Modelling and Analysis of Urban Flooding in Lundby-Kyrkbyn, Göteborg

System performance and evaluation of possible improvements

Master of Science Thesis in the Master's Programme Infrastructural and Environmental Engineering

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Department of Civil and Environmental Engineering
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CHALMERS UNIVERSITY OF TECHNOLOGY
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Cover: (a) Pipe flooding and node flooding in the current system during 10-year return period rain (b) Pipe flooding and node flooding after the implementation of sewage system improvements during a 10-year return period rain.

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ABSTRACT

The runoff generation in Lundby-Kyrkby area (Göteborg) is increasing due to a combination of larger rainfall intensity and more impervious areas. As a consequence several properties have been affected by basement flooding during heavy rainfall events when the network was severely overburdened. This thesis aims to study the management of stormwater excess in order to evaluate how drainage networks could be improved to avoid basement flooding events. The investigation is focused on the identification of critical parts of the sewage system under risk of flooding, along with the feasibility analysis of different potential solutions to handle the excess of stormwater.

The tool to achieve this target is the commercial software MIKE URBAN. The network performance was simulated with this engine by means of a one-dimensional model that represents the sewage system. The obtained results were analysed to determine the critical parts in the area and made up the basis for deciding the location and characteristics of the required measures.

The simulations confirm the insufficient capacity of the system to handle heavy rainfall with one third of the nodes flooded during a 10-year return period rain. Therefore, the effectiveness of different sustainable drainage systems (grass, ditches, detention ponds and gravel tanks) were evaluated for this case study, along with infrastructural modifications addressed to detain water or increase the system capacity (reservoirs, diversion of pipelines and enlargement of pipe diameters). The obtained results provide a valuable basis on how flooding risk can be minimised in problematic properties, as well as in the whole system.

Key words:

Stormwater management, basement flooding, MIKE URBAN, pipe filling, node flooding, pipe flooding.

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Preface

Several properties located in Lundby-Kyrkbyn area (Göteborg) have been affected by different basement flooding events. In this Master Thesis critical parts of the system under flooding risk were identified and different possible solutions were evaluated. The project was cooperation between the consultant firm Reinertsen Sverige AB, Gothenburg Water (the municipal water company) and the Division of Water Environment Technology at Chalmers University of Technology, Sweden. The thesis was carried out at Reinertsen Sverige AB from January to June 2012.

First of all, we want to thank our two supervisors Annika Malm, Gothenburg Water, and Johan Sabel, Reinertsen Sverige AB, for the guidance, valuable discussions and feedbacks during the project. We would also like to thank Helen Ander for providing us with valuable information regarding the case study area and Jonatan Larsson, our opponent, for the feedback

Special thanks also directed to all the people working at Reinertsen Göteborg for supporting us during these months and for all the nice coffee breaks we had together. In particular, big thanks to Bruno Antona for his time and patience with us.

Finally, we would like to thank our friends and family for their help during these years of hard study, in special to our parents and siblings who have always encouraged and supported us.

Göteborg, June 2012

Tania Sande Beiro & Ángela Serrano Manso

Notations

Roman upper case letters

A	Cross-sectional area
A_f	Flow area
A_i	Area of each surface
A_{eff}	Effective ditch area
B	Weir width
C_H	Level discharge coefficient
D	Pipe diameter
F	External forces acting in the pipe surface
H	Hydraulic head
H_w	Water elevation above weir crest
I_0	Bottom slope
I_f	Friction slope
L	Pipe length
Q	Discharge flow
Q_w	Flow rate
V	Mean cross-sectional velocity
V_t	Total volume of the gravel tank

Roman lower case letters

g	Gravity
h_f	Head loss due to friction
p	Preassure
t	Time
t_c	Concentration time
v	Velocity
x	Distance in the flow direction
z	Potential head

Greek letters

α	Velocity distribution coefficient
ρ	Density of water
λ	Friction factor
Δt	Simulation time step
φ	Imperviousness of the sub-catchment
φ_i	Imperviousness of each type of surface

Abbreviations

Alt	Alternative
BH	Borehole
CRS	Cross Section Shape
CSO	Combine Sewer Overflow
DHI	Danish Hydrology Institute
DWF	Dry Weather Flow
EEA	European Environmental Agency
EPA	Environmental Protection Agency
EU	European Union

GIS	Geographical Information System
RTC	Real Time Control
SMHI	Swedish Meteorological and Hydrological Institute
SWWA	Swedish Water & Wastewater Association
SWMM	Stormwater Management Model
T5	Continuous rainfall with 5-year return period
T10	Continuous rainfall with 10-year return period
T20	Continuous rainfall with 20-year return period
T50	Continuous rainfall with 50-year return period
T100	Continuous rainfall with 100-year return period
T5Block	Block rainfall with 5-year return period
T10Block	Block rainfall with 10-year return period
UNEP	United Nations Environment Programme
WMO	World Meteorological Organization
WWTP	Wastewater treatment plant

1 Introduction

Among all natural disasters that affect humanity, floods are the most common phenomenon that causes severe damage (EEA, 2001). This kind of events threats infrastructures, properties and human lives due to an excess of runoff flowing in the surface. In particular, a major problem that Sweden is facing nowadays is the management of stormwater in urban areas during heavy intense rainfalls (Swedish Commission on Climate and Vulnerability, 2007). As a direct consequence of climate change, precipitation patterns have been modified resulting in increased precipitation and more intense rainfalls. Moreover, this problem is intensified in urbanized areas where the infiltration into groundwater is hindered by paved surfaces with high imperviousness, resulting in more runoff. Consequently, the risk of urban flooding is considerably increasing in drainage systems with limited capacity, but also networks with current excellent performance will be overloaded in upcoming years (Swedish Commission on Climate and Vulnerability, 2007).

According to the models carried out by the Swedish Commission on Climate and Vulnerability (2007), the western coast of Sweden will specially suffer a sharp increase of the precipitation and its intensity. This means that the city of Gothenburg will likely register more frequent and severe flooding events. Therefore, one challenge that water management authorities must face is the protection of areas under flooding risk, but also the analysis of potential upcoming districts threatened by flooding. The organisation on charge of the water utility in urban areas in Gothenburg is Gothenburg Water and they have started the work of flooding analysis and prevention. Of special importance has shown to be Lundby-Kyrkbyn area since several properties have been affected by basement flooding during different heavy intense rainfall events (Gothenburg Water, 2006). The sewage system was overloaded by an excess of runoff. This involves that the water could not be conveyed towards the outlet point since the pipe capacity was temporarily exceeded. As a result, some properties had their basements flooded due to a combination of surcharge flow and backflows in the network. Inspections, system performance simulations and infrastructure improvements have already been carried out by Gothenburg Water in order to optimize the sewage system performance in Lundby area. However, many properties have been recently flooded despite of the implemented measures.

In conclusion, there is a need of studying the stormwater management in Lundby-Kyrkbyn area in order to design the required improvements in the sewage system and, consequently, decrease the risk of basement flooding.

1.1 Aim of the project

This thesis has two main goals: identify the critical parts of the sewage system in Lundby area with risk of flooding (specially the properties which have been already flooded) and analyse the feasibility of different possible solutions for this problem. This study has been conducted in collaboration with Gothenburg Water and it is intended to be a preliminary analysis for further investigations.

The current network performance is analysed regarding stormwater excess management. The identification of critical parts of the sewage system with basement flooding risk will be carried out in today's situation, as well as in potential future scenarios. The performance of the system is simulated taking into account the upcoming changes in precipitation and land use to analyse how the system will stand the effects of climate change and urbanization.

In addition, the feasibility of sustainable drainage systems and other infrastructural modifications in the network are also studied. Thus, an evaluation of how the improvements influence the basement flooding risk is carried out.

1.2 Method

The performance and capacity of the drainage system is simulated by means of a one-dimensional model designed in MIKE URBAN software (DHIb, 2011).

The model that represents the network was provided by Göteborg Vatten from a previous study in the same area. It consists on a hydrological model (simulation of the runoff generation within the area) and a hydraulic model (conveyance of the water within the system). The results obtained in the hydrological simulation are then the input for the hydraulic simulation.

After determining the critical areas, this model is also used as a tool to design the required modifications in the network, as well as to check their effects in the overall performance. The modifications are introduced in the model with approximate hand-calculated required dimensions and characteristics, and then adjusted according to the simulation results.

Figure 1 shows an overall scheme of the work strategy to accomplish the aim of the thesis.

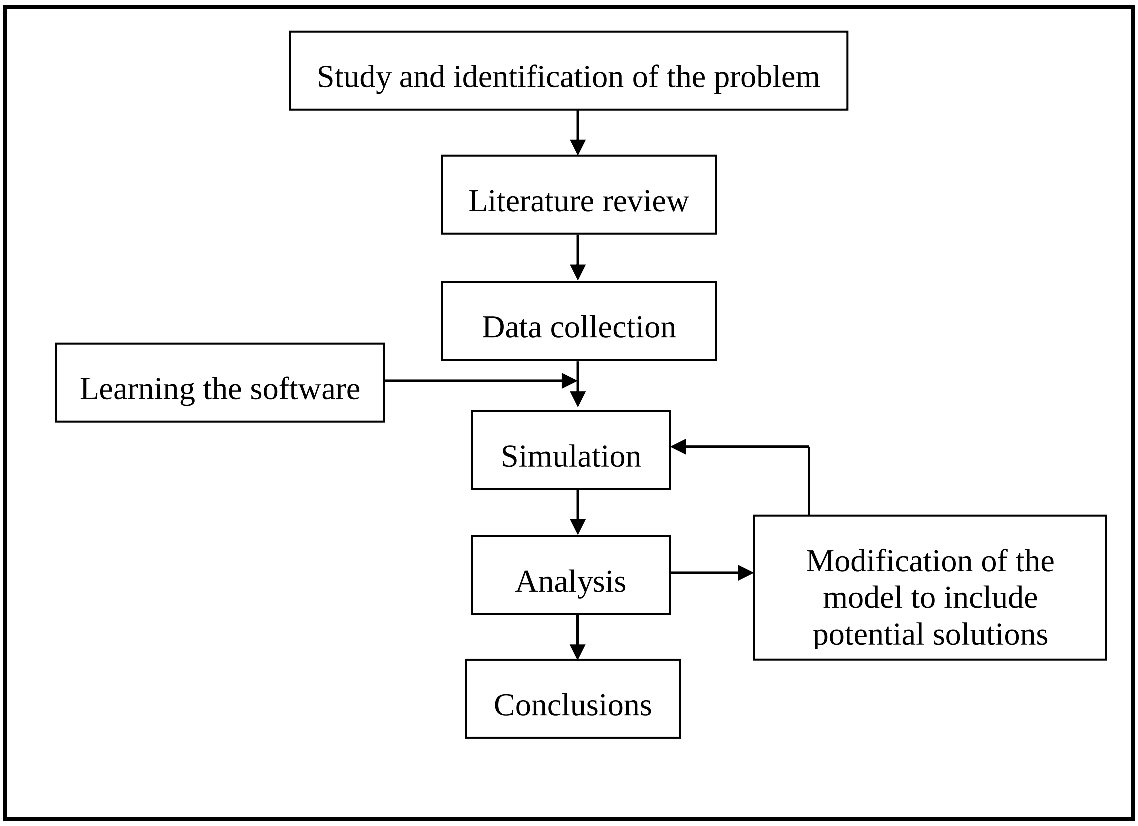

Figure 1. Scheme of the working procedure.

More detailed information about the methodology is included in Chapter 4.

1.3 Limitations

The main tool used in this study is the software MIKE URBAN and the model of the network created with this program. Hence, the results are limited to the calibration of the model and the accuracy of the simulations of this program.

The model used in this study as a base tool was created in 2006 and it was calibrated and evaluated for previous projects. However, the network has suffered multiple modifications since then, which are not included. In order to update the model, some actions undertaken by Gothenburg Water (basically pipe diameter enlargements) were included in the model, although just those ones reported in the documents of the previous flooding investigations in the area.

The potential solutions evaluated in this thesis are just some alternatives that could be implemented in the network, although there are other multiple alternatives that could have been studied. However, this thesis was planned to be made during a 20 week semester and the number of simulations was limited to the time availability. In addition, the simulations of the one-day rains were reduced to 6 hours when the peak discharge occurs (three hours after and three hours before) since the entire simulation of 24 hours takes about 30 minutes.

1.4 Outline of the thesis

This report is divided into 8 Chapters. After the introduction, Chapter 2 contains general background information regarding stormwater management and flooding, as well as guidelines in the field and some hydraulic theory.

Chapter 3 presents the social, urban development, topographical and hydro-geological characteristics of Lundby-Kyrkby area, along with a detailed description of the current sewage system. In addition, this chapter includes an analysis of the flooding problem in this case study.

In Chapter 4 it can be found a description of MIKE URBAN software, the base tool of this study. In addition, the characteristics of the model used for modelling the flooding events and the simulations carried out in this thesis are also included.

In Chapter 5 the obtained results from the simulations are presented, both the runoff and the network results.

Chapter 6 consists of two sub-chapters. The first one contains theoretical information about the evaluated modifications to improve the network, while the other one includes the results obtained after including these changes in the model and have simulated the response of the system.

Finally, the discussion about the obtained results can be found in Chapter 7, while the conclusions drawn in this study are included in Chapter 8 along with the suggested further investigations.

2 Background

2.1 Stormwater and sewage system

The natural water cycle is altered by human activity in two main forms: water is extracted for water supply for human activity and the natural drainage is altered by the shift in land use with more impervious areas (Butler & Davies, 2000). Hence, it raises the need to drainage of wastewater and stormwater. However, stormwater becomes more important when it comes to flooding since the quantity of water is much higher.

The stormwater results from all kind of precipitation (snow melt, rainfall, etc...) and comprises the water flowing in the surface (Butler & Davies, 2000). Therefore, the characteristics of both the rainfall and the catchment area represent important factors in the stormwater properties. Indeed, part of the water of the rainfall goes to initial losses as interception, depression storage, infiltration and evapotranspiration. The remaining water is the runoff (Durrans & Haestad Methods, 2003).

An important social aspect is to maintain public health and safety, hence an efficient drainage of stormwater and wastewater it is essential to avoid impact of flooding on life and property. In addition, the current environmental awareness involves the protection of the receiving waters from the pollutants that may be dragged by water flowing in the surface during heavy rain events (Viessman et al., 2009).

Nowadays there are two types of sewage network system, although three types of pipelines. The combined system conveys wastewater and surface run-off in a single pipe, which in case of high flows relies on combine sewer overflows (normally called CSO's) to evacuate water from the system and thereby relieve the amount of water to manage by the treatment plant. The negative aspect is the spill of untreated water to the watercourses (UNEP, 2004). The separated system comprises two separate pipelines for waste and stormwater protecting from flooding in the basement and floors of houses in low-lying during extreme rainfalls, as well as avoiding the release of pollutants into the environment (EPA, 1999). Stormwater is normally less polluted than sewage water, so that it can be led to detention basins or watercourses saving energy and cost, whereas wastewater requires a deeper treatment.

2.2 An overview on flooding

In Sweden, flooding events do not represent massive disasters, but substantial material damage in infrastructures and properties. They are commonly caused by heavy rains or snowmelt, as well as overflowing of rivers and lakes during summer (SMHI, 2012).

Figure 2 shows the number of extreme rainfall events with at least 90mm of precipitation over 1000 km^2 for a 24-hours period in Sweden during last decades. As can be seen, the tendency is slightly increasing since the 70's with the highest number of events reached in the last decade.

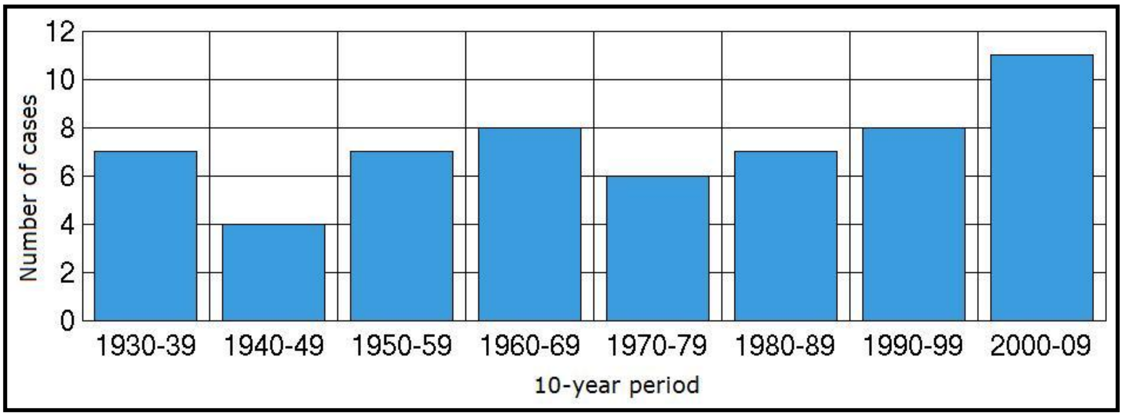

Figure 2. Number of extreme rainfall events in Sweden (SMHI, 2012).

A flood can be described as an event with extreme runoff water (EEA, 2001). A general and fairly common classification is based on the geographical area (rural or urban flooding) in combination with the water body which is responsible of the flood (coastal, river, flash precipitation, groundwater or sewer flooding) (Ashley et al., 2007). Enormous damages can be caused by floods, but they can be worst in the case of urban floods. Human and material damages, breaks down on the drinking water or electricity supplies are some of the undesirable consequences.

Particularly, Lundby-Kyrkbyn area is affected by a combination of flash and sewer flooding. Flash flooding is caused by heavy, short and intense rainfalls which result in high speed flowing stormwater and rising flood waters. Sewer flooding is the consequence of either the system failure or the insufficient capacity to convey high quantity of stormwater during heavy rainfall events (EEA, 2001).

The risk of flooding is determined by natural factors, along with human intervention (EEA, 2001). In the Table 1 below can be seen some driving forces leading to more frequent and severe floods.

Table 1. Factors contributing to flood (WMO, 2008)

Meteorological Factors	**Hydrological Factors**	**Human factors aggravating natural flood hazards**
Rainfall Cyclonic storms Small-scale storms Temperature Snowfall and snowmelt	Soil moisture level Groundwater level prior to storm Natural surface infiltration rate Presence of impervious cover Chanel cross-sectional shape and roughness Presence or absence of over blank flow, channel network Synchronization of runoffs from various parts of watershed High tide impending drainage	Land-use changes (e.g. surface sealing due to urbanization, deforestation) Occupation of the flood plain obstructing flows Inefficiency or non-maintenance of infrastructure Too efficient drainage of upstream areas increases flood peaks Climate change affects magnitude and frequency of precipitations and floods Urban microclimate may enforce precipitation events

Among all the factors, of importance are the changes in the urban planning and climate change since they are the today’s most influential factors determining the risk of urban flooding (Swedish Commission on Climate and Vulnerability, 2007).

In following subchapters the drainage system with regards to stormwater conveyance and the surcharge on it will be further described, along with the effects of climate change and the urbanization process in the runoff generation.

2.2.1 Effects of climate change in flooding events

The climate in Sweden is classified as temperate moist, characterized by abundant precipitation during the whole year. However, the climate has changed over the last decades as a result of the global warming, especially in the recent years, leading to a rise of both temperatures and precipitation (Swedish Commission on Climate and Vulnerability, 2007).

Climate change is induced by greenhouses gasses, in particular CO_2 emissions. Even if the concentration of greenhouse gases decrease in the atmosphere or, at least remains the same generation rate, climate change is likely to continue affecting the global warming for many decades (Swedish Commission on Climate and Vulnerability, 2007).

According to the predictions, the temperature will rise up from 3°C to 5°C by 2080s in comparison with 1990 (Swedish Commission on Climate and Vulnerability, 2007). Moreover, precipitation is the major parameter within climate change causing the greatest flood impacts (Semadeni-Davies et al., 2008). The annual precipitation in Europe is estimated to increase in a 1% to 2% per decade (IPCC, 2001). In addition, intensity and frequency increment of rain events are expected, especially in the west coast of Sweden due to the movement of low pressures from west to east. In fact, the precipitation depth during days of heavy rainfall will increase around 20-50 mm over the average by the 2020's in the west coast, mainly during the driest seasons (Swedish Commission on Climate and Vulnerability, 2007).

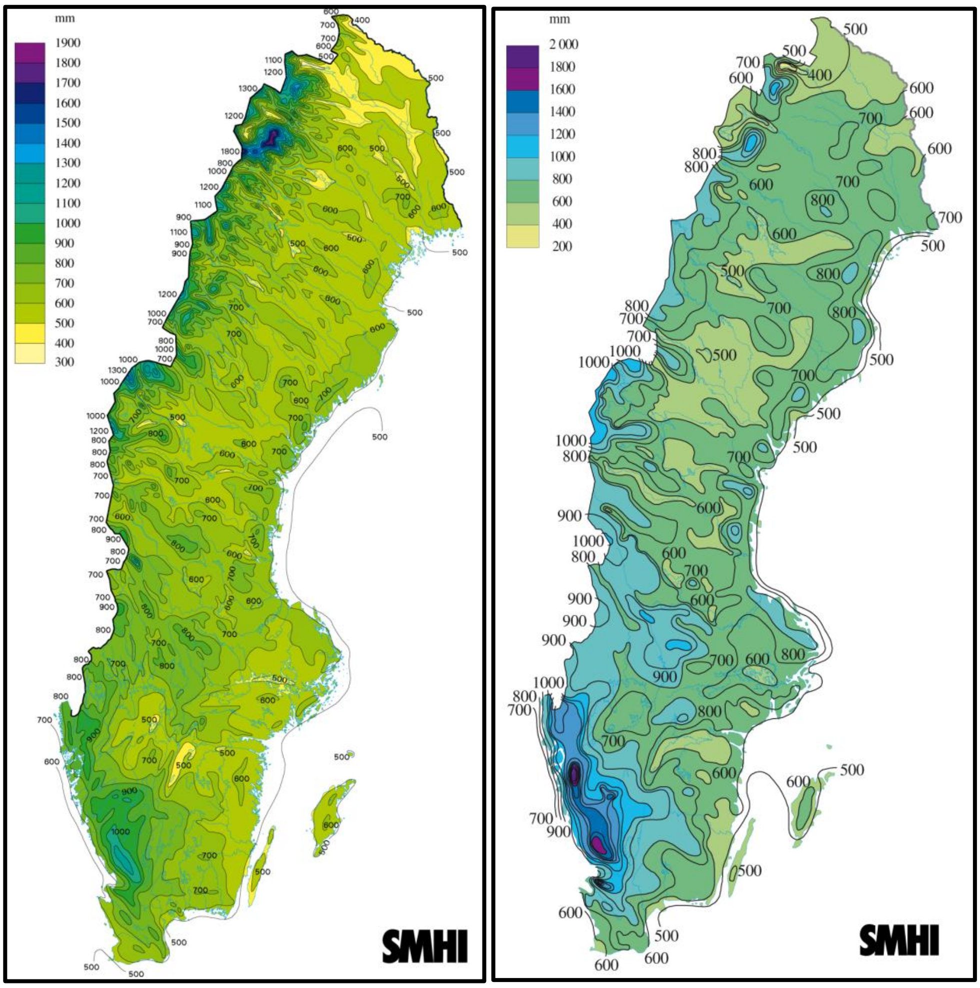

Figure 3. Evolution of precipitation (SMHI, 2012).

The annual average precipitation was 855 mm during the period 1961-1990 in Göteborg (SMHI, 2012). In the most probable case of future emission of CO_2, the forecasts show an annual average precipitation of 930 mm during the period 2011-2040. In such a case, an increment of almost 9% of precipitation is predicted (Gustafsson, 2010). Figure 3 shows the evolution along the years from an average value estimated during 1961-1990 and an average value of the previous decade, specifically in 2008.

Moreover, the runoff will increase as a direct consequent of the precipitation modifications. The 100 year flow (probability of flow reaching this flow value once every 100 years) will suffer and important increment. Sewer system will be heavily burdened due to the inflow increments across the weak seasons: autumn, winter and spring when the ground is saturated and the evaporation rates are low. In addition, enlargements of the base flows have knock-on effects such as sediment transport (Semadeni-Davies et al., 2008).

2.2.2 Effects of urbanization in flooding events

The evolution of the land use is much related with urban development and the increment of floods derived from it. In the undeveloped areas the water coming from precipitation infiltrate in the soil filling the holes between particles until the storage capacity (saturation) is fulfill. After that, the runoff generation starts on the surface.

However, within urbanized areas the paved and other impervious surfaces hinders the capacity of the soil to absorb water. Consequently, the velocity of the runoff is increased leading to sharp peak discharges and greater amount of water in the surface (EPA, 2003). Figure 4 show the influence of urbanization in the runoff generation. As can be seen, the water cycle balance is modified since the groundwater table level decreases and the runoff is raised instead.

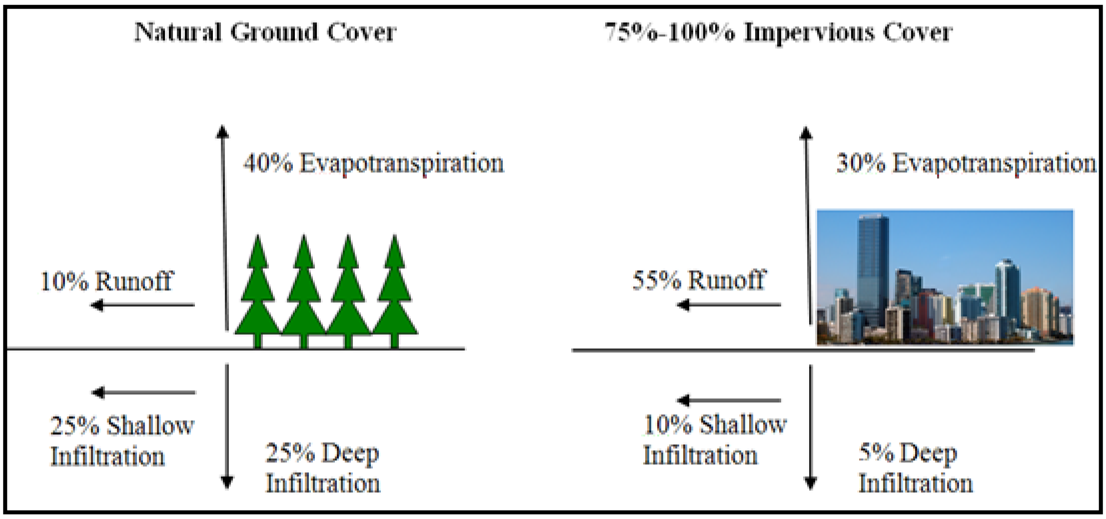

Figure 4. Influence of urbanization in runoff generation (Modified from EPA, 2003).

But not only the quantity of water is modified, water quality is also affected. Organic and nutrient pollutant concentration in the water increases considerably if the water flows above the surface. As a result, the increasing pollutant concentration decreases the possible water uses (Choi, 2004).

