:
        Q_exf = exf008(H) # Find Q_exf from Exfiltration functions

    elif 0.08 <= H < 0.16:
        Q_exf = exf016(H) # Find Q_exf from Exfiltration functions

    else:
        Q_exf = exf120(H) # Find Q_exf from Exfiltration functions

    if Q_exf < 0:   #As a compensation for negative values from the
                    #Exfiltration functions
        Q_exf=0
```

```
    if H < H_max and Q_in > Q_exf:
        dQ = Q_in - Q_exf
        h = dQ / area_08 * dt / 1000
        Q_det = 0
        Inf = Q_exf

    elif H >= H_max and Q_in > Q_exf:
        Q_det = Q_in-Q_exf
        Inf = Q_exf

    else:
        Q_det = 0
        Inf = Q_in

    time2.append(timeValue)
    detention.append(Q_det)
    detention2.append(Q_det*dt/1000)
    infiltration.append(Inf)
```

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