2.2.3 Under-designed system

Göteborg sewer system was mainly built during the middle of the last century. The network was mainly built as a combined system until the 1950's.Thereafter, sewage network was built up mainly as a separated system which was considered as more suitable. Most parts of the sewer system (~50%) dated back from the 1960's and 1970's (Malm, 2009). The pipeline lifetime depends on the material and ground conditions, among other factors, but as an average it could be estimated as 100 years. Hence, theoretically the network should be renewal about 2060. Nevertheless, as many lines operate correctly, it is difficult to estimate the rate of renewal for the following years (Malm, 2009). As an average, the current rate of renewal of waste water and stormwater pipes in Sweden is lower than one percent per year (Swedish Commission on Climate and Vulnerability, 2007).

An important cause of flooding is the insufficient capacity of current sewer systems. The combination of increasing population that leads to a change in the wastewater production patterns and the higher rates of stormwater to be conveyed result in a high risk of overloading the capacity of the system (EEA, 2001). Consequently, overburden networks are exposed to the emergence of surcharge that may cause flooding.

2.3 Guidelines

The main current legislation regarding the assessment and management of flood risk in Europe is the Directive 2007/60/CE (European Parliament, 2007), where the following statement is included: "Floods have the potential to cause fatalities, displacement of people and damage to the environment, to severely compromise economic development and to undermine the economic activities of the Community".

This directive is mandatory for every EU country, and should be able to assess and reduce flood risk in order to protect human health, environment, properties and economic activities. To fulfil with succeed this document three main measures must be considered: complete a preliminary flood risk assessment, flood hazard maps along with flood risk maps and flood risk management plan. This directive affects every kind of floods, from those referred to river banks and coastal to urban floods caused by surface runoff or the overload of the sewage system. These steps will be followed in this thesis, starting with the situation of floods in the area, subsequently the determination of hazard and risk points within the network and finally the measure to be adopted in order to manage them.

Several measures can be adopted, but most of them will consider the Directive 2000/60/CE (European Parliament, 2000). This document has the target of maintain and improve the aquatic environment. Stormwater, in its path until the inlet points, drag an important load of pollutants that either finish into the wastewater treatment plant or are discharged directly into the river without any treatment.

Göteborg is involved in an international cooperation to accelerate sustainable development and related domestic policies named Agenda 21. It is United Nations programme of local actions plans where everyone has the chance to participate and influence on the conservation of natural resources and reduce pollutants in air, soil and water. Göteborg council formed in 1986 an Environmental Policy Management Group to identify responsibilities in order to take care in all municipal departments (UnHabitat, 2012).

2.3.1 Swedish regulation for the design of public sewer network (P90)

The main trigger in a flooding is the precipitation and for this reason it is important to evaluate different events. The return period is the recurrence interval between events that match or exceed a given magnitude. It depends on the security level for flooding required by society, the precipitation pattern (intensity and annual recurrence) or flows exceeding structures capacities.

The design and evaluation of sewer system in Sweden is regulated by the "Swedish regulation for the design of public sewer network", usually known as P90 (Svenskt Vatten, 2004). Figure 5 shows a table extracted from P90 regarding the design rain used for wastewater systems.

Type of area	Design state of filled pipe		Return period for pressure line in	
	Stormwater pipe	Combined pipe	Ground level of stormwater pipe	Basement level combined pipe
Not confined* area outside city centre	1 year	5 years	10 years	10 years
Not confined* area within city centre	2 years	5 years	10 years	10 years
Confined area outside city centre	5 years	10 years	10 years	10 years
Confined area within city centre	10 years	10 years	10 years	10 years

* Not confined area refers to an area where runoff is generated by surface gravity flow through gutters, ditches and streams (watercourses)

Figure 5. Design rain for wastewater systems. Swedish regulation: P90 (Pettersson, 2011)

Depending on the location of the network design (confined/not confined or within/outside city) and the requirement to be fulfil (pipe type), this regulation recommend the return period of precipitation event in order to ensure a normal operating status (Svenskt Vatten, 2004).

For the case of Lundby-Kyrkby where runoff is generated by surface gravity flow through gutters, ditches and streams (not confined areas) the return period in the case of pipe filling requirements are 2 or 5 years for stormwater and combined pipes respectively. In addition, the pressure lines in every node in the network should not be higher than the basement level for the case of rainfall event of 10 years return period. However, depending on the characteristics of the water conveyed by the pipe, this requirement could be an exception. As Figure 6 shows, if the node belongs to a stormwater pipe the pressure line is allowed to reach the ground level, since this kind of system is not connected to the houses or industries (users). Nevertheless, in combined system the pipes are connected to the buildings. Then, in case of heavy rainfall events if the pressure line exceeds basement floor levels, backflows may occur creating the risk of basement flooding.

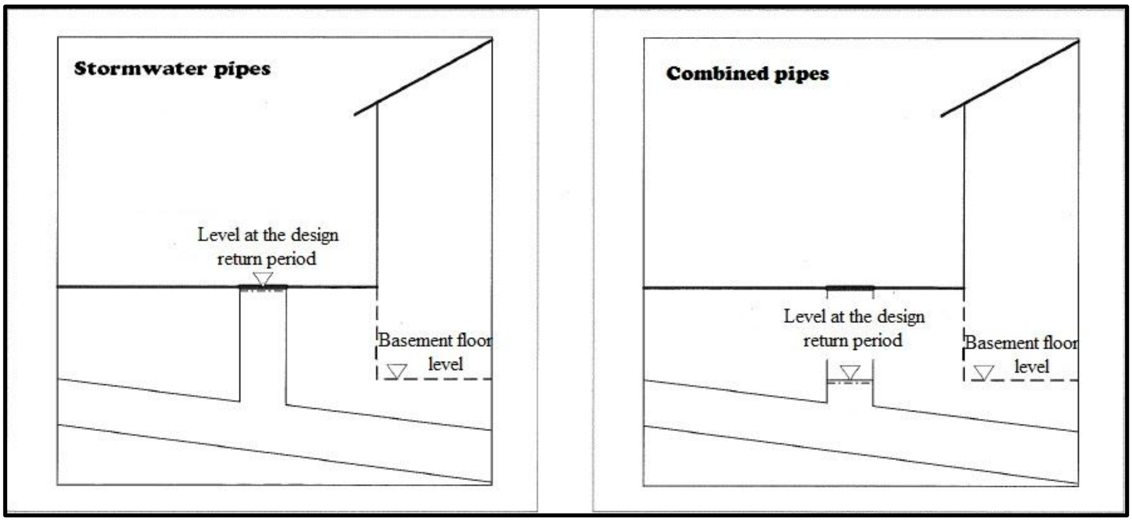

Figure 6. Required level at the design of 10 years return period rain for stormwater pipes and combined pipes (Svenskt Vatten, 2004).

2.4 Theory: hydraulic background

The engineering calculations used in the design and evaluation of drainage systems are, to a large extent, applications of the fundamental physical laws of conservation of mass, energy and momentum (Hager, 2010).

Flow in drainage systems can be classified in mainly two types: pipe flow running under pressure and open channel flow, characterized by water conveyed by gravity with a free surface at atmospheric pressure. However, when it comes to sewer systems the flow is a combination of both of them, resulting in a part-full pipe flow. Water runs by gravity with a free surface and only fills the pipe area when the capacity of the pipe is exceeded (Butler & Davies, 2000).

The most basic principle of the hydraulics of pipelines is the conservation of mass (Larock, Jeppson & Watters, 1999). This principle is expressed by means of the continuity equation for steady incompressible flows in a conduit, see Equation (1).

$$Q = A_1 \cdot V_1 = A_2 \cdot V_2 \tag{1}$$

where: $Q = discharge\ (\frac{m^3}{s}); A = cross\ sectional\ area\ (m^2);\ V = mean\ cross-sectional\ velocity$

The linear momentum principle is governed by the impulse-momentum equation. Equation (2) is the momentum equation for steady incompressible flow in a pipe (Larock, Jeppson & Watters, 1999).

$$\sum F = \rho \cdot Q \cdot v \tag{2}$$

where $F = external\ forces\ acting\ in\ the\ pipe\ surface$ [N], $\rho = density\ of\ water\ [\frac{kg}{m^3}],\ Q = flow\ [\frac{m^3}{s}],\ v = velocity[\frac{m}{s}]$

The flow energy, in terms of head depth (m), consists on the sum of three terms (pressure head, velocity head and datum head), see Equation (3). This equation is also referred as to Bernoulli´s Equation (Butler & Davies, 2000).

$$H = \frac{p}{\rho g} + \frac{v^2}{2g} + z \tag{3}$$

where $H = hydraulic\ head\ [m],\ p = pressure\ [Pa],\ \rho = density\ of\ water\ [kg/m^3],\ g = gravity\ [m/s^2],\ v = velocity\ [m/s],\ z =$ potential head $[m]$

Water always flows from regions with higher energy level to regions with lower energy level (Butler & Davies, 2000). The energy grade line (EGL or EL) represents the total amount of energy available in the system, i.e. it is drawn a vertical distance from the datum equal to the sum of the potential head, velocity head and pressure head. Therefore, to calculate the energy grade line it is necessary to determine the losses through the system. The hydraulic grade line (HGL) represents the water level in an open channel, whereas it is the elevation to which water would rise in a pressurized conduit. It is calculated by subtracting the velocity head term from the energy grade line (Brown, Sten & Warner, 2009).

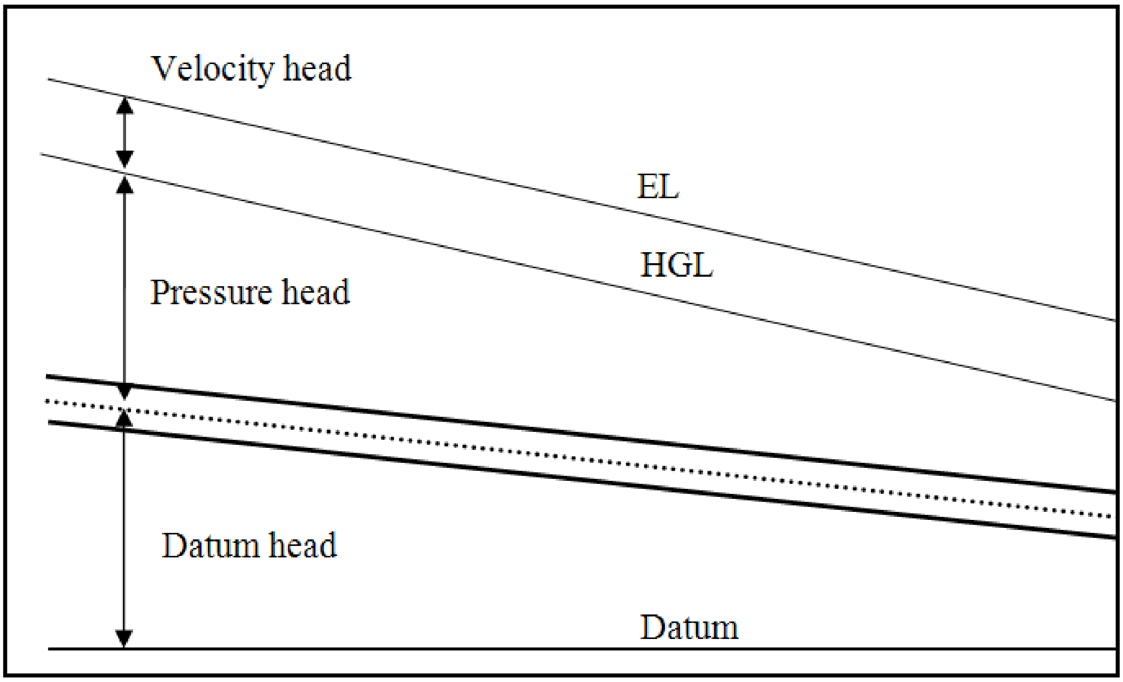

Figure 7. Hydraulic and Energy Grade Lines in pipe flow (Modified from Butler & Davies, 2000).

The energy losses in a pipe consist of friction losses and local losses. Friction losses involve forces between the liquid and the solid boundary, while local losses account for disruptions to the flow at local features. The friction losses are represented by the Darcy-Weisbach Equation (4), valid for both laminar and turbulent flow (Durrans & Haestad Methods, 2003).

$$h_f = \frac{\lambda L}{D} \cdot \frac{v^2}{2g} \tag{4}$$

where $h_f = head\ loss\ due\ to\ friction\ (m), \lambda = friction\ factor,$

$L = pipe\ length\ (m),\ D = pipe\ diameter$

When it comes to urban flooding, two important hydraulic concepts arise: surcharge and backflow. A sewage system is surcharged when its capacity is exceeded, i.e. it receives greater volume of water than the system can convey. As a result, the water level rises upstream due to the network overloading (Butler & Davies, 2000). If the energy line reaches higher points downstream than upstream, water may change the regular flow direction leading to backflow, see Figure 8.

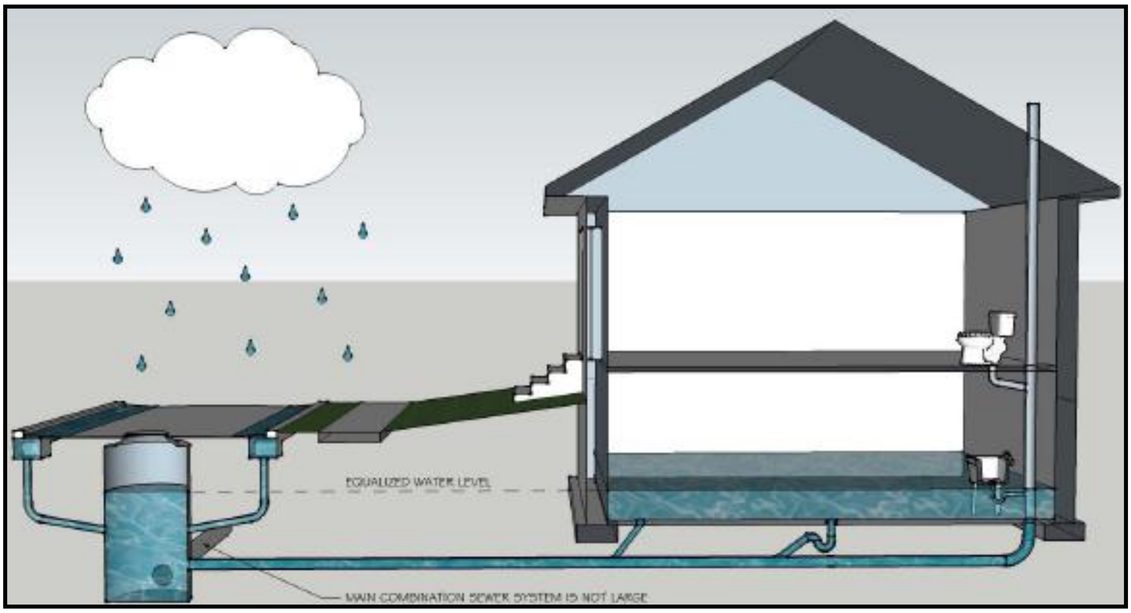

Figure 8. System surcharge and backflow effects (Zoulek, 2010).

Figure 8 shows an example of basement flooding due to the overloading of the sewage system which results in backflow into the properties.

3 Case Study: Lundby-Kyrkbyn Area

3.1 General description

Lundby-Kyrkby area is one of ten districts in Göteborg. It is located in the island of Hisingen, in the north part of the city, see Figure 9.

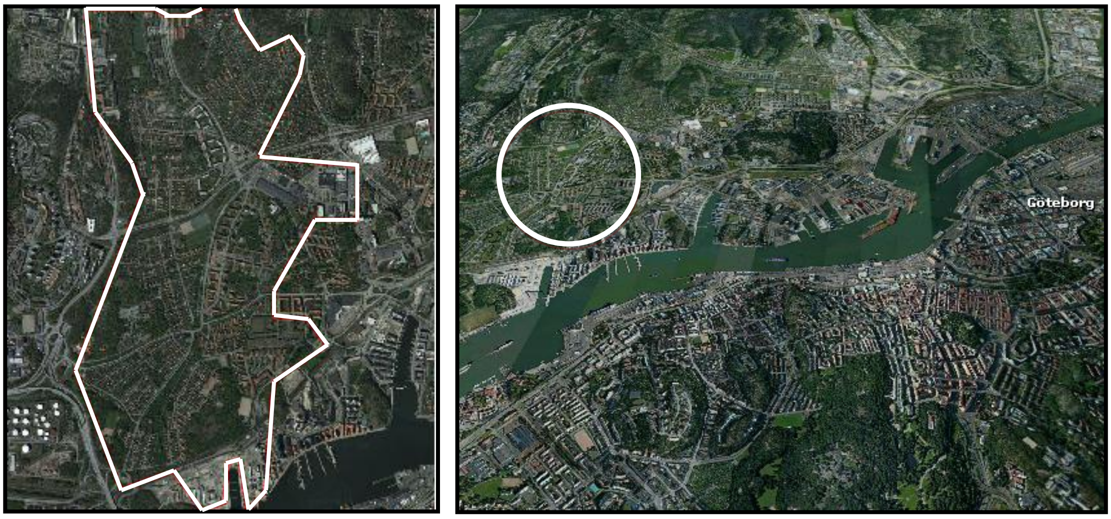

Figure 9. Location of the case study (Google Maps, 2012)

With a population of about 42,000 inhabitants, it is the smallest district in Göteborg. Nevertheless, it is the neighbourhood that has grown the fastest in the recent years due to new housing and business areas. In fact, the population is expected to reach 46,000 inhabitants by 2015 (Göteborgs Stad, 2012).

The majority of the houses and buildings were built in the mid-1900s, although more of them have been built after 1990s than before 1940s. The dominant type of construction is mostly apartments in 3 or 4- storey buildings and, to a lesser extent, single-family houses. In fact, there are 23,000 housing units in the neighbourhood of which over 19,000 are in apartment buildings (Göteborgs Stad, 2012). In addition, some industries are established in the area, of which Volvo Lundby is the most extensive in space.

Moreover, the most important infrastructures in the case study area are three main roads and a tunnel. Two of the roads and the tunnel connect the area of Älvsborgs Bridge with the eastern part of the island, crossing the district from west to east in four different locations, while the third road runs towards north.

The case study district is bounded by areas whose characteristics differ considerably. The east edge consists on a highly urbanized area, whereas the northern part is much less dense. The western part in surrounded by buildings and an industrial area and, a bit further, there is an extent forest. The Göta Älv is the boundary in the south.

3.2 Hydrogeology and topography description

Lundby-Kyrkby district is characterized by a highly flat relief with small hills, see Figure 10. The general slope is about 0.9%, dipping from the northern part towards Göta Älv. The height above sea level in the north part of Lundby reach 45 meters, while in the harbour zone is about 12 meters. The steeper areas are the west edge and the north part, which consist mainly on small hills bordering the urbanized area.

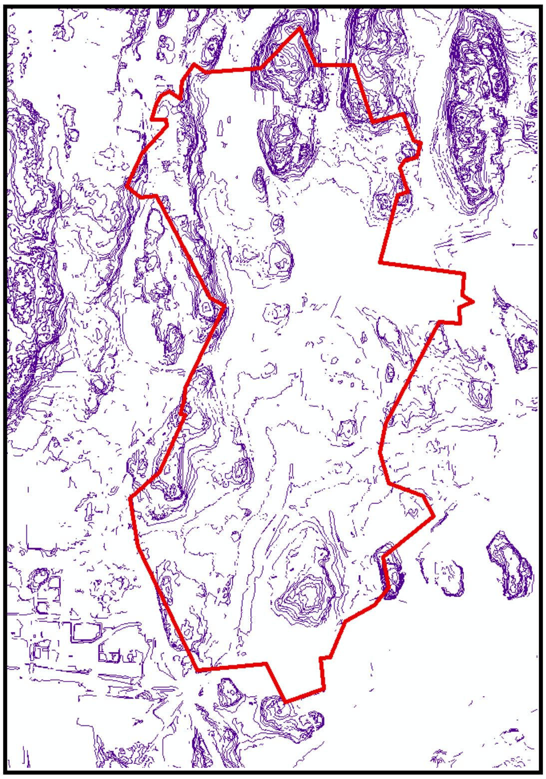

Figure 10. Topography of the area (Gothenburg Water, 2012).

The geology in the area is quite uniform. The bedrock consists mainly on a combination of granodiorite and granite, which both are acidic intrusive rocks (SGU, 2012). However, the bedrock tends to be porphyritic or augen-bearing in the most northeaster part. The soil is basically glacial clay with some bedrock outcrops in the highest parts. In addition, wave washed sediment as gravel or sandy till are present in small areas in the middle-southern part.

The groundwater flows from north towards Göta Älv. Depending on the geology, the exploitation capacity varies from 200-600 l/h in bedrock to 1-5 l/s in gravel and sandy soils located under impermeable sediments (SGU, 2012).

3.3 Sewage system

The sewage system is designed to collect, convey, detain, treat and discharge both wastewater and stormwater. The largest proportion of the sewer network in Göteborg was built between 1950 and 1980. The commonly used material was concrete, although others as polyethylene, polypropylene or PVC were also used to a lesser extent (Malm, 2009). The latest modifications were done using polypropylene Ultra Rib. Most part of the network is still combined, despite of the fact that the trend is to replace the combined system for the separate to improve water management and quality.

In Lundby-Kyrkbyn area the sewer system is mostly combined, but some parts have separate pipelines for storm and wastewater, see Figure 11. Nevertheless, new plans for the renewal of the network, as well as the need to solve flooding problems are leading to the installation of different pipes for stormwater and sewage. The combined part of the system is mainly from the 50's, whereas the duplicated part was built during the 60's and the 70's (Malm, 2009). The sewage system was designed to use gravity as the conveyance power.

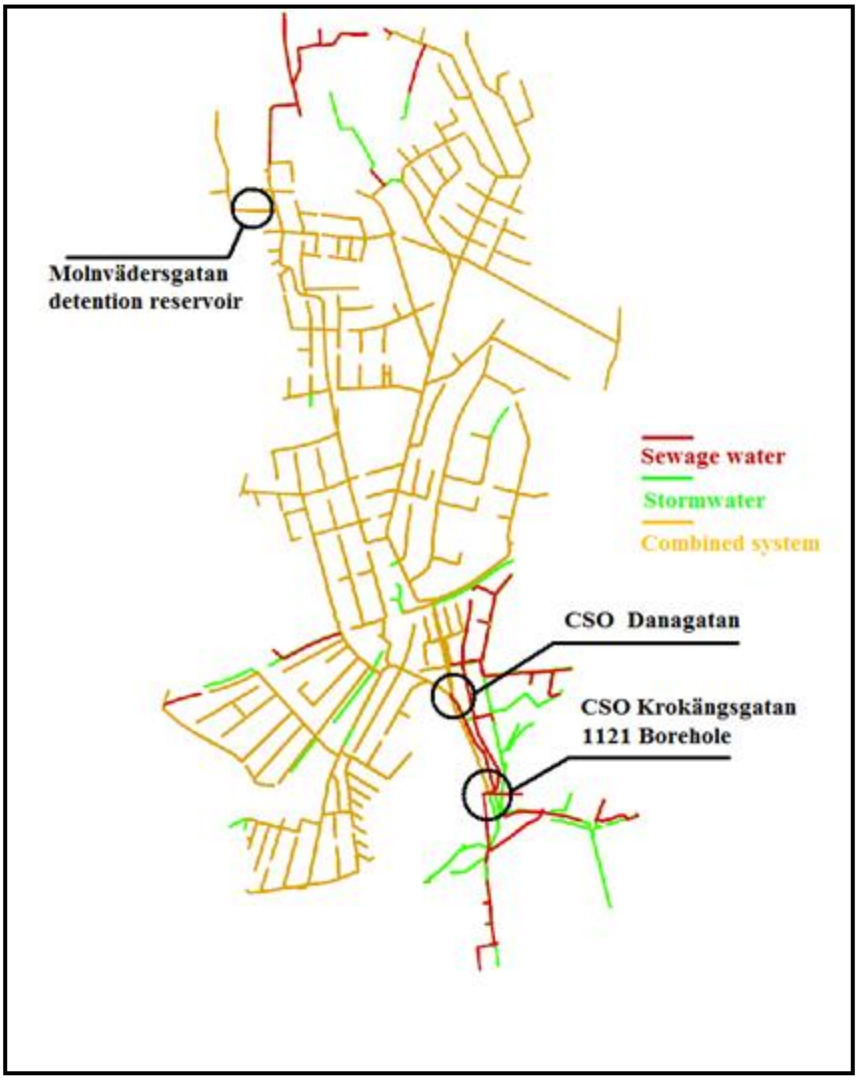

Figure 11. Components of sewage system in Lundby-Kyrkbyn area (Gothenburg Water database, 2012)

Four types of components are mainly present in the network: inlets, manholes or junctions, pipes and outlets. In addition to the basic components of the system, it also counts with two combined sewer overflows and a reservoir.

The system is connected to Gryaabs WWTP through the borehole BH1121, located nearby Krokängsparken overflow in the southern part of the district. When the capacity of the system is overloaded, there is an additional outflow point in Göta Älv that discharges the excess of water coming from the overflows, but also stormwater from the southern part of the area.

3.3.1 Pipelines

The pipes are made by concrete to a great extent, although other materials as plastics are implemented in new improvements. The dimension of the pipes differs considerably within the system according to different flow conditions depending on population, runoff and other factors. Thus, the smallest diameter is 150 mm, used in the connection of the properties to the public system, and the greatest pipe is the outlet into the Göta Älv with a diameter of 2000 mm.

3.3.2 Combine sewer overflows

There are two CSO's in the area: BB4526 – Krokängsparken and BB4525 – Danagatan.

The CSO located at Krokängsparken is the third largest CSO in Göteborg. The average DWF is about 30 l/s, although the discharge may reach 220 l/s when it is working at its maximum capacity (Jiverö & Torstensson, 2006). The weir is 1.90 meters long.

In the CSO located at Danagatan water is diverted by a 5.5 wide weir which has two heights: 16.66 and 16.43 meters (Gothenburg Water, 2012).

3.3.3 Molnvädersgatan underground detention reservoir

The storage facility is located in the northwester part of the catchment. It is excavated in bedrock underneath a parking lot and it has a capacity of about 1000 m^3. The function of this structure is to detain the water and attenuate the outflow rate in order to avoid overloading the system with an excess of stormwater (Jiverö & Torstensson, 2006).

3.4 Previous flooding events

Some properties had their basements flooded in previous heavy rainfall events due to insufficient capacity of the sewage pipelines or inadequate performance of the system see Figure 12.

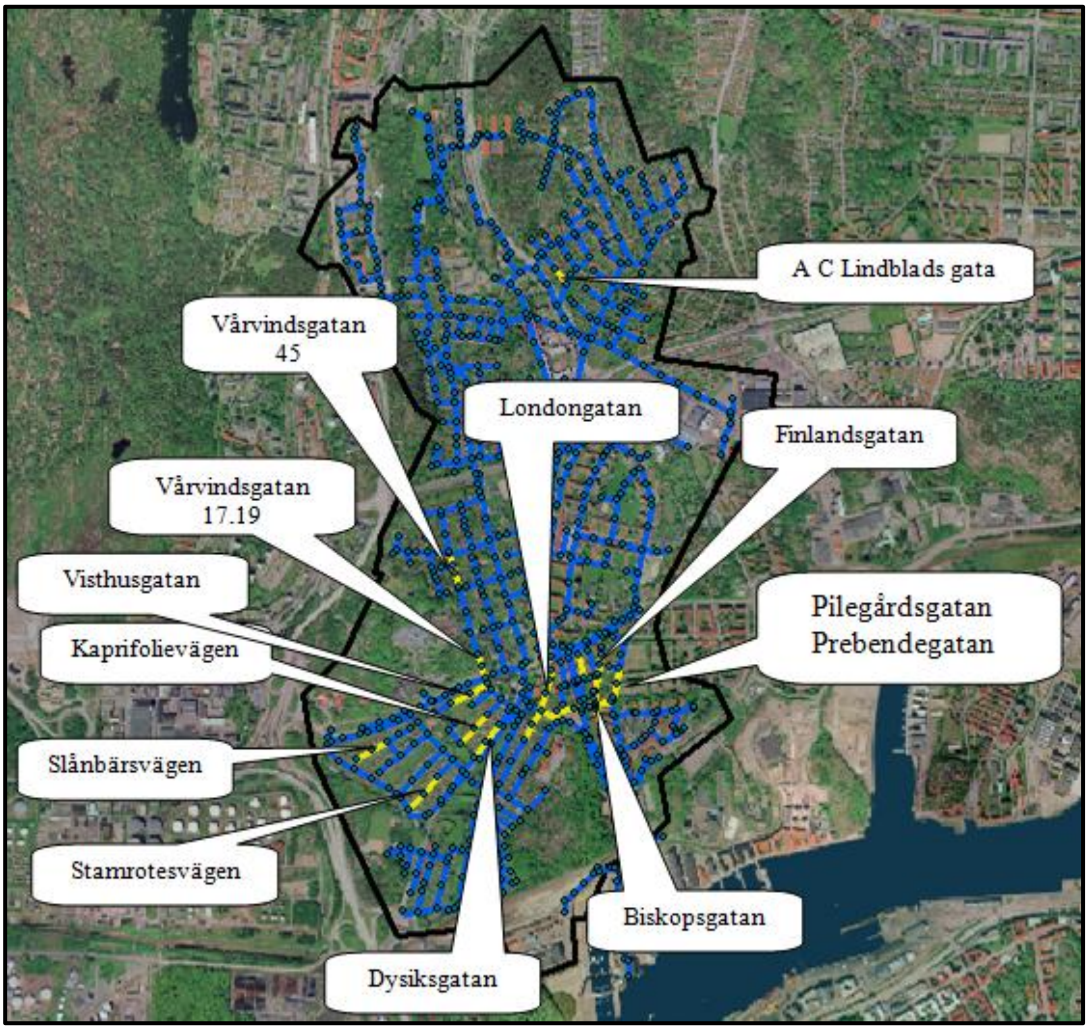

Figure 12. Location of the properties (Gothenburg Water, 2006).

The most severe flood occurred 27th August of 2006, resulting in 34 properties affected. However, most of these basements were flooded previously and subsequently. Table 2 below shows a compilation of the flooded properties.

Table 2. Compilation of flooding in Lundby-Kyrkby area. Source: Gothenburg Water Solen X Database.

	2011	**2010**	**2006**	**1997**	**1996**		**2011**	**2010**	**2006**	**1997**	**1996**
A C Lindblads Gata, 5						Stamrotesvägen, 27					
Biskopsgatan, 17						Visthusgatan, 8					
Dysiksgatan, 17						Vårvindsgatan, 17					
Dysiksgatan, 19						Vårvindsgatan, 19					
Dysiksgatan, 23						Vårvindsgatan, 45					
Finlandsgatan, 2						Biskopsgatan, 5					
Kaprifolievägen, 18						Biskopsgatan, 7					
Kaprifolievägen, 22						Pilegårdsgatan, 32					
Kaprifolievägen, 24						Pilegårdsgatan, 30					
Kaprifolievägen, 26						Prebendegatan, 26					
Kaprifolievägen, 20						Prebendegatan, 24					
Kaprifolievägen, 16						Prebendegatan, 30					
Londongatan, 19						Prebendegatan, 13					
Londongatan, 17						Prebendegatan, 18					
Londongatan, 21						Prebendegatan, 15					
Londongatan, 10						Prebendegatan, 19					
Londongatan, 8						Prebendegatan, 20					
Slånbärsvägen, 26											

After the flooding occurred on 27/08/2006, an investigation of the network system was carried out to determine the causes of the flooding, as well as to identify the required improvements in the system. The system was surveyed in order to identify operation faults, although the investigated areas were reduced to the flooded ones. In addition, Göteborg Vatten and DHI simulated the performance of the system by using a computer model. The following properties were inspected and depending on the cause of failure different proposals or measures were suggested.

- AC Lindblads Gata 5

The inspections found material accumulations along the pipe, so that regular flushing was suggested.

- Biskopsgatan 17

The overflow at Danagatan CSO (downstream) caused the raise of the energy line at Biskopsgatan (upstream). According to the simulations, the pipe can manage a 10-year return period rainfall, so no measure was suggested.

- Dysiksgatan 17, 19, 23

The investions found sediments clogging the pipe. In addition, the simulation results indicated that the pipe can lead a 10-year rain. Consequently, regular flushing were suggested.

- Finlandsgatan 2

The problem was found to be the under-size of the system downstream, which cannot handle a 10-year rain. No measure was suggested since it was the first time that this property had problems with flooding and the main problem is located downstream.

- Kaprifolievägen 16, 18, 20, 22, 24, 26

The investigations found the capacity of the pipes insufficient, while the pipes downstream performance was correct. As a result, a local measure consisting in new pipes with greater diameter in that street was suggested and implemented.

- Londongatan 17,21,10,8

The simulation showed that this line should cope with either a 5- or a 10-year return period rainfall. The conclusion was that the rainfall might be heavier than a 10-year return rain and no modification was implemented.

- Slånbärsvägen 26

The pipes are undersized and cannot handle simulations of 10-years rain. The problem is local, so it was decided to increase the pipe diameter.

- Stamrotesvägen 27

The simulations showed that the problem was the local insufficient capacity of the pipe, so that the suggested measure consisted on increasing the pipe diameter in that street.

- Vårvindsgatan 17, 19, 45

The simulations showed the incapacity of system to handle a 10-year rain, but also the inspections found material accumulations. In the case of the properties 17 and 19, the problem was solved by incrementing the pipe diameters of two stretches which were undersized. However, this measure did not affect property 45.

- Visthusgatan 8

Material accumulation was found during the inspections. In addition, the improvements in the line downstream suggested to solve Vårvindsgatan was supposed to reduce the water level in this street. Consequently, no measure was implemented.

- Pilegårdsgatan

The area is affected by geotechnical problems, since the subsidence resulting from the clay movements is affecting the pipes. For this reason, it was proposed to improve the pipes to guarantee a correct performance.

- Prebendegatan

The failures found in this street were material accumulation (mainly at the beginning of the line but also downstream), burns and transversal displacements in the separate line. However, non-structural changes were implemented.

As it can be seen, all the measures carried out solved the problem locally, avoiding floods in particular cases. Nevertheless, the problem was moved downstream in the network, where the system has to manage larger amounts of water with the same capacity.

4 Flood Modelling

The model is designed to predict how the network system will cope with the forecasted rainfall events. The performance of the structures will be simulated under different scenarios to obtain resulting flows, headwater elevations, water velocity within pipes and so forth. The tool to accomplish this goal is a MIKE URBAN model that simulates the sewage system performance in Lundby-Kyrkbyn area. It was originally created, calibrated and validated by DHI and Gothenburg Water with measurements in previous projects. However, some changes were needed in later stages to simulate the improvements of the system considered in this thesis.

4.1 MIKE URBAN

MIKE URBAN is a software developed by DHI (Danish Hydraulic Institute) to model urban systems based on Geographical Information Systems (GIS). Two modeling tools are integrated in the software: EPANET and SWMM, both developed by the US EPA. These engines are integrated on the modeling of water distribution and in the sewer and stormwater systems, respectively (DHIb, 2010).

Moreover, MIKE URBAN has also the DHI's MOUSE engine, which is its own application. MOUSE, in cooperation with SWMM, enables the modeling of complex hydrology, stormwater drainage systems and sewer networks, as well as the analysis of advanced hydraulics for both open and close conduits. Thanks to this application, different flood return periods can be obtained in different parts of the sewer system, it is possible to determine the main causes of the surcharge of the system, the occurrence of backflow or the insufficient capacity in pipes, among others functions (DHIb, 2010).

As an overview of the program operation, the modeling process can be described in the following steps (DHIb, 2010):

- Definition of the runoff and network data
- Specification of the boundary conditions
- Adjustment of the computation parameters and running the simulations
- Results analysis

It is important to ensure that the computed results make sense with reality. For that reason, the calibration and validation of the model are essential steps in the modeling process (DHIb, 2010).

4.1.1 Hydrological model

The runoff is simulated by means of a hydrological model for urban catchments. MIKE URBAN includes two types of models: the surface runoff model and the continuous hydrological model (DHIa, 2011). On one hand, the surface runoff model considers just the surface runoff generated during a rainfall event, resulting in discontinuous runoff hydrographs. On the other hand, the continuous hydrological model computes the precipitation volume balance including both overland and sub-surface runoff. The most suitable model for an urbanized area affected by heavy and intense rainfall events (this case study) is a surface runoff model, since most of the runoff is generated on impervious surfaces during rains and ceases when the event finishes (DHIa, 2010).

Among the four different types of surface runoff models that the program can handle, the Time-Area model was chosen to be the most suitable one. The data requirements are reduced to three main parameters to control the amount of generated runoff (the initial loss, the size of the contributing area and the continuous hydrological loss), whereas, the shape of the runoff hydrograph depends on the concentration time and the time-area curve (DHIa, 2010).

To compute the runoff the total catchment area is divided in a certain number of smaller cells depending on the simulation time step, see Equation 5.

$$n = \frac{t_c}{\Delta t} \qquad (5)$$

where $t_c = \mathit{concentration\ time}$ and $\Delta t = \mathit{simulation\ time\ step}$

The area of each cell is calculated according to the defined time-area curve and the total area of all cells is the specified impervious area. The time-area curve accounts for the catchment reaction speed and shape, relating the flow time and the corresponding catchment sub-area (DHIa, 2010).

The surface runoff starts when the rain depth reaches the value of the initial loss, which determines the water held in the land surface either in depressions and irregularities or by surface tension. In the same way, the runoff generation ceases once the rain depth is below the initial loss parameter (DHIa, 2011). The volume of runoff in each cell is calculated as a balance of inflow from the upstream cell, the outflow to the downstream cell and the rainfall in the cell. The outflow in the most downstream cell is the surface runoff hydrograph. However, the runoff is then reduced in the impervious surfaces to account for the specified hydrological reduction (evapo-transpiration, imperfect imperviousness, etc.) (DHIa, 2010). The computed runoff will be the load of the collection network.

4.1.2 Hydraulic elements

The pipe network performance is analyzed with the MOUSE Pipe Flow Model engine, which simulates unsteady flow in both free surfaces and pressurized conditions. The computations of the flow conditions in the network are performed by solving the complete Saint Venant equations in several points of the pipes and manholes. The Equations (6) and (7) represent the conservation of mass and momentum, respectively (DHIb, 2010).

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (6)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (\alpha \frac{Q^2}{A})}{\partial x} + gA\frac{\partial y}{\partial x} + gAI_f = gAI_0 \qquad (7)$$

where *Q = discharge [m3/s],* A_f *= flow area [m2], y = flow depth [m], g = acceleration of gravity [m/s2], x = distance in the flow direction [m], t = time [s], α = velocity distribution coefficient,* I_0 *= bottom slope,* I_f *= friction slope*

These equations have some basic assumptions that are not valid in the case of pressurized flow. Nevertheless, it is possible to generalize the equations for free surface flow into pressurized flow in closed conduits by introducing a fictitious slot in the top of the conduit (DHIb, 2010).

4.1.2.1 Links

Links are considered in MIKE URBAN as one-dimensional water conduits that convey water between nodes. In this case, the model just includes closed conduits links (pipes), although there is also the possibility to model open channel links. Links are defined by three main properties: cross-section geometry, bottom slope and friction properties (DHIa, 2011).

The cross-section geometry of the pipes can be defined by either adopting a standard section (circular, rectangular, O-shapped, Egg-shaped or quadratic) or using CRS to define alternative shapes. The slope is determined by the upstream and downstream levels and the length of the pipes. The link material is characterized by a hydraulic friction loss coefficient, which can be either the Manning coefficient, the Colebrook-White coefficient or the Hazen-Williams coefficient. The choice of one hydraulic friction loss formulation is of importance to determine the flow pattern at pipes (DHIa, 2011).

The flow description can be simulated with three different approximations. The dynamic wave approach uses the full momentum equation and includes acceleration forces to fully simulate transients and backflow profiles. The diffusive wave approach just models the friction bed, gravity force and the hydrostatic gradient in the momentum equation, so that it is suitable for backflow analysis when the link bed and wall resistance forces dominate. The kinematic wave approach calculates the flow as a balance between friction and gravity forces, so that backflow effects are not considered (DHIa, 2010).

4.1.2.2 Nodes

Nodes are the start and end points of one pipe in a network. If more than one pipe starts or ends in the same node, it is then called junction. The nodes can represent the circular manholes in the sewage network, other storage facilities as basins or tanks with only one computational point where the water level is measured. The program distinguishes between four types of nodes: manholes, basins, outlets and storage nodes (DHIa, 2011).

Manholes are access points that allow the inspection and cleaning of the system. They are located in changes of direction, heads of runs, changes in gradient, changes in size or in major junctions with other sewers (Butler and Davies, 2000). In the model, they are represented by circular cylinders defined by the bottom levels (invert level), ground level, diameter and shape.

Structures or also called basins are nodes that represent facilities with considerably high volume: non-circular manholes, tanks, reservoirs, basins and natural ponds. They are storage facilities that handle the runoff to reduce peak flows. Hence, the excess water should be stored and released under control afterwards (Butler and Davis, 2000). The volume of the structures is added to the overall system volume and is included in the computations (DHIa, 2011).

The outlet structures are the discharge points where the water is released into water bodies or conveyance systems to wastewater treatment plants (Durrans & Haestad Methods, 2003). In the model, the outlets connect the sewer system with external water volumes whose level is independent to the receiving water, such as rivers, lakes or sea. The outlets are defined by the bottom and the water surface level. The outlets do not modify the flow of nearby links, so that backflow effects are avoided (DHIb, 2010).

Flow conditions in the nodes are important in the overall flow description, since general flow equations are only valid for continuous conduits. The hydraulic conditions at nodes are calculated as water levels and velocity head, which also depend on the effective node area. The calculations are carried out in base of the mass continuity formulation along with the energy balance that accounts for energy losses due to flow disturbances (DHIb, 2010). Local head loss in the nodes can be considered with five different computational models, although in this study only two of them are used: No cross section changes and MOUSE Classic (Engelund) models. No Cross Sectional Change implies that there are not any changes in the cross section of the nodes or the links resulting in no energy loss in the water. The MOUSE Classic head loss simulates the energy loss in a node due to changes in the diameters of links and nodes (DHIb, 2010).

4.1.2.3 Functions

Functions are used to define the functional relation and hydraulic conditions between nodes or in links in characteristic points of the system (DHIb, 2011). In this model, weirs and pumps are used.

Weirs are used to model overflows which discharge part of the water into a recipient when the system is working under extreme flow conditions, but could also be used to simulate internal distribution of the flow within the system. Weirs can be modelled in nodes defined as manholes or structures, but not in outlets. They can be defined with a Q/H relation between the water level in the upstream node and the released discharge or with the built-in overflow formula that depends on the crest elevation, structure width orientation relative to the flow and crest type, see Equation (8) (DHIb, 2010).

$$Q_w = C_H \cdot B \cdot \sqrt{2g} \cdot (H)^{\frac{3}{2}} \tag{8}$$

where Q_w = *flow rate [m^3/s], C_H =level discharge coefficient, B = weir width [m], g = acceleration of gravity [m/s^2], H_w= water elevation above weir crest [m]*

The function of a pump is to add energy to the water where the gravity force cannot convey the flow due to topographical levels (Butler and Davies, 2000). A pump is topographically defined by means of two nodes and its performance is determined by an operation range: start and stop levels, along with the Q/H or Q/ΔH relation curve. To simulate a correct performance of the pump and to avoid a sharp start-up, the pump dynamics have to be dampened so that the discharge capacity is increasing gradually (DHIb, 2010).

4.2 Modelling rainfall

The required rain input data in urban drainage applications depends on the nature of the engineering task. In this study both continuous hydrographs and block rains will be considered.

The rain data is commonly measured as intensity (mm/h) or depth (mm) and it is related to a statistical concept: frequency. The frequency is normally represented as the return period, which is the probability that a rainfall with certain intensity will be exceed or equalled in any year (Durrans & Haestad Methods, 2003).

In this study, two input data were considered: continuous and block rainfall. The continuous rains were obtained from the Publication P104 (Rainfall data for design and analysis of sewage systems, Svenskt Vatten), while the data of the block rains was supplied by Annika Malm from Gothenburg Water. Table 3 shows the data used to create the rain blocks. As can be seen, the duration of the rainfall is inversely proportional to the intensity.

Table 3. Rain intensity for different rainfall durations and return periods (Gothenburg Water Database, 2012)

Block rain		
Duration	T5	T10
10 min	186 l/s, ha	231 l/s, ha
20 min	129 l/s, ha	158 l/s, ha
30 min	100 l/s, ha	121 l/s, ha

Moreover, in the Figure 13 the rainfalls included in the P104 are represented during the 120 minutes with higher intensity. Appendix 1 contains tables with all the rainfall data. It is important to point out that the continuous rains reach higher intensities than the block rains, although the duration is shorter.

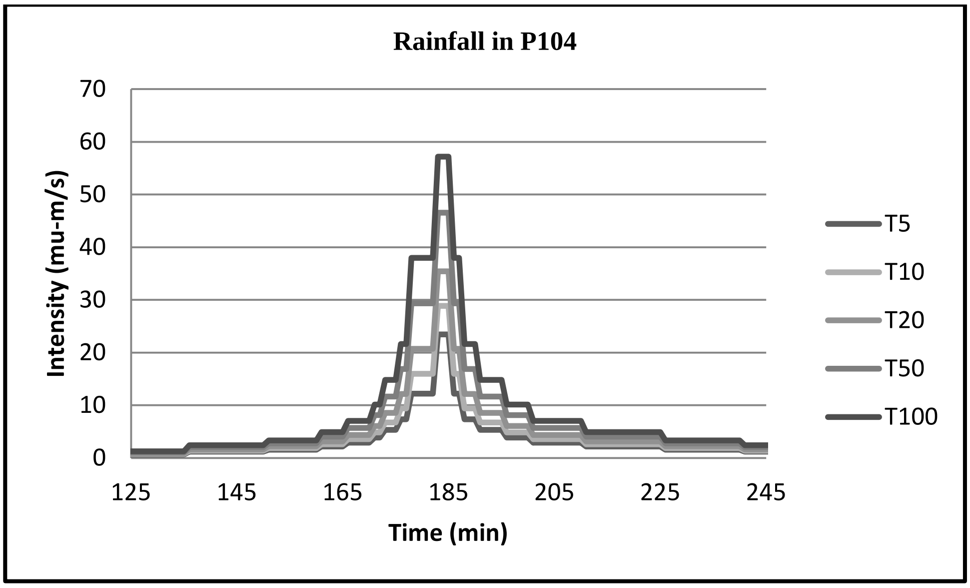

Figure 13. Rainfall intensity during the 120 minutes with higher intensity.

These rainfall data is used as input in the hydrological model to generate the runoff in each sub-catchment.

4.3 Runoff evaluation

According to Durrans & Haestad Methods (2003), the runoff is the amount of water in a rainfall that is not lost to interception, evapotranspiration or infiltration, so that it ends on water bodies or stormwater collection structures after running through the surface. Hence, the amount and characteristics of runoff not only depends on the rainfall pattern, but also on the catchment properties.

The catchments could be described as hydrological units where storm runoff and infiltration are generated in the basis of a single set of model parameters and input data. They represent the level of spatial discretization of the hydrological model (DHIa, 2011).

To simulate the generation of runoff, some parameters must be determined in the program to define the sub-catchments properties. The catchment was divided in 733 smaller sub-catchments connected to one node. This involves that 733 nodes out of the 835 existing ones are linked to sub-catchments and collect the runoff generated within the area. The required input data to define the properties of the sub-catchments is set by the choice of one hydrological model. As the Time-Area method was chosen, the parameters are reduced to imperviousness, initial loss, reduction factor, time of concentration and time-area curve.

The initial loss, reduction factor and time of concentration were assumed to be constant in all the sub-catchments and equal to 0.30 mm, 0.90 and 7minutes, respectively. In addition, all the sub-catchments were assumed to be rectangular and consequently the default time-area curve 1 was used.

The imperviousness was calculated for each sub-catchment according to the percentage of different surfaces, see Equation 9.

$$\varphi = (A_1 \cdot \varphi_1 + A_2 \cdot \varphi_2 + \cdots + A_n \cdot \varphi_n)/(A_1 + A_2 + \cdots + A_n) \tag{9}$$

where φ =*imperviousness of the whole sub-catchment,* φ_i =*imperviousness of each type of surface,* A_i =*area of each surface*

Table 4 includes the coefficients of imperviousness recommended in the P90 for each type of surface:

Table 4. Impervious coefficient for different types of surface (Svenskt Vatten, 2004)

Type of surface	Imperviousness coefficient
Roofs	90%
Concrete and asphalt surfaces	80%
Paved surfaces with gravel joints	70%
Gravel road, sharply slope mountainous park area without significant vegetation	40%
Outcrop with not significant slope	30%
Gravel paths with undeveloped parts of soil	20%
Parks with lush vegetation and rugged mountainous woodland	10%
Cultivated land , lawn or grassland	0-10%
Flat woodland	0-10%

The model covers a total area of 3.38 km^2, of which 0.93 km^2 consist of impervious surfaces as roofs or roads, resulting in an average value of 27.5% of non-permeable surfaces.

The runoff obtained in the simulation is then used as input data at each node connected to a catchment.

4.4 Description of the network model

As previously mentioned, the modelled area is divided into 733 sub-catchments. The network consists of 836 nodes, 839 links, 2 pumps and 1 weir, see Figure 14.

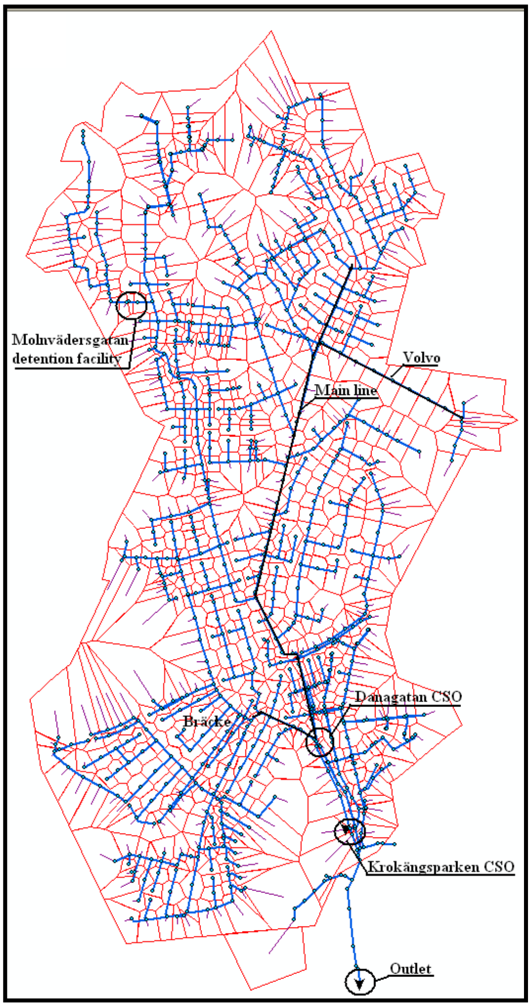

Figure 14. Sketch of the model.

The nodes are all modelled as circular manholes of one meter of diameter. The bottom and ground levels were imported from the Solen X database property of Gothenburg Water. It has been assumed that there are no energy losses in the manholes, except in 29 junctions where a loss coefficient of 0.25 was considered (nodes where different pipes are merged into the same pipe outlet). The head losses are calculated according to the MOUSE Classic (Engelund) model.

The links were considered to be all of them normal concrete with a Manning coefficient of 65 with circular geometry, except the two pipes connecting Krokänsparken overflow and the BB1121 borehole with a Manning coefficient of 75 and one stormwater pipe made of iron.

The combine sewer overflows have not been modelled with the same method. In the case of Krokänsparken the overflow was modelled by means of a weir, while in Danagatan a pump was used instead.

The weir is modelled as the CSO located at Krokänsparken. The specified discharge coefficient value is 0.3730. Moreover, the weir formula option is used at computations since standard rectangular overflow is the weir type. The overflow weir orientation is orthogonal (90º) including flow kinetic energy in the calculations of the weir flows. Crest features are an elevation of 12.79 m and a width of 1.2 m. The operation mode is static (No control) since weir control has not real time control (RTC).

Two pumps are used in model: one in the siphon at Molnvädersgatan and the other one in the Danagatan CSO. Both are statics (no control) and work at constant speed.

The capacity curve is a relation of water level (H) and the flow discharge (Q) by the pump.

The siphon pump is located after a large pipe of 3.1m that simulates the detention facility. The pump operates between a start level of 46.09m and a stop level of 42.07m. Sudden increments or decrements of flow at the pumps start and stops events are dampened by times of 10 s in both cases.

The CSO pump divides the flow from the combined pipes into two different pipes: one combined finishing at the CSO in Krokänsparken and a stormwater pipe discharging directly to Göta Älv. At Node_13 (location of the CSO), the water level difference separates the inflow from the combined pipes. The pumping starts at level of 16.44 m operating with stormwater and a stop level of 16.43 m. Sudden increments of flow at the pumps start events are dampened by acceleration times of 60 s, while sudden decrements of the flow at stop events are dampened by deceleration times of 10 s.

In addition, the modelled system interacts with the receiving waters in the two outlets included in this model. The first one is connected to Gryaabs wastewater treatment plant (WWTP) and is defined by an invert level of 11.8m. The receiving flow discharge leaves the system without surface elevation at the outlet. The second one spills the flow directly to Göta Älv. Its invert level is 7.54 m and the external water level is included as a boundary condition with a fixed value equal to 10.14 m.

Moreover, the model includes boundary conditions to represent various types of water loads, as infiltration or fixed water levels. The precipitation is introduced into the model by associating each sub-catchment to the rainfall time-series. The general network performance is determined by infiltration rates and the average wastewater production, i.e. the dry weather flow (DWF). The DWF accounts for the average wastewater produced by the households and conveyed by the system to the wastewater treatment plant. The quantity of water produced depends directly on the day of the week and on the specific hour in the day. Hence, the average wastewater production in each node was multiplied by a cyclic variation that has been created to account for these variations in the water production, see Figure 15.

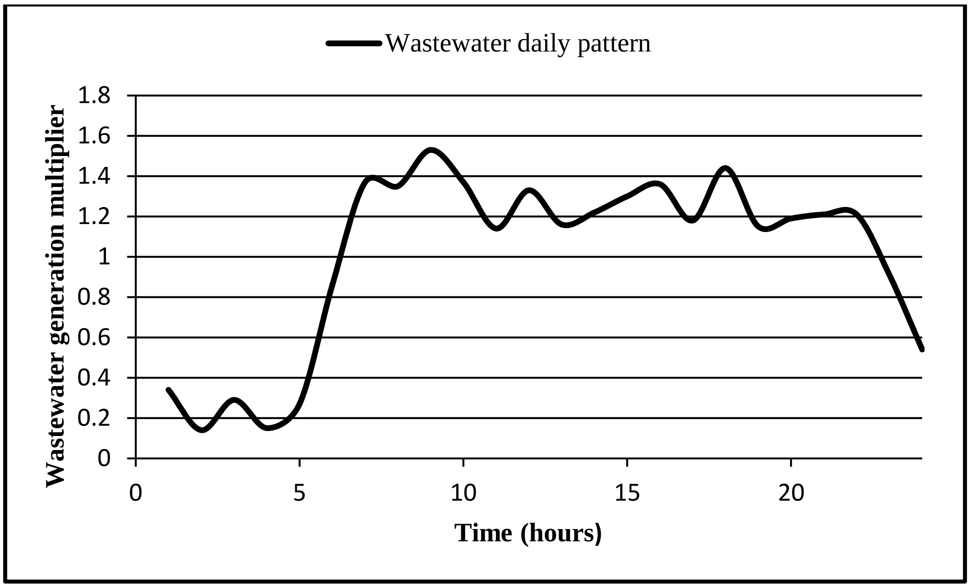

Figure 15. Wastewater production pattern.

Moreover, there are also two infiltration loads in the system. One of them affects to the majority of the nodes in the whole model with an inflow rate in the system of

0.00001 m^3/s, whereas a second infiltration pattern of 0.00012 m^3/s is associated just to a small area nearby Säterigatan.

4.5 Simulations

Different case scenarios have been considered in this study to obtain a fully understanding of the system performance under multiple working conditions. Firstly, the model has been run with the continuous and block rainfall events with different return periods to analyse the current performance, as shown in Table 5.

Table 5. Simulations with the original model.

	Type of simulation	Rain data	Description
S0(a)	Runoff and network	T5	Runoff generation within the area and network performance
S0(b)	Runoff and network	T10	Runoff generation within the area and network performance
S0(c)	Runoff and network	T20	Runoff generation within the area and network performance
S0(d)	Runoff and network	T50	Runoff generation within the area and network performance
S0(e)	Runoff and network	T100	Runoff generation within the area and network performance
S0(f)	Runoff and network	Block5	Runoff generation within the area and network performance
S0(g)	Runoff and network	Block10	Runoff generation within the area and network performance

Secondly, different local improvements in the system nearby the properties which have been flooded in previous years were modelled and analysed. Table 6 shows a summary of the simulations carried out, although further information about the modifications is included in Chapter 6.

Table 6. Simulations with the modified model including local improvements. Simulations run under T5, T10, Block5 and Block10 rains.

	Street	Type of simulation	Description
S1-Alt1	**A C Lindblads**	Network	Local reservoir (1400m^3, 1.40m height) and increased diameters
S1-Alt2		Network	Local reservoir (1400m^3, 1.80m height)and increased diameters
S1-Alt3		Runoff and network	Grass ditch
S2-Alt1	**Finlandsgatan**	Network	Extra link parallel to the existing one to act as a temporally reservoir
S2-Alt2		Network	Diversion of the line to the wastewater pipe connected to the outlet
S2-Alt3		Network	Diversion of the line to the wastewater pipe connected to the outlet and increment of some diameters in that pipeline
S3-Alt1	**Slånbärsvägen**	Network	Diversion of the line to Utmarksgatan
S3-Alt2		Runoff and network	Diversion of the line to Utmarksgatan and grass ditch
S3-Alt3		Network	Increment of pipe diameters in Slånbärsvägen.
S4-Alt1	**Visthusgatan**	Network	Increment of pipe´s diameter
S5-Alt1	**Vårvindsgatan**	Network	Diversion of the pipeline from the main line towards Vårvindsgatan
S5-Alt2		Runoff and network	Grass ditch upstream Vårvindsgatan 45
S5-Alt3		Runoff and network	Diversion of the pipe line from main line to Vårvindsgatan and grass ditch upstream årvindsgatan 45
S5-Alt4		Runoff and network	Diversion of the pipe line from main line to Vårvindsgatan, grass ditch upstream and increment of pipe´s diameter
S6-Alt1	**Pilegådsgatan Prebedengatan**	Runoff and network	Grass ditch along the cemetery
S6-Alt2		Runoff and network	Grass ditch along the cemetery and stormwater pipe connecting Biskopsgatan to downstream CSO Danagatan
S6-Alt3		Runoff and network	Grass ditch along the cemetery, stormwater pipe connecting Biskopsgatan to downstream CSO Danagatan and increment of pipe´s diameter
S6-Alt4		Runoff and network	Grass ditch along the cemetery, stormwater pipe connecting Biskopsgatan to downstream CSO Danagatan and diameters increment of pipe´s diameter in both streets
S6-Alt5		Runoff and network	Cemetery drainage connected to a node in Pilegårdsgatan, stormwater pipe connecting Biskopsgatan to downstream CSO Danagatan and increment of pipe´s diameter

Finally, general modifications were also evaluated to improve the whole system capacity. The simulations including these kinds of improvements are presented in Table 7.

Table 7. Simulations with the modified model including general improvements. Simulations run under T5, T10, Block5 and Block 10 rains.

	Location	Type of simulation	Description
S7-Alt1	**Bräcke**	Runoff and network	Grass ditch (Tyskagatan and Utmarksgatan) and detention pond (Oslogatan)
S8-Alt2	**Eketrägatan**	Runoff and network	Grass ditch from Eketrägatan until a detention pond at Östra Bräckevägen
S9-Alt3	**Volvo**	Runoff and network	Gravel detention tanks underneath parking lots
S10Alt1	**Eketrägatan**	Network	Reservoir under the football field (capacity: 9000m3, outflow: 500)
S10Alt2		Network	Reservoir under the football field (capacity: 6600m3, outflow: 800)
S10Alt3		Network	Reservoir under the football field (capacity: 9000m3, outflow: 0.500) and pipe diameters upstream increased to 1800.
S11	**Volvo**	Network	Reservoir under parking lot in the connection with Volvo
S12	**Danagatan**	Network	Reservoir under football field
S13	**Krokängsparken**	Network	Diversion of the pipeline from Bräcke to CSO Krokängsparken instead of to CSO Danagatan.

5 Results

By running the hydrological model with the rainfall data presented in Chapter 4, the runoff generation within the area was obtained. This runoff generation was then used as an input for the network simulation. The results obtained in both simulations are presented in the following subchapters. It is important to point out that a greater effort was put in the 5- and 10-year return period rainfall analysis since they are the design rains included in the P90.

5.1 Runoff simulation

The total amount of runoff generated during the different rainfall events is included in Table 8 and Table 9. As expected, the continuous rainfalls with variable intensity produce more runoff than the block rains with constant intensity due to the duration of the rainfall data.

Table 8. Total runoff generation for continuous rainfalls with different return periods. Data in m^3.

Rain event	**T5**	**T10**	**T20**	**T50**	**T100**
Total runoff	46805	55887	66768	84424	100783

Table 9. Total runoff generation for block rainfalls with different return periods. Data in m^3.

Rain event	**Block T5**			**Block T10**		
Duration	**10 min**	**20 min**	**30 min**	**10 min**	**20 min**	**30 min**
Runoff	10881	14496	16627	13580	17816	20176

Moreover, the total runoff is distributed within the area according to Figure 16. The pictures show the accumulated discharge of each sub-catchment for a T5 and T10 rainfalls. The block rains produces a similar distribution of runoff and the difference just lies in the values. By comparing both maps it is noticeable that the total discharge is considerably greater in the T10 scenario. The sub-catchments with higher runoff values are located in the north-eastern and central part of the area, along with a group of 10 sub-catchments in the western part, see Figure 16. In addition, the picture illustrates the influence of both the area and the imperviousness of the sub-catchments in the hydrological model. Thus, the majority of the sub-catchments with higher runoff discharge correspond to those with big drainage area, although there are some of them with low runoff rate due to the imperviousness. Some values of imperviousness are included inside the sub-catchment to illustrate it.

In addition, it is easy to recognize green areas that are represented in the model with small-medium size sub-catchments since the runoff generation is very low. This has its explanation in the low imperviousness in combination with the small-medium area.

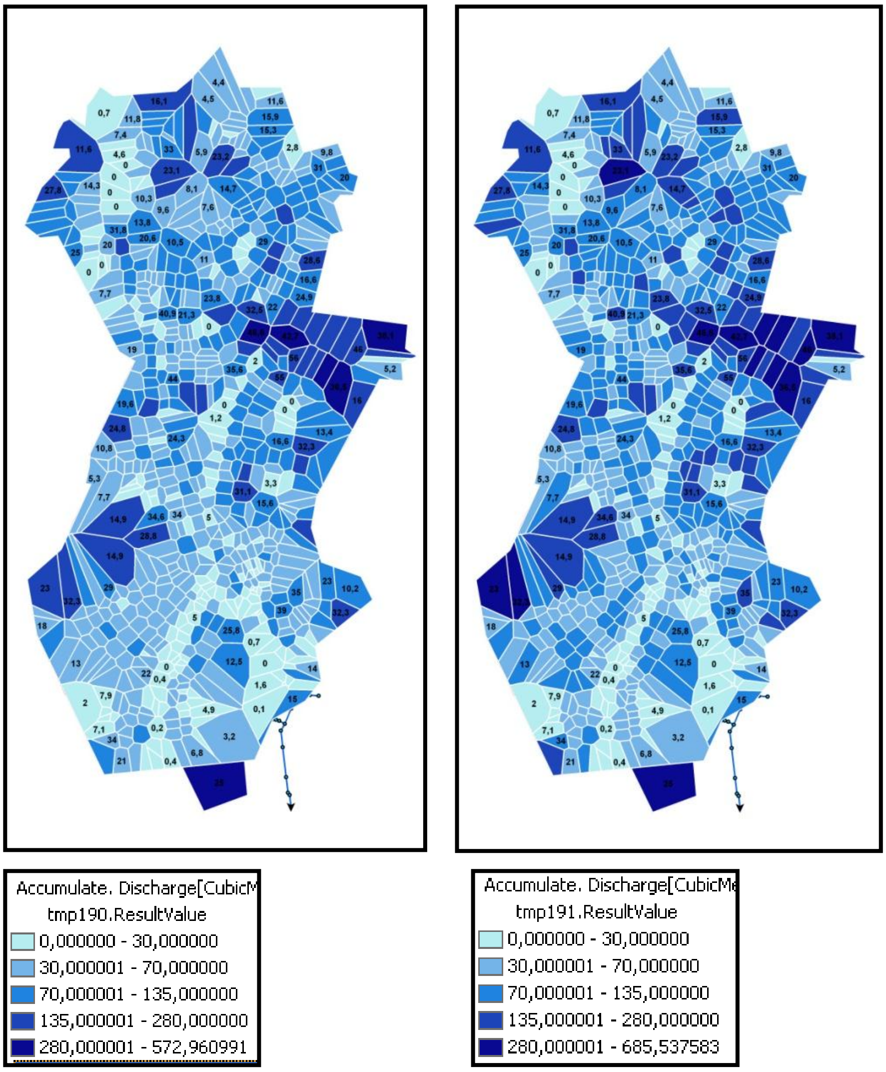

Figure 16. Accumulated discharge in all the sub-catchments and imperviousness value of some sub-catchments (a)T5 (b)T10

In order to evaluate the pattern of the runoff discharge, the water generated within one randomly chosen sub-catchment during a 5-year rain and a 10-year rain was plotted. Figure 17 shows the discharge of the sub-catchment number 144, which has 0.645 ha of area and 21.11 of imperviousness coefficient.

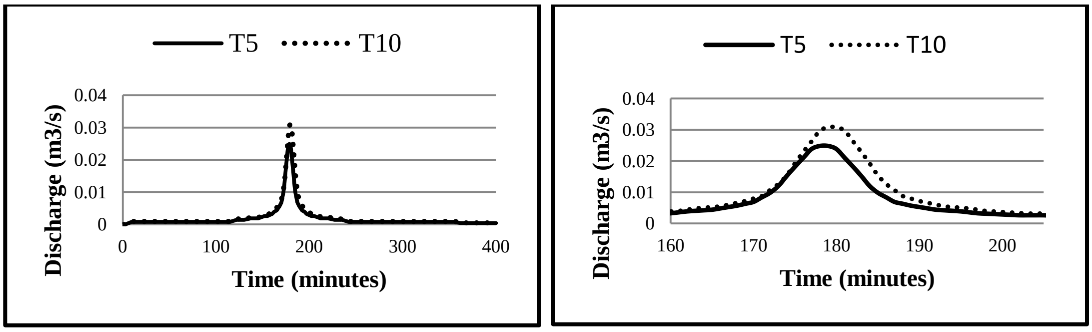

Figure 17. (a) Discharge of the sub-catchment 144. (b) Zoom in the peak discharge.

Both runoff discharges follow the same distribution with the singularity that the T5 discharge is slightly lower and shorter in time. Indeed, the peak discharge is 19.35% greater during the T10 rain. This means that the main difference between the two rain events is just the peak discharge value during a limited period of time (approximately 15 minutes). This trend is also found with all the other continuous rainfall.

5.2 Network simulation

The P90 claims that pipes in a sewage system must not work under pressure for a 5-year return period rainfall event. Moreover, sewage systems should be designed to cope with a 10-years return period rainfall in terms of water level below the surface in separate systems and water level below the basement levels in the combined ones. This implies that the flooding risk must be verified in the nodes (manholes) in separate systems, whereas water level at each user connection must be checked independently with a longitudinal profile in parts of the system which are combined. This procedure was done in the properties with previous basement flooding problems. Nevertheless, in order to facilitate the analysis of the flooding risk in the whole area it was assumed that the maximum pipe flooding (water level minus ground level) is representative for all the private pipes connected to the link under consideration. The basement levels were assumed to be 1.5 meters below the surface, so that the pipes and the nodes are classified in three regions: water level above surface (link flooding greater than cero), risky area (link flooding varies between minus one and a half to cero meters) and safe area (link flooding smaller than minus one and a half meters).

The results obtained in the simulations are gathered in Table 10. It contains the classification of nodes and pipes regarding node flooding, pipe flooding and pipe filling.

Table 10. Classification of nodes and pipes regarding to node flooding, pipe flooding and pipe filling for different rain events.

		T5	**T10**	**T20**	**T50**	**T100**	**BlockT5**	**BlockT10**
Nodes flooding	< -1.5 m	60%	44%	34%	19%	3%	67%	51%
	-1.5 – 0 m	26%	33%	31%	28%	0%	22%	29%
	>0 m	14%	23%	35%	53%	97%	11%	19%
Pipe flooding	< -1.5 m	51%	36%	28%	14%	10%	59%	43%
	-1.5- 0 m	28%	33%	28%	22%	18%	25%	30%
	>0 m	21%	31%	44%	64%	72%	16%	27%
Pipe filling	<90%	30%	21%	12%	5%	4%	36%	24%
	90%-100%	3%	2%	2%	0%	0%	2%	2%
	>100%	67%	77%	86%	95%	96%	62%	74%

As can be seen from the table, the limitations presented in P90 are not met either in the continuous rain or in the block rain. According to the simulations, 560 pipes have worked under pressure during the T5 rain, whereas 524 are pressurized during the block rain. These numbers represent 66.7% and 62.5% of all pipes, respectively. In addition, 189 and 161 nodes are flooded in the continuous and in the block rain respectively, which involves 22.6% of the nodes in the case of T10 and 19.3% of the nodes in the T10 block. Therefore, the rains with continuous hydrographs are the worst case scenario since the amount of nodes and pipes with water level below 1.5 meters, i.e. water level below basement level, is smaller than in the block rains.

Moreover, link flooding follows also this trend. Figure 18 shows the comparison of the percentage of pipes with water level in the three regions of importance: above the surface, from surface to 1.5 meters below and lower than 1.5 meters from the surface.

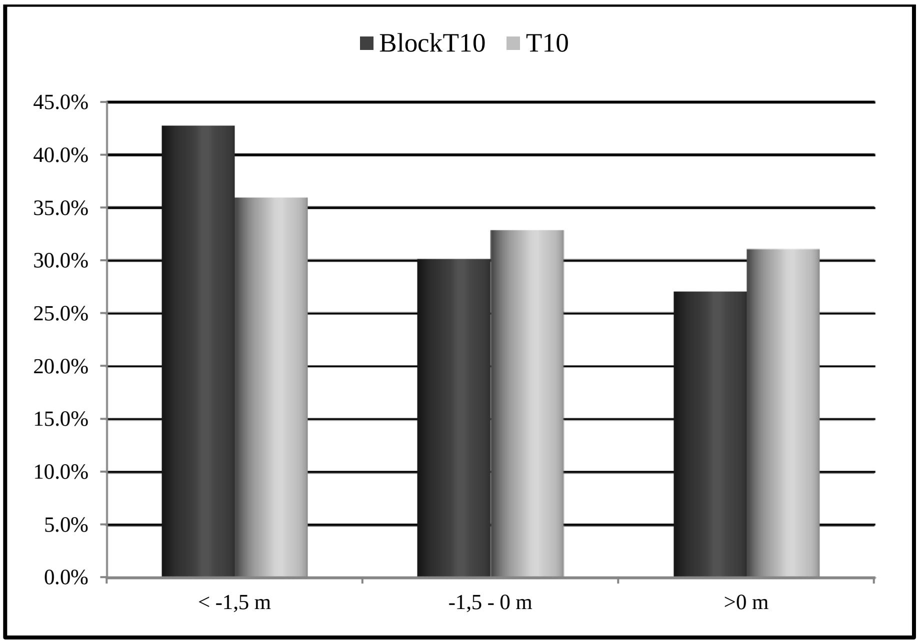

Figure 18. Percentage of links with water level in each one of the three regions.

In the case of the block rain, the percentage of links with water level below 1.5 meters from the surface is slightly higher than those with the water approaching the surface. However, in the case of the T10 rain the distribution is barely the same, but the percentage of pipes classified as risky or flooded is greater than in the block rain. Indeed, 64% of the pipes have the water level line either above the surface or 1.5 meters below it (i.e., the water level line is higher than the private connection to households).

The distribution within the area of the overloaded parts of the system is shown in Figure 19. As can be seen, the pipes working under pressure during the T5 rain are distributed in the whole area without a pattern. In the same way the pipes with water level higher than 1.5 meters below the surface during a T10 rain are also spread among the network. However, in this case there is a higher concentration of flooded links in the north-eastern part of the network.

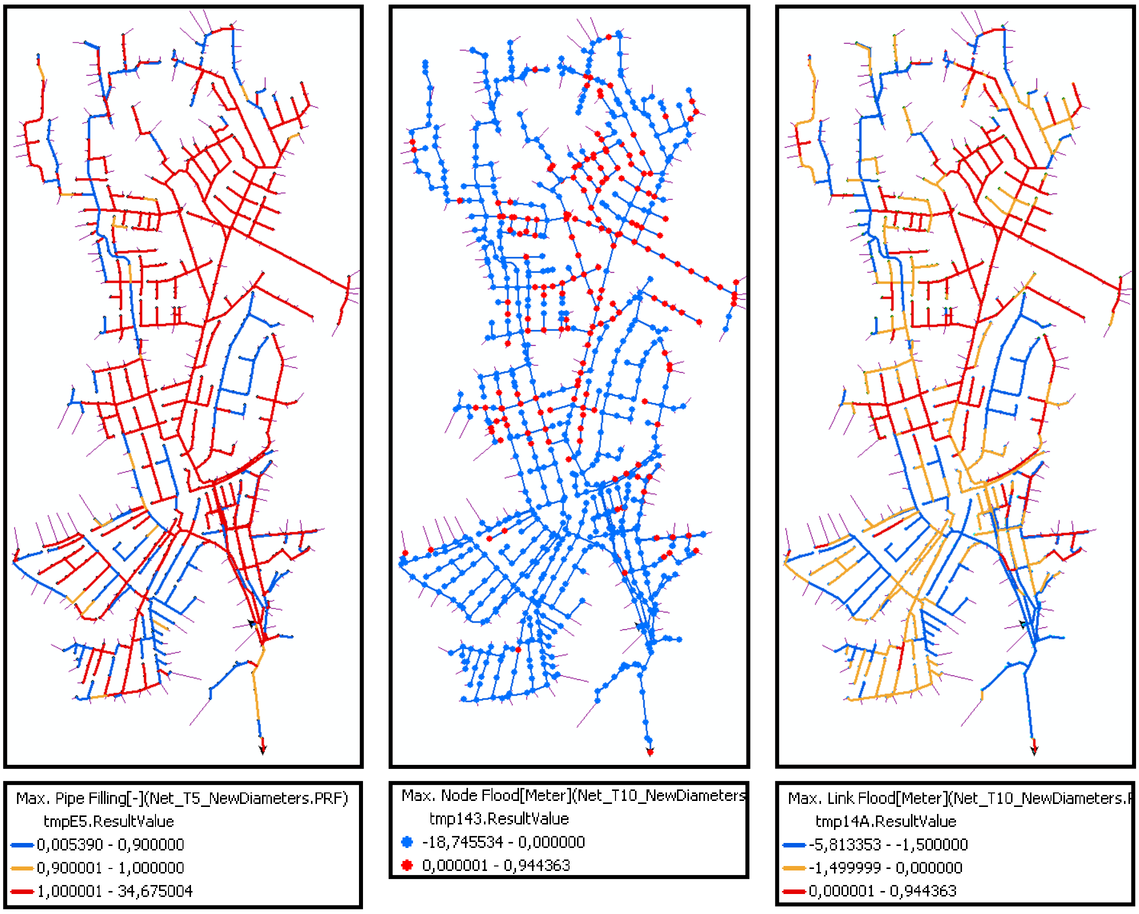

Figure 19. (a) Maximum pipe filling in T5. (b) Link flooding in T10. (c) Node flooding in T10.

Similarly to the pipe flooding and as expected, the majority of the flooded nodes are also concentrated in the north-eastern zone.

5.2.1 Properties with previous flooding problems

The simulations point out that the modifications implemented in the system after the flooding event in 2006 have reduced the risk of flooding in the majority of the properties. However, the water level is still higher than the basement level in 5 out of 39 houses. Table 11 below shows the comparison between the basement level and the water level reached in the connection point. The table just includes the houses connected to a combined system that are still under flooding risk. Appendix 4 contains a table with the results obtained for all the properties.

Table 11. Comparison of basement levels and water levels in the properties with flooding risk according to the simulations.

	BASEMENT LEVEL	WATER LEVEL T10	DIFFERENCE	WATER LEVEL T10 BLOCK	DIFFERENCE
AC Linblads Gata, 5	35.36m	36,85m	-1.49m	36.73m	-1.37m
Finlandsgatan 2	20.11m	21,31m	-1.2m	20.18m	-0.07m
Slånbärsvägen 26	27.83m	28,85m	-1.02m	28.65m	-0.82m
Vårvindsgatan 45	28.43m	30,37m	-1.94m	30.18m	-1.75m
Visthusgatan 8	27.49m	27,12m	-0.37m	27.73m	-0.24m

Moreover, the results in the separate parts of the system show also 7 nodes with water level above the surface. As Table 12 illustrates, the water level reaches almost 50 cm above the surface in 3 nodes in Pilegårdsgatan.

Table 12. Comparison of water level and ground node level in the affected streets with separate system. Data in meters.

	NODE	NODE LEVEL	T10	Difference	BlockT10	Difference
Pilegårdsgatan	1_1754	22,83	23,3	-0,47	23.30	-0.47
	1_1742	21,92	22,38	-0,46	22.39	-0.47
	1_1739	21,64	22,1	-0,46	22.10	-0.46
	1_1733	21,13	21,4	-0,27	21.18	-0.05
	1_1720	20,65	20,86	-0,21	19.60	1.06
Prebendegatan	1_1727	21,72	22,02	-0,3	21.93	-0.21
	1_1712	20,54	20,9	-0,36	20.85	-0.31

6 Management Plan

The general goal of the management plan is to reduce the amount of water entering into the system, as well as to improve the current network performance. Environmental-friendly practices are the major solutions to offset the impacts caused by urbanization by returning to nature developed spaces (Durrans & Haestad Methods, 2003). Therefore, the study was firstly focused on the implementation of sustainable drainage systems to avoid the overloading of the network. Nevertheless, other approaches based on conventional infrastructural modifications were also considered due to the critical situation of this area.

6.1 Management plan considerations

Several ideas should be kept in mind in order to design suitable solutions to avoid flooding, as well as to adjust them to the specific area. Some of these factors are presented below (Durrans & Haestad Methods, 2003):

- Ground conditions: Clay and bedrock are the main materials under the soil. The high imperviousness of these materials hinders the infiltration of water into depth ground.
- Spaces: the location of the solution should consider the topographic conditions (i.e. slopes and lands height, among others), as well as free surface space circumstances due to the high concentration of urbanized areas.
- Aesthetic: it is necessary to avoid negative aspects as eutrophication, the possibility of odours, mosquitoes and so forth since the actions will be located in an urban area.

6.2 Description of potential solutions

In the following sub-chapters a brief description of the considered solutions is presented.

6.2.1 Infrastructural actions

6.2.1.1 Reservoirs

Reservoirs are structures whose aim is to relieve the sewer network and to avoid the damage caused by fast-rising waters during heavy rainfall events. The capacity of storage depends on the dimensions of the container and the outlet characteristics, which establishes the flow discharge downstream in the system (Durrans & Haestad Methods, 2003).

The reservoir location is a main factor that determines the effectiveness of this kind of structures. As reservoirs are designed to detain water, the sewage system operation is modified downstream their location, while the influence upstream is neglected (Durrans & Haestad Methods, 2003).

Of importance in the design of stormwater storage facilities is to establish the level that the stormwater can reach. Figure 20 represents a basic hydrograph used to determine the required storage volume in order to design an optimal reservoir. The area under the hydrographs is the volume of runoff in a storm event. The required storage volume is the minimum capacity that the reservoir should have to be able to handle the rain represented in the hydrograph.

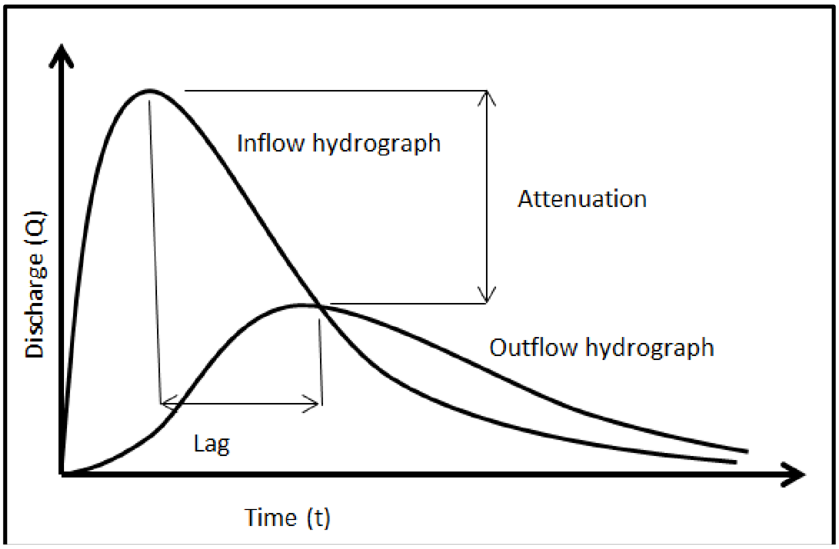

Figure 20. Hydrograph concepts associated with a temporary storage (Modified from Durrans & Haestad Methods, 2003).

The sharply growing line represents the time-discharge curve generated by a flood wave: inflow hydrograph. The other one represents the same curve after the detention storage facility: outflow hydrograph. Two main effects can be observed: attenuation (reduction) of the peak discharge and lag in peak time. The peak inflow for the detention storage is larger and it occurs earlier than the peak at the outflow hydrograph. Moreover, a redistribution of water in the system is produced due to temporal storage in the detention facility.

6.2.1.2 Enlargement of pipe diameters and new pipeline connections

The enlargement of diameters is considered as a possible solution when simulations prove enough capacity at downstream pipes, whereas upstream pipes cannot handle a certain rain. In such case, the line is undersized and an enlargement of the pipes diameter is the best option. Colebrook diagram allows, by means of the flow discharge, the preliminary estimation of the design diameter. This first approximation of the required pipe capacity is then verified with the simulations in MIKE URBAN.

In previous system modifications, concrete pipes were replaced by polypropylene ultra rib pipes. Therefore, the new dimensions were determined according to the available diameters for this kind of pipes in the commercial firm Uponor (Uponor, 2009).

6.2.2 Sustainable Urban Drainage Systems (SUDS's)

SUDS's are strategies used to improve sustainable urban development and performance. The aim of the Sustainable Urban Drainage Systems is the management of the stormwater from an environmental-friendly point of view. The major design aim of these systems is to maximize runoff volume reduction and nutrient removal. Moreover, the reduction of water entering in the combined system avoids overflows and improves water quality by infiltration and sedimentation of pollutants (Pettersson, 2011).

6.2.2.1 On-site management: Green roofs and permeable artificial pavements

Green roofs and permeable artificial pavements are on-site approaches for stormwater management. Their aim is to mitigate the peak flow from each sub-catchment individually, i.e. the peak flow reduction for the whole watershed is neglected.

Green roofs capture precipitation, evapotranspirate a small amount of water taken up by plants and temporally store the runoff before water flows into storm drain systems (EPA, 2003). Overall peak flow discharge is reduced, as well as the pollutants concentration in runoff volumes.

Vegetated roofs consist on a layered system of roofing, as can be seen in Figure 21. The layered design is required to maintain the vigorous cover which captures and stores water, as well as to avoid the water to enter in the building. Of importance is the building structural capacity since the weight of the additional water retained in the roof has to be supported by the structure.

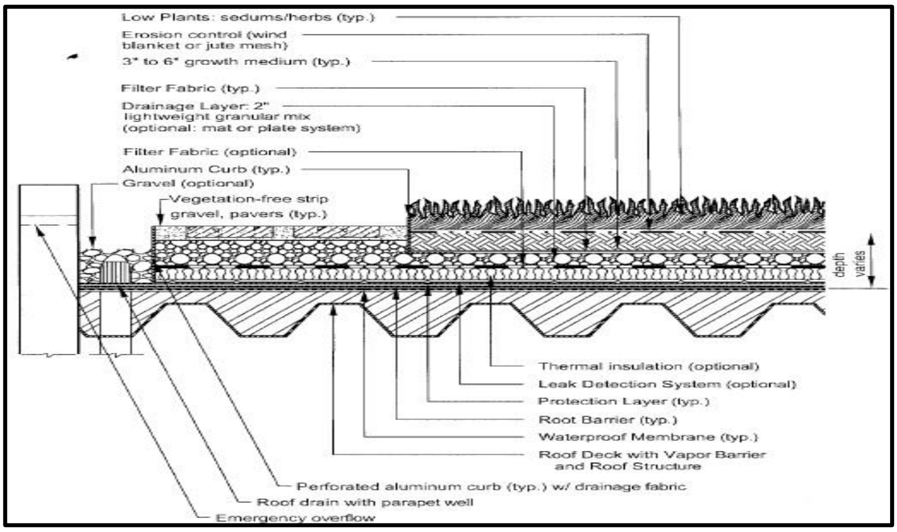

Figure 21. Cross section of green roof layers deployment (Chesapeake Stormwater Network, 2011).

Local climate and design objectives govern plant selection in order to match the plant species to the appropriate plant hardiness zone. In this case study, it was assumed that the plant to be used in the green roofs is sedum. According to a study performed in Lund (Sweden), the runoff generation can be reduced up to a 65% during rains with variable intensity if sedum green roofs are implemented (Villarreal, 2007). However, this effectiveness depends on multiple factors, such us rain characteristics. Therefore, an imperviousness value of 0.5 was assumed for the current investigation.

Permeable artificial pavement allows the infiltration, evaporation, storage and treatment of runoff. These pavement systems may be located at parking lots, sidewalks and ways for both cars and pedestrians. The permeable surface allows water movement throughout the material vertically and retains water under this surface temporally, reducing highly the peak flow discharge caused by large impervious surfaces (Scholz & Grabowiecki, 2007).

There are different types of permeable pavements, which have a layered design as the common characteristic. Surface pavement layer is formed by a porous material enabling water pass through it, see Figure 22.

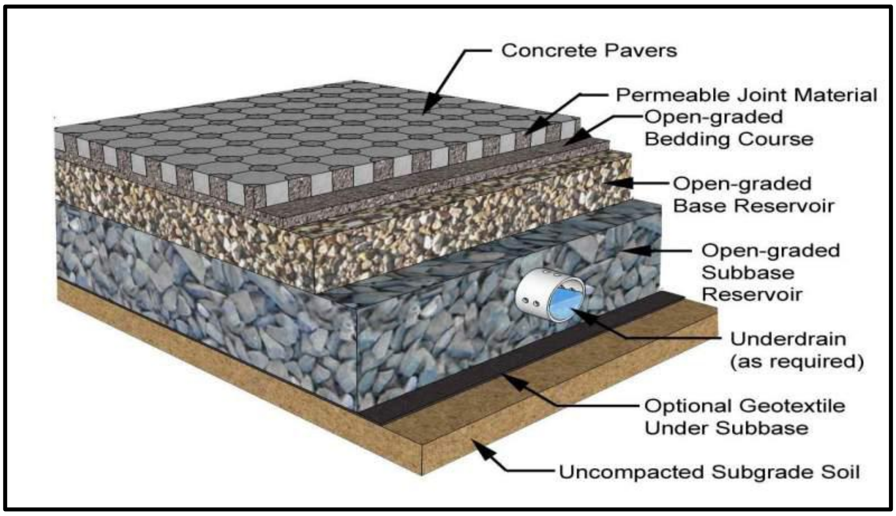

Figure 22. Cross section of permeable pavements (Chesapeake Stormwater Network, 2011).

The underlying stone aggregate is the reservoir which retains stormwater and also supports the design traffic loads for the pavement. Depending on the permeability features of the subgrade soil, a filter layer or fabric is installed on the bottom allowing water infiltration into the soil (Scholz & Grabowiecki, 2007).

6.2.2.2 Retention and infiltration management: Grass ditch

Grass ditches and filter strips are vegetated features designed to retain the flow and remove pollutants dragged by surface runoff. The process is developed in the following steps: filtration, infiltration, absorption and biological uptake (Chesapeake Stormwater Network, 2011). The precipitation is retained and conveyed by the filter strips to the grass ditch, where flow velocity is reduced, so that pollutant sedimentation is allowed, see Figure 23.

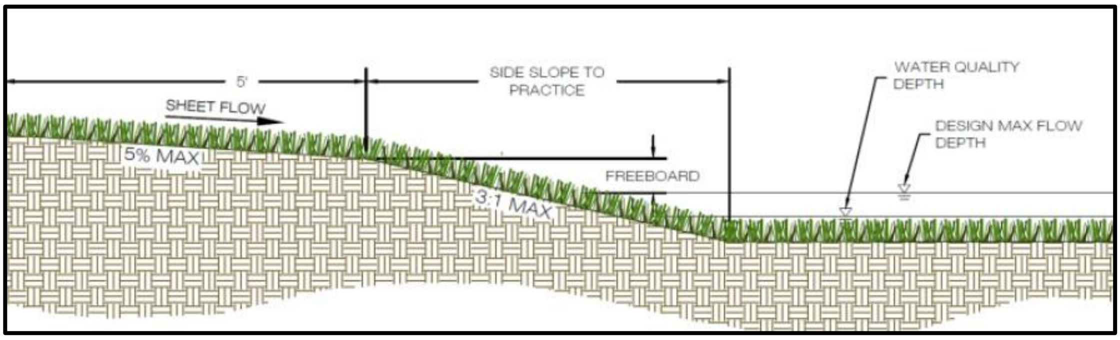

Figure 23. Grass filter for sheet flow and grass ditch (Chesapeake Stormwater Network, 2011).

Grass ditches are shallow channels located close to roads, parking lots or foothills. Its performance provides filtration and attenuation of a modest amount of runoff, i.e. detaining and reducing the inflow and the pollutant concentration at the network inputs (Chesapeake Stormwater Network, 2011). Filter strips are sloped surfaces leading the runoff into the ditch by a surface covered by vegetation ensuring that water spends enough time to reduce the discharge and allow the infiltration (Wilson et al., 2009). Figure 23 shows a grass ditch layout with the dimensions required in the design.

Deployment design of this practice depends on the urbanization density, topography and soils permeability. Moreover, the main design requirements are mainly two: the capacity should be enough to handle safely storms of 10 years return period (EPA, 1996) and the longitudinal slope is limited to less than 4% (Chesapeake Stormwater Network, 2011), since steeper slopes cause rapid runoff velocities (erosion) and do not allow enough contact time.

6.2.2.3 Detention management: Basin and gravel ponds

Basins and ponds are open areas with or without permanent water, where water may be stored in case of heavy rainfall in order to manage the excess of runoff volume. The temporary storage allows pollutants and sediments to settle on the bottom of the pond providing environmental benefits (Wilson et al., 2009). The operation is based on the collection of water from the surface runoff or from a stormwater pipe, which discharge runoff into the pond. The outflow is then evacuated in the same way: by a pipe or by a controlled surface discharge. Depending on the permeability soil features some water will infiltrate into the ground decreasing the flow discharge (CIRIA, 2012).

Gravel ponds or filter drains are gravel filled trenches where water percolate throw the stones, while pollutants are retained in the surface of the gravel pieces. Furthermore, part of the water is temporally stored in the gaps until it drains, normally, to a perforated pipe (Wilson et al., 2009). The drainage pipes lies on the bottom of the trench and collects the water infiltrating throughout the gravel. A geotextile or a fabric filter is usually set below the surface to avoid clogged by sediment accumulation into the gravel gaps, see Figure 24.

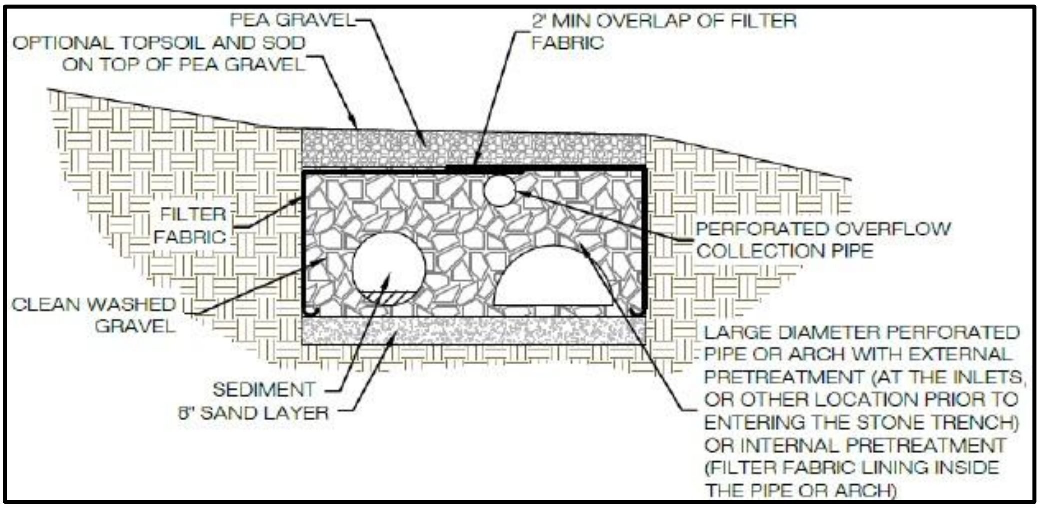

Figure 24. Infiltration section with supplemental pipe storage (Chesapeake Stormwater Network, 2011).

Gravel detention tanks are used to reduce stormwater runoff by infiltration, as well as to store and delay the entrance of runoff in the network. According to Dominico and Schwartz (1997), the gravel porosity varies between 24% and 38%. Hence, the volume of the gravel in this kind of tanks represents the 70% of the total and the required dimensions of the tank can be calculated by using the Equation (10).

$$V_t = \frac{(L \times A_{eff})}{0.30} \tag{10}$$

where $L =$ *pipe length [m]*, A_{eff} *= effective area of the ditch [m^2] (area that has been filled with water in the simulations),* V_t *=total volume required for gravel tank [m^3]*

Moreover, another type of underground storage facilities similar to the gravel detention tanks is the stormwater boxes, see Figure 25. These devices reduce the runoff generation through infiltration, storage water above the surface and balance the discharge peak in the network (Wavin, 2007).

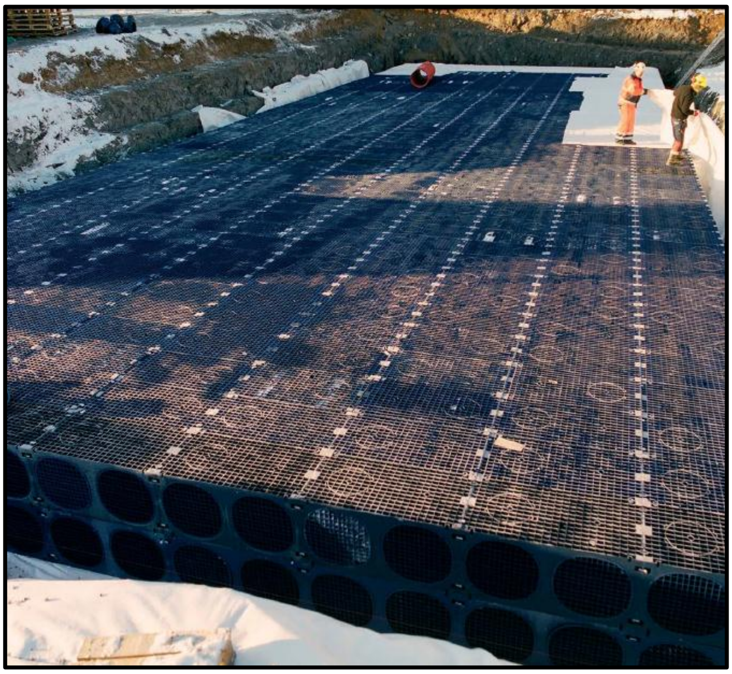

Figure 25. ”Wavin dagvatten kassett” (stormwater box) installation.(Wavin, 2007)

In these devices the use of gravel is avoided, so that the total volume required is considerably reduced since the he storage capacity is 95% (Wavin, 2007).

6.3 Evaluation of potential solutions

Based on the results of the simulations in the base system, different modifications were included in the model to improve its capacity and efficiency. As the worst case scenario corresponds to a continuous rain, the actions required in the system were designed to cope with this rain, although the capacity was also tested for the block rains. Moreover, it is important to point out that the continuous rainfall used in this study corresponds to a one-day rainfall, whose peak discharge occurs about 3 hours after the beginning of the series. For that reason, the simulations were run just during 6 hours, since after that the system recovers its normal performance.

6.3.1 Local measures

Local measures refer to specific system modifications whose aim is to reduce the water level in the pipes where properties with previous basement flooding are connected to, as well as in the surrounding ones. In the following sub-chapters a summary of the obtained results for the potential solutions are presented. The tables show the results regarding pipe filling for 5-year return period rains (water depth in pipes divided by pipe height) and the difference between water level and basement level for 10-year return period rains. The basement level in properties with previous basement flooding events was measured by Gothenburg Water, so that they are not just an approximation. Thus, pipe filling values over one means that the flow is running under pressure and negative values of the difference between water level and basement level indicates basements under flooding risk. Situation maps, longitudinal profiles and further information are included in the appendices.

6.3.1.1 AC Lindblads Gata 5

The simulations show that the volume of water in this part of the system is greater than the capacity of the system per unit of time, resulting in the surcharge of the network. In addition, the main line is overloaded, which makes the water level to rise upstream. Therefore, the modifications are addressed to delay and/or reduce the volume of water entering in the main line during the peak rain. In order to achieve this goal, a reservoir in the secondary line before the connection to the main line and a grass ditch in the upper part of the secondary line were implemented. After the surcharge was eliminated by means of a reservoir, the pipes in this street were found to be undersized and it was necessary to increase their capacity. The characteristics of the network modifications are included in Table 13, whereas a map with the location of each element is included in Appendix 5, along with longitudinal profiles of the obtained results.

Table 13. Network improvements on AC Lindblads Gata 5.

Alternative Description		Pipe filling		Difference (Basement level-water level)	
		T5	**BlockT5**	**T10**	**BlockT10**
S1 Alt1	Reservoir (dimension: 1.40x20x50m, capacity: 1400m^3, outlet: 300mm$^)$), diameter of the two pipes in the street increased to 277mm	0.69	0.65	0.21m	0.57m
S1 Alt2	Reservoir (dimension: 2.8mx10x50m, capacity: 1400m^3, outlet: 300mm), diameter of the two pipes in the street increased to 277mm	1.03	0.66	-0.2m	0.49m
S1 Alt3	Grass ditch surrounding the hill and detention pond (outlet: 200 mm)	9.50	6.15	-1.33m	-1.31m

As can be seen from the table the differences between the basement level and the water level in AC Lindblads Gata 5 are smaller during T10 than in BlockT10 and the pipe filling is larger during T5 than in Block T5. However, the variation of the results obtained with each type of rain is larger in the case of alternative 2 than in the other cases.

According to the data, any of the evaluated reservoirs has a positive impact in the network at this street, although the reservoir of alternative 1 is more efficient with 69% of pipe filling value and the water level reduced in 1.7 meter, see Appendix 5. The reservoir in alternative 2 makes the pipes to work under pressure and the water level to drop 1.29 meter, which places the water level still higher than the basement level. Hence, the dimensions have an important role in the reservoir effectiveness and, in this case, the reservoir with greater plan area works better than a reservoir with smaller cross section area and major height. Moreover, the grass ditch has shown to have a lesser impact in the hydraulic level reduction since the water level is just lowered 16 centimetres during the peak rain.

In order to compare the effectiveness of the three alternatives the water level in the same link was plotted. Figure 26 shows the water level in the link 539, which is located before the reservoir in the pipe connected to the main line (see Appendix 5). As can be seen, the reservoirs reduce the maximum level reached by water in the pipe, although it takes longer to lower the water level. In the case of the grass ditch, the water level follows the same pattern as in the current situation, with the singularity that the water level remains in the maximum values during a shorter period of time.

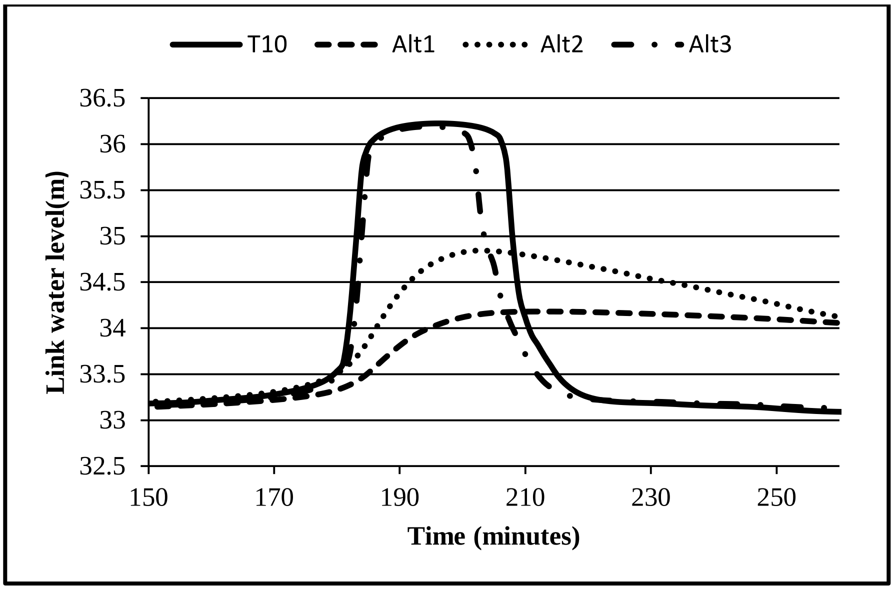

Figure 26. Comparison of link water level of the link_539 in different scenarios.

It is important to highlight that the reservoirs (alternative 1 and alternative 2) reduce the number of nodes flooding to 164, even though this is a local measure. In fact, the reservoir does not affect just this street, but all the pipes in the part of the system connected to the reservoir, see node flooding map in Appendix 5.

6.3.1.2 Finlandsgatan 2

This property is located nearby the main line and directly connected to it, so that the water level in this house is highly affected by the excess of water in the main conduit. The different alternatives that were evaluated to increase the system capacity are gathered in Table 14. Appendix 6 shows the longitudinal profile with the comparison of the water level with the current situation and with the implemented modifications, as well as the diameters of the incremented links.

Table 14. Network improvements on Finlandsgatan 2

Alternative Description		Pipe filling		Difference (Basement level-water level)	
		T5	**BlockT5**	**T10**	**BlockT10**
S2 Alt1	New pipe (Ø=1000mm) parallel to the existing one acting as a temporal reservoir (52 m^3)	6.45	6.05	-0.08m	-0.01m
S2 Alt2	Diversion of the pipes in this street to wastewater pipe directly connected to the outlet	9.72	9.57	-0.70m	-0.59m
S2 Alt3	Diversion of the pipes as in S2-Alt2 and enlargement of diameter of four pipes	3.32	2.31	0.36m	0.67m

The continuous rains represent the worst case scenario with higher pipe filling values and smaller differences between basement and water levels in all the cases.

Both alternatives 1 and 2 lower the water level in Finlandsgatan, although not enough to avoid the flooding of the basements in this street. In the case of the alternative 1, the extra pipe is located close to the main line and it is less steep than the pipes at Finlandsgatan. Thus, the extra pipe receives water from the main line when the system is surcharged and, consequently, the water level is not decreased in Finlandsgatan during the peak rain. In the case of the alternative 2, the capacity of the wastewater line is not enough to handle the extra amount of water diverted from Finlandsgatan. However, if the diameters of the pipes in the wastewater line are increased as in alternative 3, the water level is reduced below the basement level, see Appendix 6. It is important to consider that the capacity of the wastewater line is still exceeded, so that basement problems could arise in properties connected to this line, see profile in Appendix 6.

In spite of the improvements presented in alternative 3, the water level is still considerably high in Finlandsgatan 2. Hence, the diversion and enlargement of pipes should be accompanied by the installation of a non-return valve to avoid potential basement flooding. As this property is connected to a combine system, the valve should be placed in the wastewater private connection. Figure 27 show a schematic representation of the non-return valve installation.

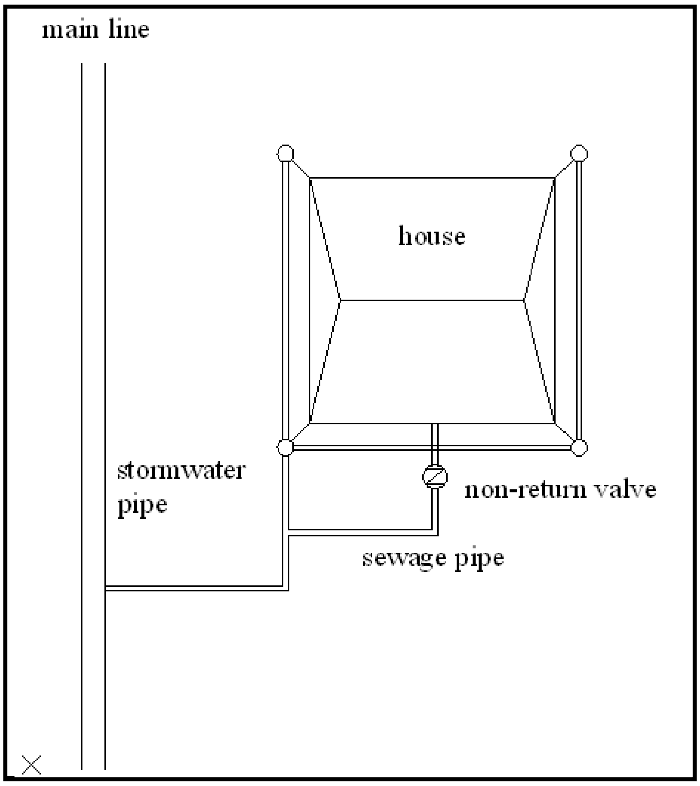

Figure 27. Scheme of a non-return valve installation in private wastewater pipes.

Moreover, it is important to take into account that the closeness of Finlandsgatan to the main line means that general modifications in the main line have a great impact in the water level in this street. Therefore, further analyses of the potential solutions for this street are included in the sub-chapter 6.2.2 (General modifications).

6.3.1.3 Slånbärsvägen 26

The network in this street deserves special attention since the pipe diameters have already been increased recently. However, the water level still exceeds the basement level. Based on this fact, it was simulated the connection of the last node of Slånbärsvägen to Utmarksgatan, which is less overloaded. In addition, a grass ditch in Utmarksgatan was also simulated to reduce the volume of water in this line during the peak rain discharge. However, the results were not completely successful, so that the enlargement of diameters has also been considered as an alternative. Table 15 shows the alternatives analysed on Slånbärsvägen. Longitudinal profiles, the dimension of the modified pipes and the characteristics of the simulated grass ditch are included in Appendix 7.

Table 15. Network improvements on Slånbärsvägen 26

Alternative Description		Pipe filling		Difference (Basement level-water level)	
		T5	**BlockT5**	**T10**	**BlockT10**
S3 Alt1	Connection of last node in Slånbärsvägen to Utmarksgatan	1.96	1.08	-0.38m	-0.12m
S3 Alt2	Connection of last node in Slånbärsvägen to Utmarksgatan and grass ditch in this street (detention function, outlet: 150mm)	1.87	1.05	0.01m	0.12m
S3 Alt3	Enlargement of pipe diameters	0.46	0.41	0.14m	0.36m

As can be seen from the results in the table, the continuous rain is again more restrictive regarding either the pipe filling or the water level.

Alternative 1 has shown to have a limited impact in Slånbärsvägen since the water level is still higher than the basement level. The diversion of water works only if it is complemented with a grass ditch to delay the entrance of water in the system (alternative 2), although the water level is close to the basement level in the 10-year return period rains and the pipes are working under pressure in the 5-year return period rains. Hence, this option should include also a non-return valve installed in the private wastewater connection to protect the basement against flooding, see Figure 27. With the enlargement of pipe diameters (alternative 3) the pipe filling and water level values are improved and the basement flooding problem is solved.

According to the results obtained with the simulations this street is affected by a combination of surcharge and backflow, as can be seen in Figure 28. This figure shows the discharge in the three alternatives as well as in the current system of the pipe where Slånbärsvägen 26 is connected to the sewage system (link 1506 in the profile included in Appendix 7).

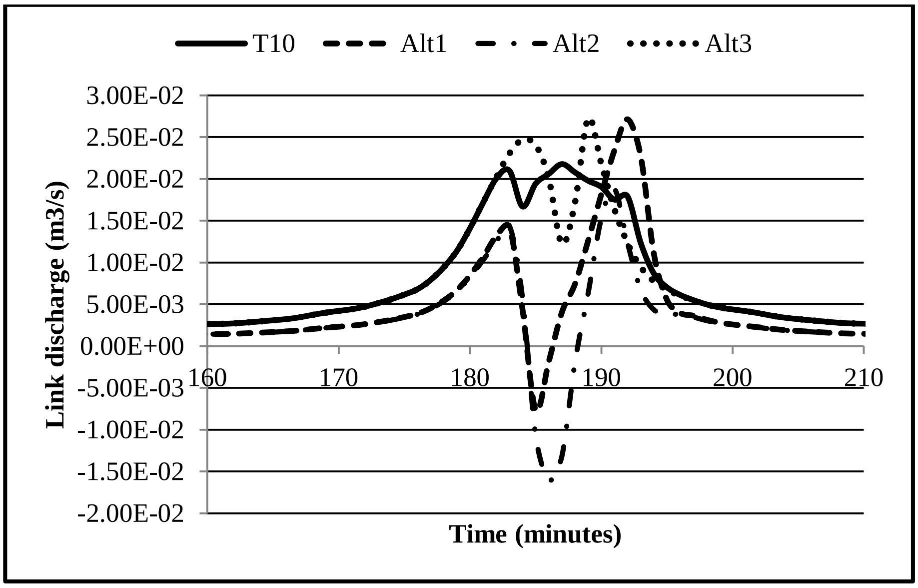

Figure 28. Link discharge of the pipe where Slånbärsvägen 26 is connected to the network.

When the system is overloaded downstream, the water level rises upstream and backflow occurs during peak discharges. Even though alternative 3 decreases the water level below the basement level, backflow effects cannot be avoided. Moreover, the connection to Utmarksgatan makes the flow to divide into two directions (Slånbärsvägen and Utmarksgatan), so that the discharge of the pipe under consideration is decreased and even the water flows in the opposite direction to the current one, see Figure 28.

6.3.1.4 Visthusgatan 8

According to the simulations, the problem in this street is the insufficient capacity of the existing pipes. Hence, the suggested modification consists on enlarging the diameters of the pipes since the network downstream has enough capacity, see Appendix 8. This appendix also includes preliminary hand-calculations of the required diameters using Colebrook's diagram and the discharge of the sub-catchments connected to those links. Table 16 shows the obtained results for the pipe filling and the water level.

Table 16. Network improvements on Visthusgatan 8

Alternative Description		Pipe filling		Difference (Basement level-water level)	
		T5	**BlockT5**	**T10**	**BlockT10**
S4 Alt1	Enlargement of pipe diameters	0.79	0.78	0.68m	0.87m

Although in this case the results of both types of rains are similar, the values are better with the block rains. The results show that the replacement of the pipes increases the network capacity, so that the property is not under basement flooding risk, see Appendix 8.

6.3.1.5 Vårvindsgatan 45

This property is especially difficult to work with since the basement level is located just 15.5 centimetres above the pipeline. Two main modifications were simulated with the intention of reducing the water level: a grass ditch with storage function and the diversion of water coming from upstream pipelines. It is important to highlight that the diversion involves the increment of the amount of water flowing along Vårvindsgatan pipe, where some pipes had their diameter increased since two properties were flooded previously. Hence, the properties Vårvindsgatan 17 and 19 were also analysed to check that the water level remains below the basement level. Table 17 includes a summary of the studied improvements for this street.

Table 17. Network improvements on Vårvindsgatan.

Alternative Description		Pipe filling		Difference (Basement level-water level)	
		T5	**BlockT5**	**T10**	**BlockT10**
S5 Alt1	Diversion of the line towards Bräcke at Våvindsgatan	3.58	2.98	-1,07m	-0.72
S5 Alt2	Grass ditch (detention function) upstream the property (outlet: 150mm)	11.67	11.38	-1,87m	-1.76
S5 Alt3	Diversion of the line and grass ditch (outlet: 150mm)	2.91	2.41	-1,05m	-0.58
S5 Alt4	Diversion of the line, grass ditch (outlet:150mm) and increment of diameters	0.67	0.65	-0,18m	0.17

Similarly to the four previous cases, the continuous rains represent also the worst case scenario in this street. However, one singularity was found in alternative 4 since this alternative works for the block rains, but fails for the continuous rain.

The diversion of water towards the main line in Bräcke (alternative 1) drops the water level in Vårvindsgatan 45, but it also overloads the receiving line in Bräcke, see situation map in Appendix 9. In addition, the reduction is insufficient to avoid basement flooding since the pipes in this street have shown to be under-sized. The same problem of the insufficient pipe capacity limits the effectiveness of the detention pond evaluated in alternative 2. Indeed, even the combination of both alternatives does not solve the basement problems. The best results were obtained with the combination of the grass ditch, the diversion of the line and the enlargement of five pipes diameters (alternative 4). The longitudinal profiles with the comparison of the water level before and after the modifications of this alternative are included in Appendix 9, as well as a map with the location of each modification. The water level in the property is considerably decreased, although is still higher than the basement level. In addition, it should be considered that the water level rises along Vårvindsgatan (main line in Bräcke) as a result of the water diversion, although the properties previously flooded (Vårvindsgatan 17 and 19) have shown to handle a 10-year return rain.

The discharge of the pipe where Vårvindsgatan 45 is connected to the system (link 81) was plotted in the different alternatives to evaluate the effects in the network, see Figure 29. As can be seen in the graph, the discharge drops during the peak rains, which indicates that this line is affected by backflow. The detention pond does not have a great impact in the discharge, it just displaces slightly the discharge curve. Nevertheless, the diversion of water to other part of the system avoids the backflow, as well as reduces both the peak discharge and its duration.

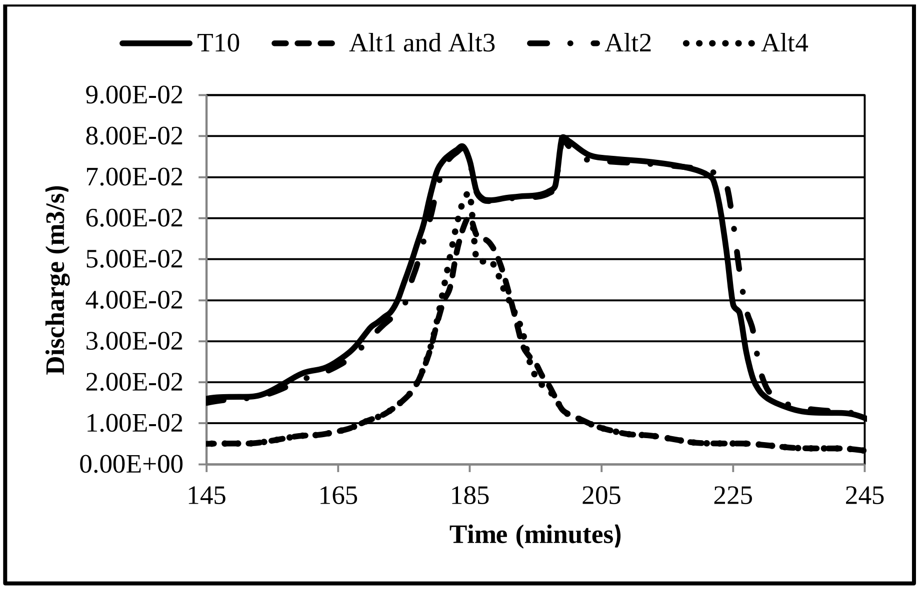

Figure 29. Discharge of the link where Vårvindsgatan 45 is connected to the network.

Despite all these improvements, the water level is not below the basement level in Vårvindsgatan 45 as mentioned before. Therefore, it is recommended to combine the alternative 4 with the installation of a non-return valve to avoid flooding. As this property is connected to a combine system, the valve should be placed in the wastewater connection. Figure 27 shows a schematic representation of the non-return valve installation.

Moreover, water detention effect was simulated by means of a grass ditch, although it is possible to use other alternatives as a gravel detention tank or stormwater boxes. By applying Equation 5 presented in sub-chapter 6.2.2.3, the total required volume for a gravel detention tank has resulted to be 184 m^3 of which 55.2 m^3 are water and 128.8 m^3 are gravel. The total required volume is reduced to 58.1 m^3 if the stormwater boxes are used instead.

6.3.1.6 Pilegårdsgatan and Prebendegatan

These two streets have a separate system. Sewage lines are working correctly, whereas stormwater lines are severely overloaded, see Appendix 10. Thus, the study was focused in the stormwater management. In addition, it was found that the cemetery located next to Pilegårdsgatan is connected to the network in one node at the end of the street. However, the model divides the cemetery in different sub-catchments connected along the street. To obtain a good understanding of the performance of the system and the different alternatives to improve it, both situations were considered and analysed. Table 18 shows the summary of the studied alternatives, along with the obtained results in the nodes and the pipes in the worst case scenario.

Table 18. Network improvements on Pilegårdsgatan (719 and 1739) and Prebendegatan (621 and 1712).

Alternative Description		Pipe filling (water height/pipe height)				Node flooding (Node water level-ground level)			
		T5		**BlockT5**		**T10**		**BlockT10**	
Link /Node		**719**	**621**	**719**	**621**	**1739**	**1712**	**1739**	**1712**
S6 Alt1	Grass ditch parallel to the street (outlet: 200mm)	10.3	9.92	10.1	9.08	0.35	0.39	0.30	0.31
S6 Alt2	Grass ditch (outlet: 200mm) and new stormwater pipe connected downstream Danagatan CSO.	10.3	9.37	10.1	9.07	0.32	0.28	0.29	0.24
S6 Alt3	Grass ditch parallel to the street (outlet: 200mm). New stormwater pipe and enlargement of pipe diameters Pilegårdsgatan.	0.78	9.37	0.75	9.07	-1.8	0.30	-1.8	0.25
S6 Alt4	Grass ditch parallel to the street (outlet: 200mm). new stormwater pipe and enlargement of pipe diameters in Pilegårdsgatan and in Prebedengatan.	0.78	1.54	0.75	1.25	-1.8	-0.8	-1.8	-1.0
S6 Alt5	New stormwater pipe. Enlargement of pipe diameters and the drainage of the cemetery is connected to the last node at Pilegårdsgatan.	0.90	2.37	0.81	1.83	-0.7	-0.2	-1.1	-0.6

Similarly to the previous cases, the continuous rains represent the worst case scenario in these streets. As can be seen from the table, either the water level or the pipe filling are greater in Prebendegatan than in Pilegårdsgatan.

A grass ditch which receives the drainage from the cemetery (Alternative 1) was modelled to delay the entrance of water in the system and to reduce the peak flow. The water level was lowered, but the capacity of the system was still exceeded. In order to decrease the water level, it is necessary to disconnect these streets from the main line, so that a new stormwater pipe connected to the line going to Göta Älv was also added to the network (Alternative 2). However, these measures were not enough to avoid the surface flooding due to the first pipes in the streets are undersized to handle the considered rains. Hence, the diameters of some pipes had to be increased as well (Alternative 3 and 4). The best results are obtained with Alternative 4, although some pipes are running under pressure during T5 and BlockT5 in Prebendegatan.

Appendix 8 includes a map with the modifications implemented in S6-Alt4 and a table with the modified diameters, as well as the longitudinal profiles with the performance of the system in the current situation and also with the measures. The water level drops considerably and no nodes are flooded, although the network is still surcharged during the peak rain in specific parts of these streets. The grass ditch influence and the system performance can be seen in Figure 30, which represents the discharge of the grass ditch in the system and the discharge of Pilegårdsgatan.

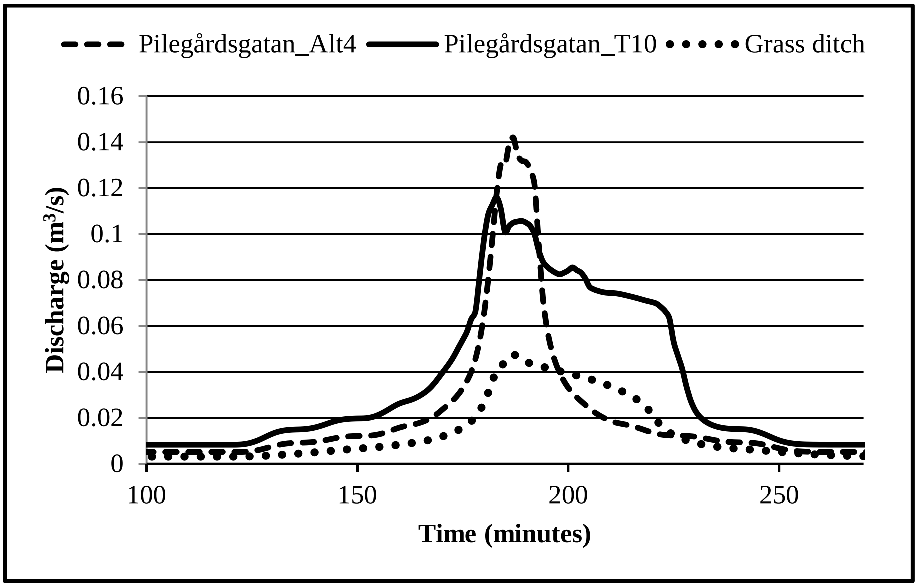

Figure 30. Discharge of the last pipe in Pilegårdsgatan and the last pipe in the grass ditch

In the current system, the discharge does not reach a value as high as in the alternative 4 since the main line is surcharged and water cannot be conveyed, as well as the backflow reduce the discharge value. The new stormwater pipe increases the discharge, despite the water level is reduced since this part of the system is not as overloaded. In addition, the grass ditch makes the peak discharge to move in time, as well as it stores water which is deliver to the system after the critical peak discharge.

Moreover, the model has been adapted to consider just one drainage outlet from the cemetery (Alternative 5). The water level decreases upstream the new connection point, but it increases downstream. Appendix 8 also includes longitudinal profiles with the alternative S6-Alt5. By analysing the profile it can be seen that the drainage of the cemetery has a great impact in the surrounding streets. In fact, the water level pattern differs considerably between the unaltered model and the new connection.

6.3.2 General measures

General measures are focused on improving the performance of an extent part of the network, making the current system more effective or increasing its capacity. The different actions that were simulated can be classified in three groups: sustainable drainage systems, reservoirs and diversion of pipelines to parts of the system with enough capacity. The results are presented as percentage of pipes running under pressure (pipe filling larger than 1) and the percentage of nodes flooding. The current system performance regarding these parameters is included in Table 19 to provide a basis to compare the effectiveness of the evaluated modifications.

Table 19. General pipe filling and node flooding results for T5, BlockT5, T10 and BlockT10

Pipe filling		**Node flooding**	
T5	**BlockT5**	**T10**	**BlockT10**
67%	62%	23%	19%

6.3.2.1 Sustainable drainage systems

The implementation of sustainable drainage systems in Lundby is limited by the space availability and the topography. However, three locations were found to be suitable for this kind of measures: Bräcke, Eketrägatan and Volvo industry. Table 20 shows the summary of the simulations that include sustainable drainage systems.

Table 20. Summary of the different alternatives evaluated with SUD's.

Alternative Description		Percentage of pipes under pressure		Percentage of nodes flooding	
		T5	BlockT5	T10	BlockT10
S7	Grass ditch from Tyskagatan and Stenåldersvägen until detention pond at Oslogatan (Bräcke). Outlet: 100mm.	63%	59%	21%	15%
S8	Grass ditch from Eketrägatan until a detention pond at Östra Bräckevägen (Sotérusgatan). Outlet: 150mm.	66%	62%	23%	19%
S9	Permeable pavement and gravel detention pond in Volvo parking lots. Outlet: 150 mm.	66%	61%	22%	19%

Appendix 11 includes longitudinal profiles with the obtained results for each one of the alternatives, along with situation maps.

The grass ditch and the detention pond in Bräcke (S7) result in a reduction of water level over the area, even though the catchment discharge is small due to the low imperviousness. In fact, the hydraulic line has dropped up to one meter in the line leading to CSO Danagatan, see Appendix 11.

However, the grass ditch implemented from Ekretagatan until Västra Bräckevägen (S8) is not as effective as the previous one. In this case, the results do not show any changes in the main line since the water reduction introduced by the grass ditch is negligible in comparison with the upstream discharge in the main line.

The alternative S9 (sustainable drainage systems at Volvo) has a local effect reducing the water level in Kohagsgatan (the street where Volvo is placed). Thus, the peak discharge into the main line is slightly decreased and delayed, although there is not a remarkable influence in the whole system. When modeling this alternative, green roofs and permeable pavements were to be simulated as well. However, some difficulties were faced in the process since the majority of the new calculated imperviousness values were found to be greater than the ones included in the model, see Appendix 11. Hence, it was decided not to evaluate the effect of green roofs and permeable pavements in this area.

6.3.2.2 Reservoirs

Reservoirs in the main line were implemented to regulate the water flowing in the system during the critical moments of the rainfall. Three different locations were studied, as well as different dimensions and characteristics in one of them in order to analyse how these parameters affect the network capacity. An overview of the simulated modifications and the obtained results is included in Table 21.

Table 21. Summary of the different alternatives evaluated.

Alternative Description		Percentage of pipes under pressure		Percentage of nodes flooding	
		T5	BlockT5	T10	BlockT10
S10-Alt1	Reservoir at Eketrägatan (football field). Capacity: 9000 m3 (3x30x100m). Outflow pipe: 500mm	61%	56%	18%	16%
S10-Alt2	Reservoir at Eketrägatan (football field). Capacity: 9000 m3 (3x30x100m). Outflow pipe: 500mm. Upstream pipes with increased diameter (link 177, 1646, 1492, 1640 from 1200 to 1800 mm)	60%	55%	15%	13%
S10-Alt3	Reservoir at Eketrägatan (football field). Capacity: 9000 m3 (3x30x100m). Outflow pipe: 800mm	67%	67%	18%	16%
S11	Reservoir in the connection with Volvo. Capacity: 6300 m3 (3x30x70m). Outflow pipe: 500mm	63%	56%	16%	14%
S12	Reservoir before Danagatan CSO (football field). Capacity: 9000 m3 (3x30x100m). Outflow pipe: 500mm	65%	60%	22%	18%

Appendix 10 includes the maps showing the location and characteristics of the reservoirs, as well as a table showing a comparison of the node flooding and pipe flooding situation in each alternative.

The three variants of the reservoir located underneath the football field at Eketrägatan improve considerably the system operation and the water level is dropped in the whole main line. Alternative S10-Alt2 also modifies the water level upstream the reservoir. However, the water level in the alternative with bigger outlet (S10-Alt3) is still too high downstream the reservoir. Alternative S11 (reservoir in the connection with Volvo) provides similar results to S10, besides a slightly reduction of water level upstream the reservoir. The reservoir at Danagatan (S12) does not have a great impact in the network since it is located too close to the outlet.

To evaluate the feasibility of each alternative a statistical analysis of the pipe flooding is included in Figure 31. As can be seen, alternatives S10-Alt2 and S11 get the maximum reduction of pipes flooding, despite the percentages in all alternatives do not present major differences. It is important to point out that the reservoir located in the connection with Volvo (S11) has a similar performance to the previously mentioned ones. In fact, the number of pipes flooding is practically the same and the difference lies in a higher percentage of pipes with the water level up to 1.5m below the surface.

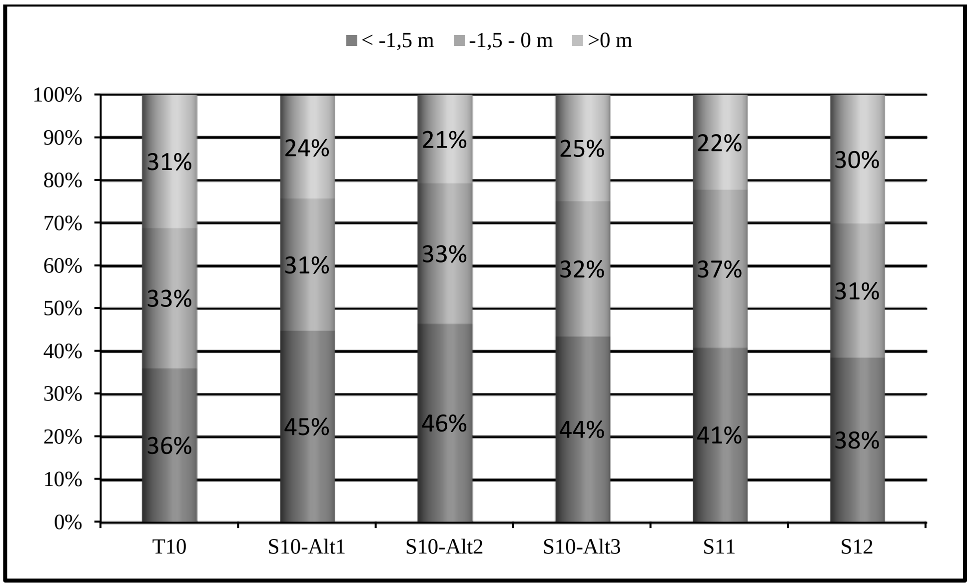

Figure 31. Pipe classification regarding pipe flooding for T10.

Therefore, according to the statistical study the proper location for a reservoir in the main line has resulted to be the middle of this line. Appendix 10 includes the pipe flooding classification within the area and a longitudinal profile of the alternative S10-Alt2.

Moreover, the performance of the reservoirs in alternative S10 has been analyzed to determine the optimal characteristics. Figure 32 shows the comparison between the inflow and the discharge in the reservoirs.

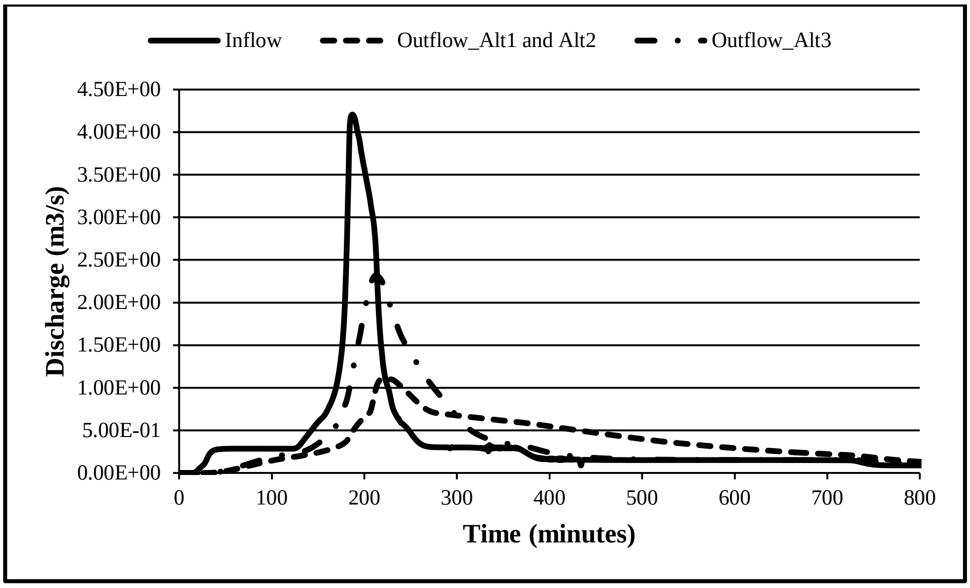

Figure 32. Inflow and discharge of the reservoirs.

As can be seen, the reservoirs have basically two effects in the network: reduction of the peak discharge downstream and displacement of the peak discharge in time. In fact, the peak discharge is moved 31 minutes from the greatest inflow with the simulations S10-Alt1 and S10-Alt2, while the peak discharge occurs 25 minutes later the maximum inflow in the simulation S10-Alt3. In addition, the diameter of the outflow link has a great impact in the time required to empty the reservoir. The reservoir with an outlet link of 800mm takes about 2.5 hours to discharge all the water, while the reservoir with the outlet of 500mm takes about 10 hours.

6.3.2.3 Diversion of pipelines

The diversion of water towards less overloaded stretches of the network is another approach considered in this study to solve the flooding problems. The water is redistributed within the existing network in order to optimize the system performance and make it more efficient. Hence, this alternative fits only parts of the system where an overloaded pipe is close to a pipe working with enough capacity. In addition, the topography and the invert level of the existing pipes represent also limitations for executing this action. For this reason it was not possible to implement this kind of modification in all the parts of the system that could have worked if the requirements were met. An overview of the diversion suggested and the obtained results is included in Table 22.

Table 22. Summary of alternative evaluated consisting in pipe diversion.

Alternative Description		**Percentage of pipes under pressure**		**Percentage of nodes flooding**	
		T5	**BlockT5**	**T10**	**BlockT10**
S13 Alt1	Diversion of Bräcke main line towards Krokängsgatan CSO instead of Danagatan CSO.	67%	61%	23%	19%

Longitudinal profiles of the lines affected by this modification are included in Appendix 11, along with a graphical description of the diversion. The diversion of the pipe coming from Bräcke towards Krokängsgatan CSO decreases the water level in the stormwater line leading to Göta Älv. Nevertheless, the water level is increased in the pipe coming from Bräcke, while it remains unaltered in the main line upstream Danagatan CSO. In addition, the general situation of the network does not have any noticeable changes, as Table 23 shows. This means that the water level decreases after Danagatan CSO, but the problem is moved to Bräcke pipeline instead.

Table 23. Pipe flooding and node flooding for S14 alternative for T10

		T10		Diversion to CSO Krokängsgatan	
Nodes flooding	< -1,5 m	371	44%	371	44%
	1,5- 0m	276	33%	275	33%
	>0 m	189	23%	190	23%
Pipe flooding	< -1,5 m	302	36%	300	36%
	1,5-0 m	276	33%	273	33%
	>0 m	261	31%	266	32%

In conclusion, this alternative is just valid to reduce the water level in the stormwater pipe, but it does not have a great impact in the general system performance.

7 Discussion

This chapter includes an evaluation of the results obtained in this study, besides the theories that support these findings.

7.1 Model and input data

It is important to consider that the reliability of the obtained results depends completely on the accuracy and calibration of the base model. In addition, the rainfall input data also determines the results. As both the model and the input data were provided, they were assumed to be representatives for this case study. However, it was found that the imperviousness of the sub-catchments and the drainage area do not always fit the reality, since they were modified to calibrate the model. As an example it can be mentioned that the imperviousness in some catchments in the Volvo area is underestimated. Thus the model is not completely reliable in some specific parts of the system, although it represents the overall network performance.

The sub-catchments are not created based on the topography, but they were created with a program assistant tool through bisectors between the nodes. This means that the real runoff distribution and generation may differ from the simulated one. In addition, all the sub-catchments were defined with the same initial loss, concentration time and reduction factor regardless the type of surface, size or conditions. As a result, the runoff in some parts of the system may be either over- or underestimated, although the model is calibrated to simulate the overall performance of the network. Indeed, it has been found that the nodes connected to big sub-catchments in green areas are flooded in the simulations, when the actual conditions should not lead to such event. Therefore, these nodes have not been considered in this study.

Moreover, hand calculations of the concentration time of two random sub-catchments were calculated to compare with the assumed value of 7 minutes. By using the empirical formula included in P90, the time of concentration in a sub-catchment of 0.5215 ha drainage area is 1.036 minutes, whereas in a sub-catchment of 0.186 ha drainage area is 0.6 minutes. Although the amount of runoff generation is calibrated for the characteristics of this area, the previously mentioned findings differ considerably from the assumed values and, consequently, the real runoff hydrograph shape may slightly vary.

The continuous rain is defined during one day, while the block rain is divided in three blocks of 10, 20 and 30 minutes. Therefore, the total runoff generation of the continuous and block rains cannot really be compared due to the characteristics of the data. However, it is possible to evaluate the duration and the value of the peak discharge of both rains. The maximum discharge during the 5-year return period block rain is 186 l/s·ha during 10 minutes, whereas the continuous rain discharges counts with lower values than 186 l/s·ha in the minutes before and after the peak discharge and reaches 234.7l/s·ha during just 2.5 minutes. Even though the intensity is higher during more minutes in the block rain, the simulations showed the continuous rains to be the most problematic scenario. Therefore, it can be affirmed that the problem for this particular network is not the duration of heavy rains, but the maximum extreme intensity. This idea is also supported by comparing the three block rains between them. The 10 minute rains creates the lower amount of runoff, although it is the rain with worst consequences of the three blocks.

7.2 Network performance results

First of all, it is important to remind that the calculations in MIKE URBAN are an approximation to the real conditions, but the exactitude is very difficult to achieve. Hence, these results should be considered just approximations to the real system performance.

In order to analyse the response of the network under the different rains in the whole area three main results were analysed: node flooding, pipe flooding and pipe filling. To check the likelihood of basement flooding, it was assumed that all the basements are located 1.5 m below the surface. Based on this, the pipes were classified according to the maximum water level in three categories: flooded (link flood greater than 0), risky area (link flood between -1.5 and 0) and safe area (link flood below -1.5m). However, this classification introduces two error sources: the basements level may considerably differ from 1.5 meters below the surface and the maximum water level in a certain point of the pipe is assumed to the whole pipe. Hence, pipe classification based on the pipe flooding is just an approximation to the exact results since some pipes may be classified as failed, even if the basements are not connected in the part of the pipe with higher water level, see Figure 33. In this way, the water level reached in some connections will be overestimated which means that the study is carried out with a safety factor always bigger than one (safe side).

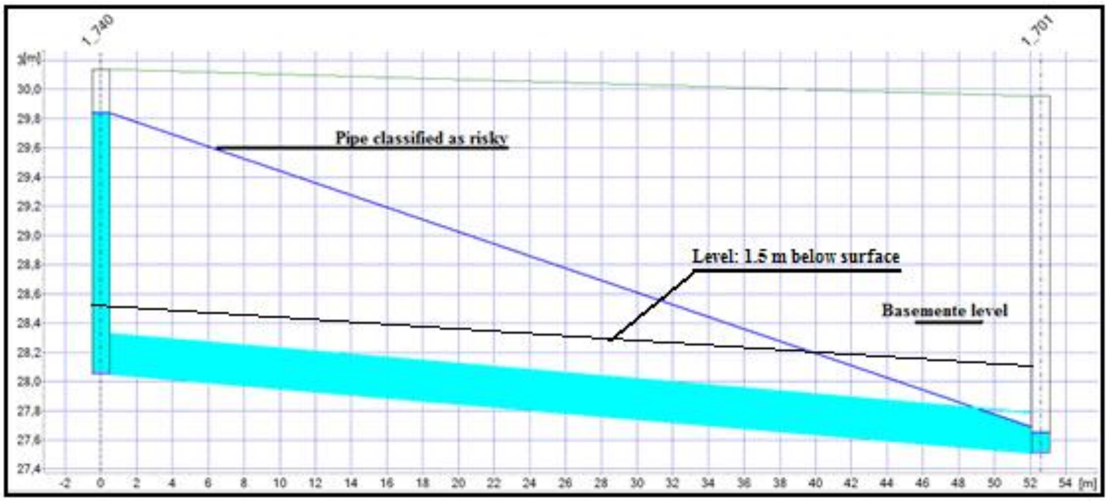

Figure 33. Pipe flooding classification and basement level location within the pipe. The pipe is classified as risky when the private connection is not affected by the water level.

As the sub-catchments with higher runoff generation are concentrated in the north-eastern part of the network, the main line and the secondary pipes connected to it are highly overloaded in the simulations, see Figure 34. The surcharge of the main line raises the water level at upstream areas, which worsen even more the situation. These results differ from the expected ones, since historical flooding events point out the southern part of Bräcke as the most likely flooding area instead of the north-eastern part of the system. As the calibration points were located in the south part of the model, an explanation for these unexpected results could be the large distance between the calibration points and that part of the system.

In addition, the other two problematic spots are Krykbyn and the north part of Bräcke (nearby Vårvindsgatan), see Figure 24. The choice of the properties to analyse was made in base of the flooded basements reported by the affected households. Hence, if any other basements were flooded but not reported to Gothenburg Water, they are not included in this investigation.

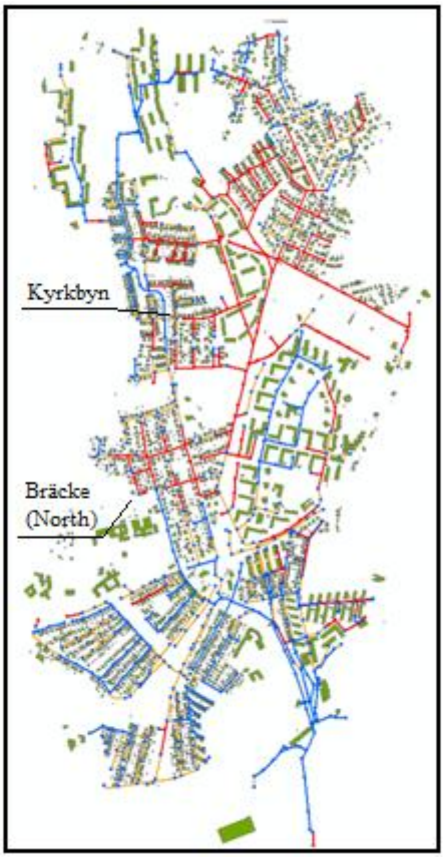

Figure 34. Pipe flooding, node flooding for T10.

Focusing the analysis on the previous flooded properties, the actions undertaken by Gothenburg Water to improve the system capacity have worked as expected according to the simulations. Indeed, the properties with basement flooding risk have been reduced from 39 to 5. This means that the enlargement of pipe diameters is a procedure that works properly in this area. Nevertheless, there is an exception in Slånbärsvägen 26, where the basement level is still lower than the water level despite three pipe diameters had their diameter increased. The simulations show that the local capacity of the pipes in this street is not enough to convey the runoff, which indicates that the required capacity is underestimated for the rainfall data considered in this study.

Moreover, the sanitary system in Pilegårdsgatan and Prebendegatan has shown to work correctly, although the stormwater system is severely overloaded. As many flooding problems were reported in these streets, it is reasonable to conclude that infiltration of runoff may occur in the sewage pipes, leading to the overburden of the sanitary system and basement flooding events.

All the network modifications were addressed to increase the system capacity in certain streets, which increases the rate of water to be conveyed in the whole system. Therefore, other approaches focused on decreasing the water in the system should also be considered.

7.3 Network modifications

Not all the modifications evaluated in this study have resulted to be suitable for this area. However, it is important to analyse their effects in the network in order to have a wide understanding of the sewage system operation. In addition, in those properties were system modifications are not feasible for economical or material reasons, non-return valves in the wastewater pipes could be an effective way of preventing basement flooding.

The grass ditch nearby AC Lindblads Gata has almost no effect in the network since the surcharge of the main line has too much influence in the water level upstream. However, the grass ditch seems to be a suitable solution to decrease the water entering in the system as long as the water level in the main line is reduced. Otherwise, the reservoir has resulted to be a suitable solution, although the dimensions determine the effectiveness. In this case a reservoir with greater top view area works better than a narrower reservoir with higher height, as expected. It is important to point out that the reservoir is acting upstream, which is an uncommon effect of the reservoirs. This is due to the reservoir receives and stores water from the main line during the peak rain and, in this way, it avoids the water level to rise upstream. Not only is reduced the water level in the street under study, but also in all the pipes in this area.

In the case of Finlandsgatan the only alternative that solves the basement problems comprises the replacement of four pipes and rises the water level in other parts of the system. With this alternative the pipes are running under pressure during the 5-year return period rain, which means that the pressure restriction included in the P90 is not met. In addition, the water is diverted towards the Gryaab WWTP, increasing the rate of water to be treated. Therefore, the diversion is not recommended for this street. As a reservoir in the main line makes the water level to drop in Finlandsgatan, this option is suggested instead.

In Slånbärsvägen and Visthusgatan the most feasible option is the enlargement of pipe diameters since the network has enough capacity downstream. In addition, the good results of the similar modifications implemented in past years support this alternative. However, the case of Slånbärsvägen is special since the pipes have been recently replaced. If the pipes have to be enlarged again, it may be reasonable to duplicate the system in this street.

The potential solutions evaluated at Vårvindsgatan 45 are not as effective as expected. Indeed, it was not possible to lower the water level below the basement with the local modifications. However, the combination of the alternative 4 with a reservoir in the main line can solve the basement problems in this property. If this alternative were implemented, it is important to consider that the grass ditch is located in bedrock with low permeability. Hence, the function of the detention facility (grass ditch, stormwater box or gravel detention tank) is just the temporal storage of water since infiltration is not possible in this area. As this alternative involves high investment with low benefits (the basement flooding risk cannot be avoided), the alternative of a non-return valve in the private wastewater connection may arise as suitable solution to prevent flooding.

The case of Pilegårdsgatan and Prebendegatan is singular since the network in these streets is duplicated, but the flooding problems remain. According to the results it is highly recommended to disconnect the separated pipes from the main line and divert the runoff into the stormwater pipe located downstream Danagatan CSO instead. With this modification not only would decrease the water level in the pipes, but also the water load going to Gryaab WWTP and the combine water discharged in the CSO are decreased. The drainage of the cemetery should also be modified to collect the runoff through a grass ditch in order to delay the peak flow in the stormwater pipe. In addition, the pipe diameters in Pilegårdsgatan and Prebendegatan should also be increased to handle the runoff.

Apart from the local modifications, it also needed to improve the overall network performance since the current system has not enough capacity to handle the 10-year return period rains. It is not a punctual problem, but several parts of the network spread within the whole area are overloaded. This makes more difficult to find suitable solutions to decrease the water level in general. In fact, the reservoirs have resulted to be the most effective solutions, although the maximal reduction in nodes flooding is just about 8%. Anyway, the reservoir in the main line reduces considerably the water level, which indirectly reduces the water level in the secondary pipelines (branches) connected to it. The reservoir located underneath the football field at Eketrägatan in combination with the increment of the diameter of the pipes prior to it (S10-Alt2) gives the maximum reduction of node and pipe flooding. However, either the same reservoir without the pipe increment (S10-Alt1) or the reservoir upstream in the main line (S11) leads to similar results with less investment. Comparing S10-Alt1and S11 it can be seen that the percentage of nodes and pipes flooding is smaller in the second one, but the percentage of nodes and pipes with water level below -1.5m is greater in the S10-Alt1, i.e. the number of pipes and nodes in the risky area is bigger in S11. Hence, S11 reduces the nodes and pipes flooding, but increases the percentage of the ones in the risky area. With smaller volume and similar results, the reservoir in alternative S11 stands as a very feasible solution to improve the general system performance. Nevertheless, if this alternative is to be combined with the reservoir in AC Lindblads Gata, the optimal location would be the football field instead.

The effectiveness of sustainable drainage systems depends on each specific case. For example, the implementation of a grass ditch along Bräcke is highly recommended since it reduces up to one meter the water level in the main line, whereas the effect of the grass ditch along Sotérusgatan could be neglected. As the runoff generation in these sub-catchments is very small due to the assumed imperviousness parameters, the effectiveness is reduced. In the case of Volvo, green roofs or impermeable pavements could not be simulated due to the characteristics of the model. However, gravel detention ponds lower the hydraulic line in the main line. Since Volvo is a private industry, Gothenburg Water can suggest the implementation of SUD´s in that area, but the final decision depends on Volvo Industry.

Moreover, even though the infiltration and evaporation rates are not significant in this area, SUD´s modelling can be improved by considering these two processes. In general, the effectiveness of sustainable drainage systems is quite limited during peak rains in this case study, but they have shown to be a good complement to other infrastructural modifications during heavy rain events. Therefore, SUD´s stand as proper options to reduce the percentage of runoff to be conveyed by the drainage system.

Finally, the reduction of water in CSO Danagatan by diverting the main line from Bräcke towards CSO Krokängsgatan has shown to move the problem to other area. As this alternative does not introduce any advantage, it should not be considered.

If all the recommended modifications were implemented in this sewage system, the nodes flooding during T10 would be reduced in 11%, whereas the percentage of pipes running under pressure during T5 in 12%. Figure 35 shows a graphic representation of how the system would be.

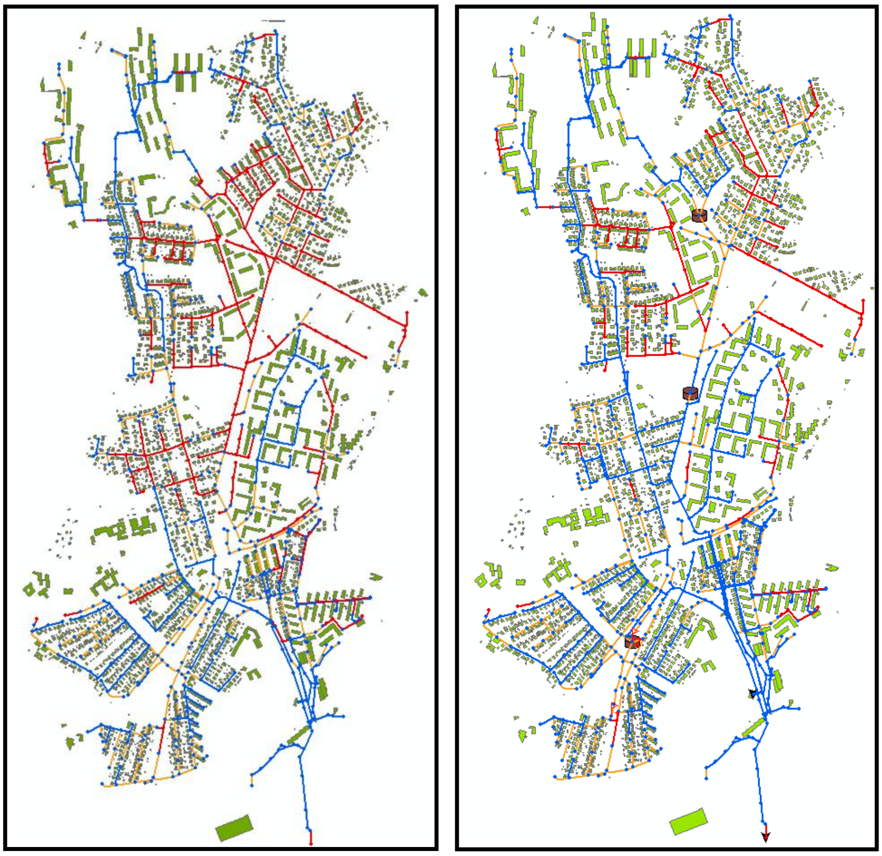

Figure 35. Comparison pipe flooding and node flooding for T10 after the optimal modifications: S1-Slt1, S3-Alt3, S4-Alt1, S5-Alt4, S6-Alt4, S7-Alt1 and S10-Alt1

8 Final Remarks

8.1 Conclusions

Conclusions that can be drawn from this case study are summarised in this sub-chapter.

- Continuous rains have shown to be the worst case rainfall scenario in this sewage system. Indeed, the duration of the rainfall is not as influent as the intensity when it comes to flooding risk, i.e. heavy instantaneous rains are more dangerous than medium-intensity long rains.

- With 67% of pipes running under pressure during 5-year return period rain and a 23% of nodes flooded during a 10-year return period rain it can be affirmed that the sewage system situation is critical. The flooding risk is very high due to the sewage system is undersized to cope with the current rainfall rates, but also is very limited to face the upcoming predicted rainfall. The most affected areas are the main northern main line and Bräcke, although other singular spots are also likely to be affected by flooding.

- The actions undertaken by Gothenburg Water, mainly replacements of pipelines, are working correctly so far, with the exception of just one street. This means that the enlargement of pipes is a suitable approach to solve local flooding problems. However, this kind of system modifications should be carefully studied since they could move the problem to other parts of the system instead of solving the flooding risk.

- Sustainable drainage systems are suitable options to reduce the peak flows in the system, but they are not enough to avoid flooding in this area. Hence, it is necessary to combine environmentally friendly measures with new infrastructure to increase the system capacity.

8.2 Suggestions for further research

Some recommendations are presented below in case further studies about stormwater management in Lundby-Kyrkbyn area are to be carried out with the current model.

- Pipe diameters should be completely updated according to Solen X (Gothenburg Water database). The model includes some pipes whose dimensions do not correspond to the database, so that a review of the model is recommended to increase the model reliability.
- Since the main line has shown to be severely overloaded, it is recommended to study the feasibility of increasing its capacity either by duplicating the network or replacing the current pipes.
- Evaluation of a grass ditch network in the whole area connected to Göta Älv in order to reduce the runoff to be conveyed by the sewage system, as well as to lower the rate of water to be treated in the WWTP. The performance of the grass ditch network should be optimized to collect as much water as possible from more impervious areas.
- Study local system improvements for two critical areas in Kyrkbyn previously mentioned. According to the obtained results for similar areas, the problem in

these areas should be the local insufficient capacity, so that pipe enlargement stands as a feasible solution that should be investigated.

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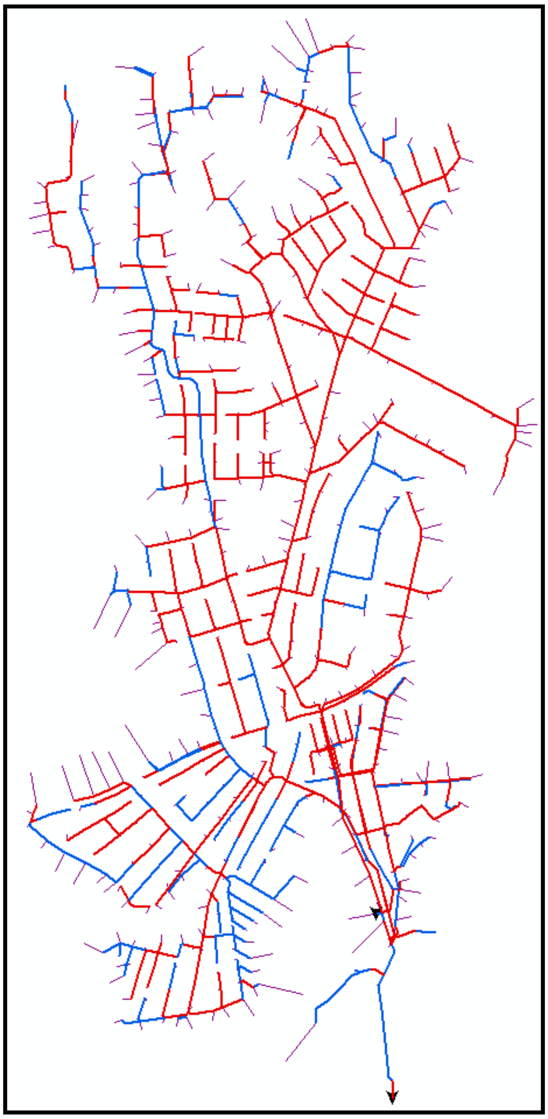
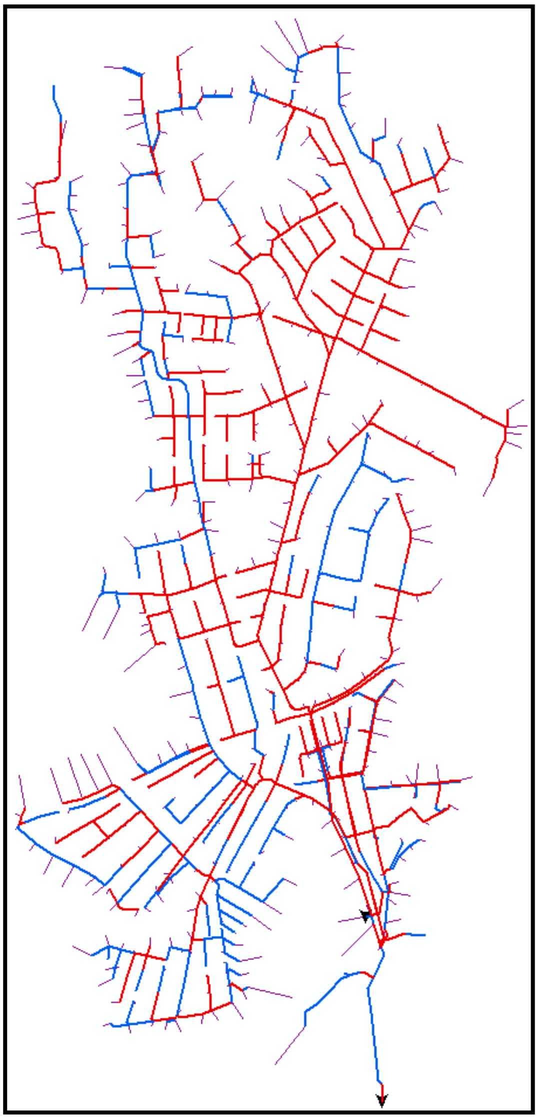
APPENDICES

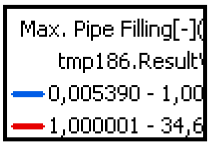
NOTE: The pictures in colour are published in the online version of this thesis.

Appendix 1. Rainfall data

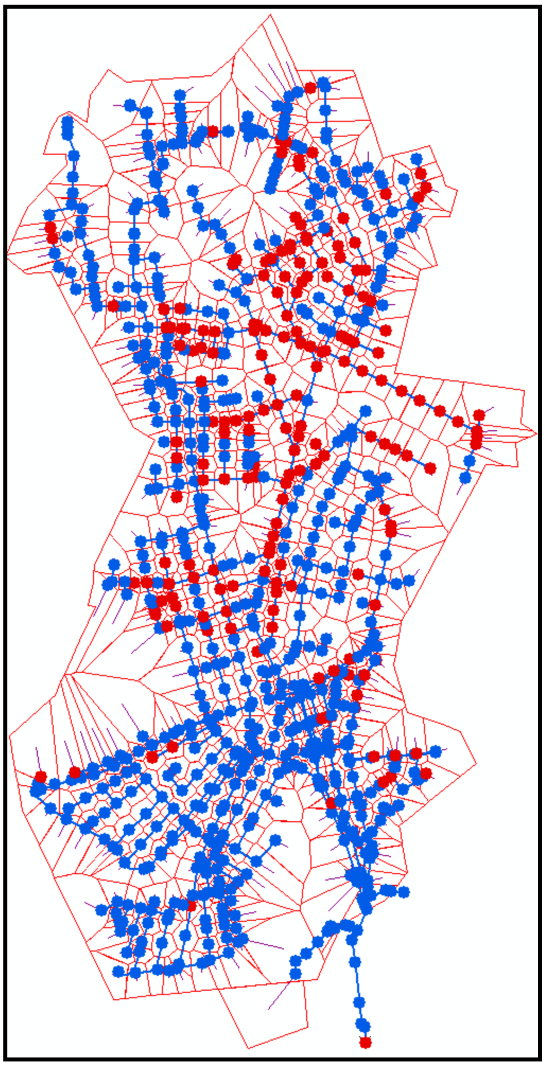
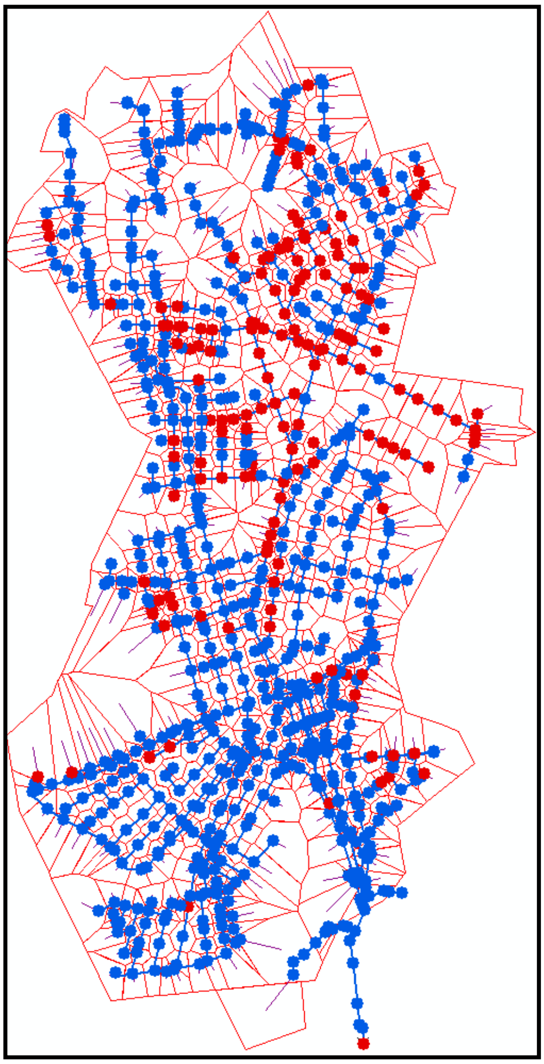
Time	SWE_TYP1_5 [mu-m/s]	SWE_TYP1_10 [mu-m/s]	SWE_TYP1_20 [mu-m/s]	SWE_TYP1_50 [mu-m/s]	SWE_TYP1_100 [mu-m/s]
2006-03-02 12:00:00.000	0	0	0	0	0
2006-03-02 14:00:00.000	0.66	0.77	0.89	1.08	1.24
2006-03-02 14:15:00.000	1.17	1.39	1.63	2.03	2.40
2006-03-02 14:30:00.000	1.53	1.83	2.20	2.78	3.30
2006-03-02 14:40:00.000	2.13	2.58	3.15	4.05	4.90
2006-03-02 14:45:00.000	2.87	3.53	4.33	5.70	7.00
2006-03-02 14:50:00.000	3.87	4.83	6.03	8.10	10.10
2006-03-02 14:52:30.000	5.33	6.73	8.53	11.67	14.80
2006-03-02 14:55:00.000	7.33	9.40	12.13	16.87	21.60
2006-03-02 14:57:30.000	12.20	15.93	20.73	29.33	38.00
2006-03-02 15:02:30.000	23.47	28.83	35.43	46.53	57.20
2006-03-02 15:05:00.000	12.20	15.93	20.73	29.33	38.00
2006-03-02 15:07:30.000	7.33	9.40	12.13	16.87	21.60
2006-03-02 15:10:00.000	5.33	6.73	8.53	11.67	14.80
2006-03-02 15:15:00.000	3.87	4.83	6.03	8.10	10.10
2006-03-02 15:20:00.000	2.87	3.53	4.33	5.70	7.00
2006-03-02 15:30:00.000	2.13	2.58	3.15	4.05	4.90
2006-03-02 15:45:00.000	1.53	1.83	2.20	2.78	3.30
2006-03-02 16:00:00.000	1.17	1.39	1.63	2.03	2.40
2006-03-02 18:00:00.000	0.66	0.77	0.89	1.08	1.24
2006-03-03 00:00:00.000	0.35	0.39	0.45	0.53	0.60
2006-03-03 12:00:00.000	0.21	0.23	0.26	0.30	0.34

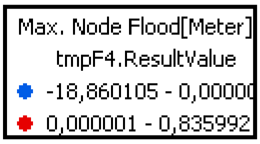
Appendix 2. Maximum pipe filling in pipes for 5-year return period rainfall

Continuous rainfall	Block rainfall
	



Appendix 3. Maximum nodes flooding for 10-year return period rainfall

Continuous rainfall (T10)	Block rainfall (BlockT10)
	

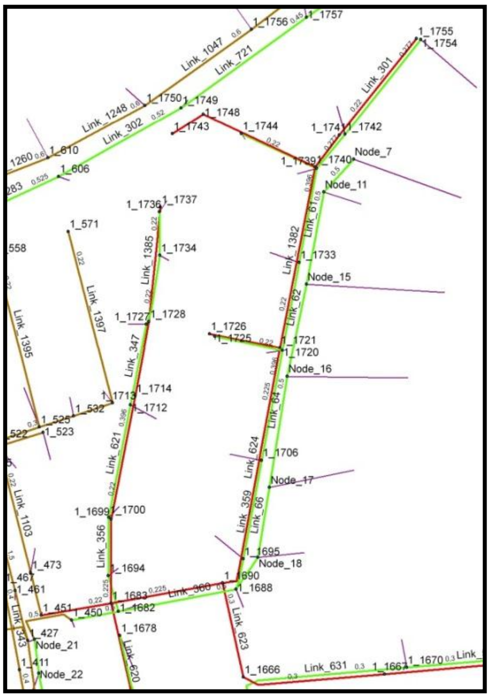


Appendix 4. Pipe filling and water level in properties previously affected by basement flooding

	Maximum pipe filling	
	T5	**BlockT5**
AC Lindblands Gata, 5	7.56	7.09
Biskopsgatan 17	1.69	1.59
Dysiksgatan 17	1.81	1.69
Dysiksgatan 19	1.81	1.69
Dysiksgatan 23	2.19	1.90
Finlandsgatan 2	7.05	6.39
Kaprifolievägen 16	0.55	0.49
Kaprifolievägen 18	0.55	0.49
Kaprifolievägen 20	0.42	0.54
Kaprifolievägen 22	0.42	0.54
Kaprifolievägen 24	0.42	0.54
Kaprifolievägen 26	0.42	0.54
Londongatan 8	0.43	0.37
Londongatan 10	0.43	0.37
Londongatan 17	0.70	0.68
Londongatan 19	0.56	0.68
Londongatan 21	0.56	0.68
Slånbärsvägen 26	2.95	2.16
Stamrotesvägen 27	0.70	0.79
Vårdvindsgatan 17	0.68	0.63
Vårdvindsgatan 19	0.68	0.63
Vårdvindsgatan 45	9.48	12.7
Visthusgatan 84	6.1	5.4

COMBINED SYSTEM	**BASEMENT LEVEL**	**T10**	**Difference**	**BlockT10**	**Difference**
A C Lindblads Gata, 5	35.36	36.85	-1.49	36.73	-1.37
Biskopsgatan 17	19.37	18.51	0.86	18.22	1.15
Dysiksgatan 17	22.7	22.61	0.09	22.29	0.41
Dysiksgatan 19	22.91	22.71	0.2	22.33	0.58
Dysiksgatan 23	23.54	22.89	0.64	22.61	0.93
Finlandsgatan 2	20.11	21.31	-1.2	20.18	-0.07
Kaprifolievägen 16	24.65	24.16	0.49	24.11	0.54
Kaprifolievägen 18	24.66	24.24	0.42	24.21	0.45
Kaprifolievägen 20	24.66	24.3	0.36	24.32	0.34
Kaprifolievägen 22	24.89	24.41	0.48	24.43	0.46
Kaprifolievägen 24	24.83	24.52	0.31	24.55	0.28
Kaprifolievägen 26	25.18	24.61	0.57	24.63	0.55
Londongatan 8	19.76	19.41	0.38	19.38	0.38
Londongatan 10	19.81	19.11	0.8	19.01	0.8
Londongatan 17	20.27	19.45	0.82	19.51	0.68
Londongatan 19	20.47	19.84	0.63	19.71	0.76
Londongatan 21	20.58	20.51	0.07	19.9	0.76
Slånbärsvägen 26	27.83	28.85	-1.02	28.65	-0.82
Stamrotesvägen 27	27.6	27.42	0.18	27.36	0.24
Vårdvindsgatan 17	26.85	26.38	0.47	26.35	0.50
Vårdvindsgatan 19	27.06	26.45	0.61	26.42	0.64
Vårdvindsgatan 45	28.43	30.37	-1.94	30.18	-1.75
Visthusgatan 84	27.49	27.82	-0.33	27.73	-0.24

Note: Data in meters.



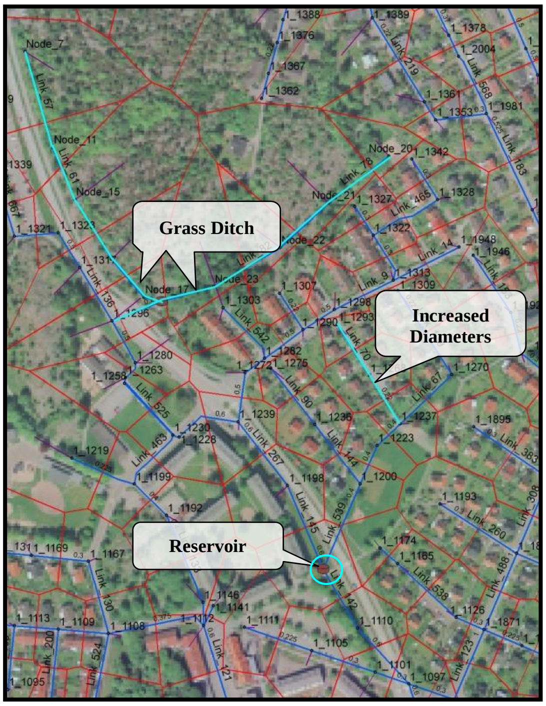
	Link	T5	BlockT5
Biskopsgatan	361	3.57	3.34
Pilegårdsgatan	552	11.19	10.99
	719	11.44	11.04
	486	11.44	11.01
	353	10.22	7.91
	352	6.37	2.23
	359	5.79	2.32
	625	6.84	4.83
Prebedengatan (Including Biskopsgatan 5)	723	6.02	5.28
	622	8.83	8.24
	347	10.19	9.32
	621	9.96	9.12
	356	4.00	3.26
	357	5.39	4.96

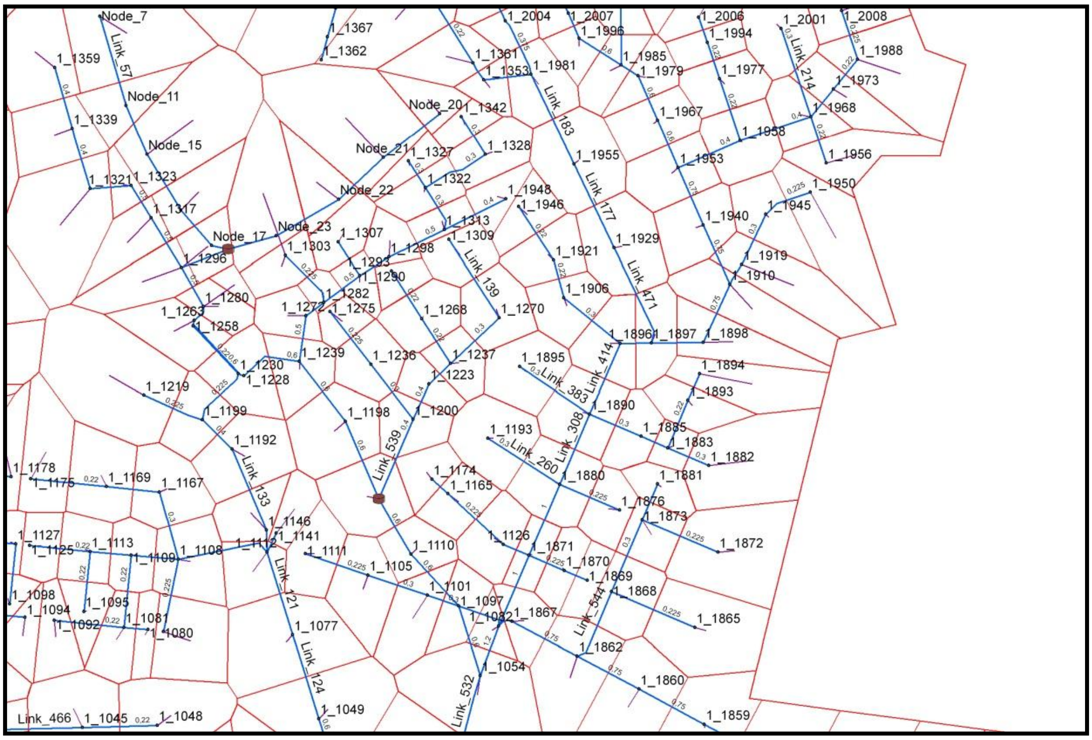
	Node	Node level	T10	Difference	BlockT10	Difference
Biskopsgatan	1_1450	20.07	18.98	1.09	18.70	1.37
Pilegårdsgatan	1_1754	22.83	23.3	-0.47	23.30	-0.47
	1_1742	21.92	22.38	-0.46	22.39	-0.47
	1_1739	21.64	22.1	-0.46	22.10	-0.46
	1_1733	21.13	21.4	-0.27	21.18	-0.05
	1_1720	20.65	20.86	-0.21	19.60	1.05
	1_1706	20.46	20.38	0.08	19.48	0.98
	1_1695	20.67	19.6	1.07	19.36	1.31
	1_1688	20.77	19.21	1.56	19.20	1.57
Prebedengatan (Including Biskopsgatan 5)	1_1736	22.94	22.48	0.46	22.09	0.85
	1_1734	22.78	22.4	0.38	22.06	0.72
	1_1727	21.72	22.02	-0.3	21.93	-0.21
	1_1712	20.54	20.9	-0.36	20.85	-0.31
	1_1700	20.33	19.6	0.73	19.38	0.95
	1_1694	20.29	19.3	0.99	19.11	1.18
	1_1682	20.23	19.02	1.21	18.91	1.32

Note: Biskopsgatan 5 is connected to the line in Prebedengatan

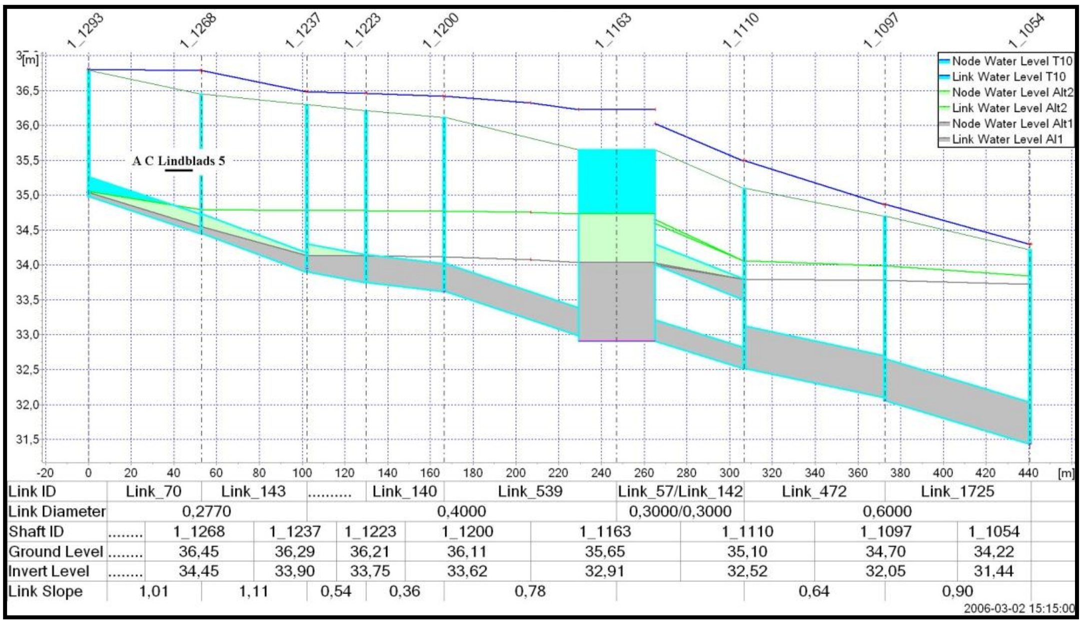
Appendix 5. Results of the network modifications in A C Lindblads Gata

Situation map





A C Lindblads Gata- Alternatives 1 and 2



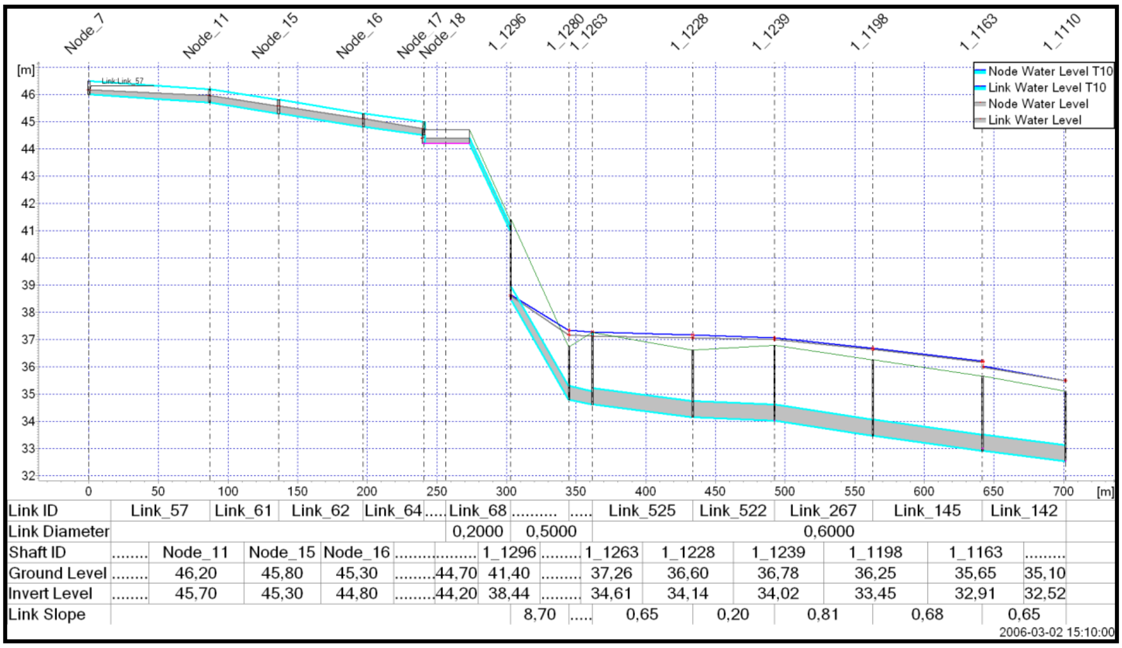
Diameters

Link	Slope (%)	Maximun Discharge (m^3/s)	Diameter Colebrook (mm)	PP_Urib (Inner) (mm)
70	1.01	0.026	200	277
143	1.11	0.062	230	277

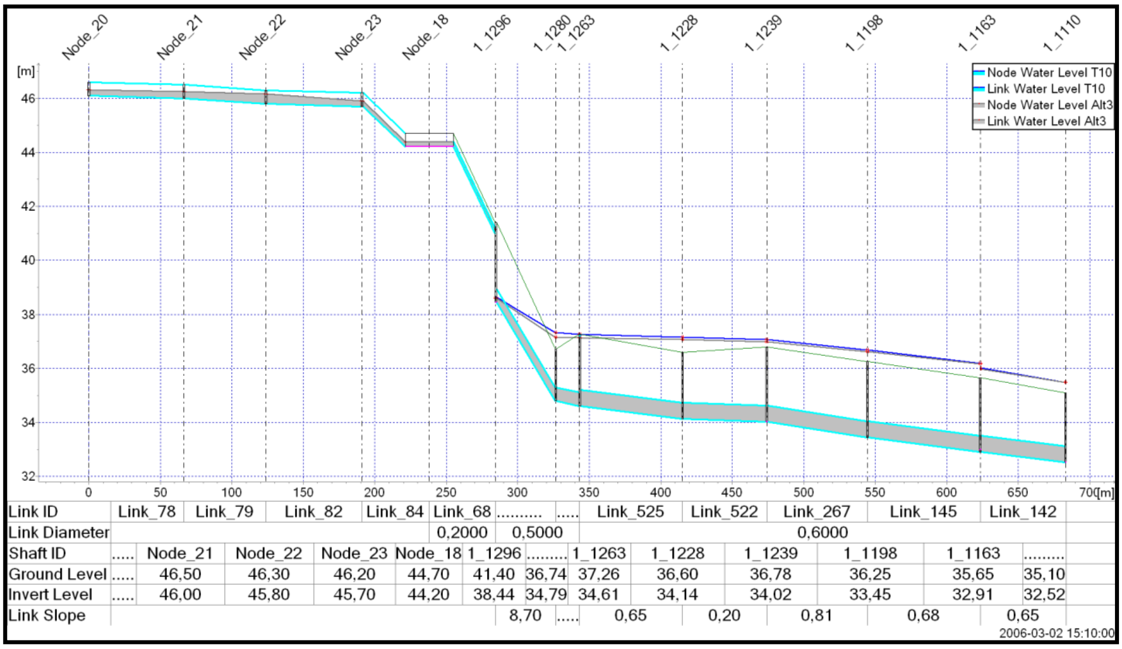
Node flooding – Alternative 1

A C Lindblads Gata- Alternative 3

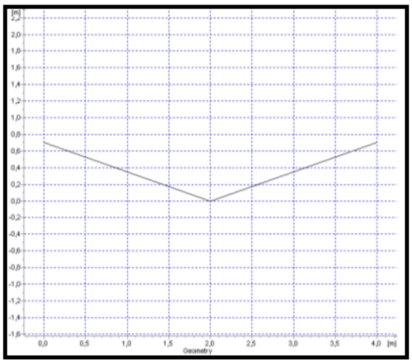
Northern grass ditch

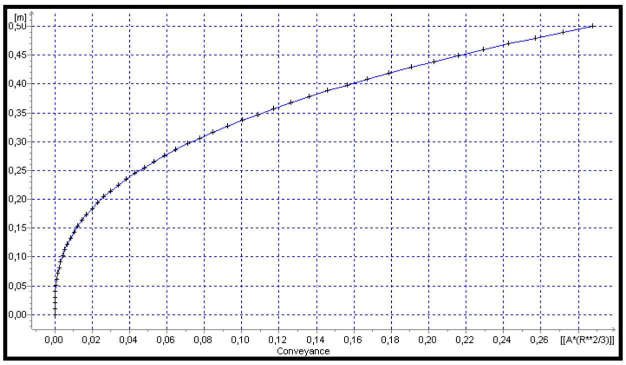


Eastern grass ditch



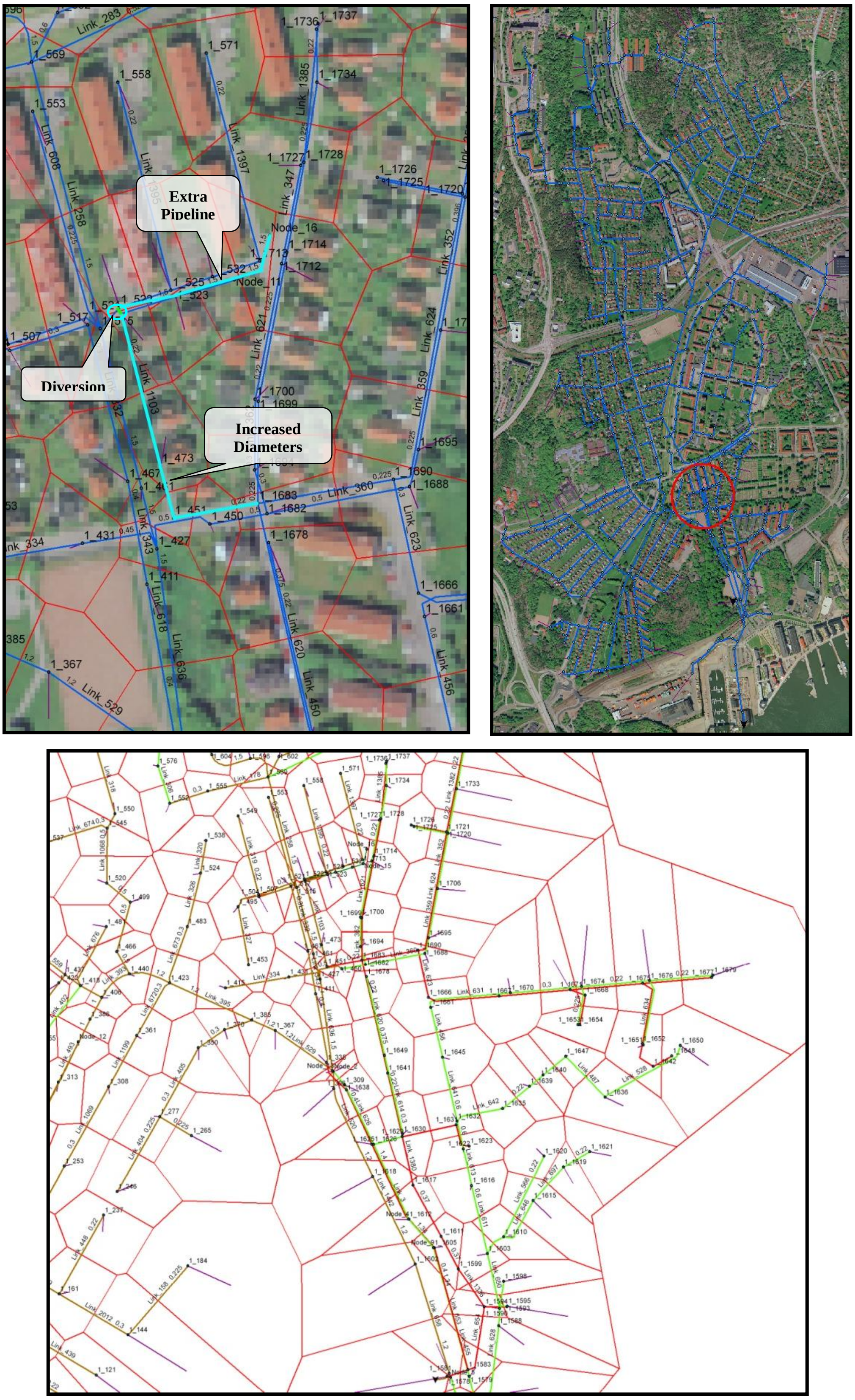
Grass ditch characteristics (n=0,04)



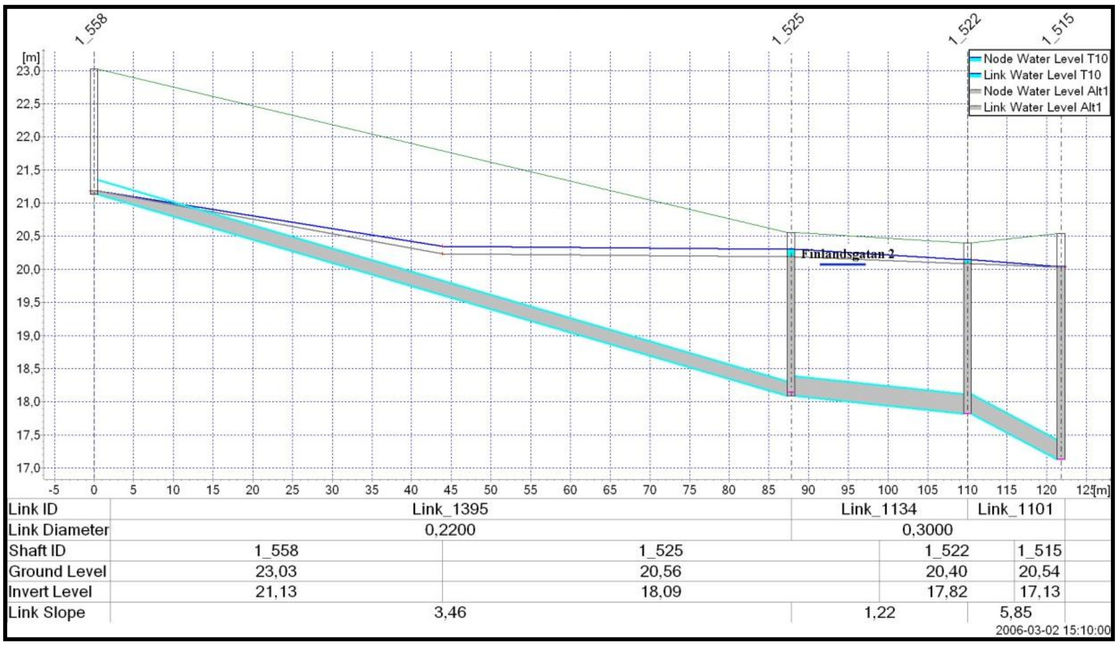


Appendix 6. Results of the network modifications in Finlandsgatan

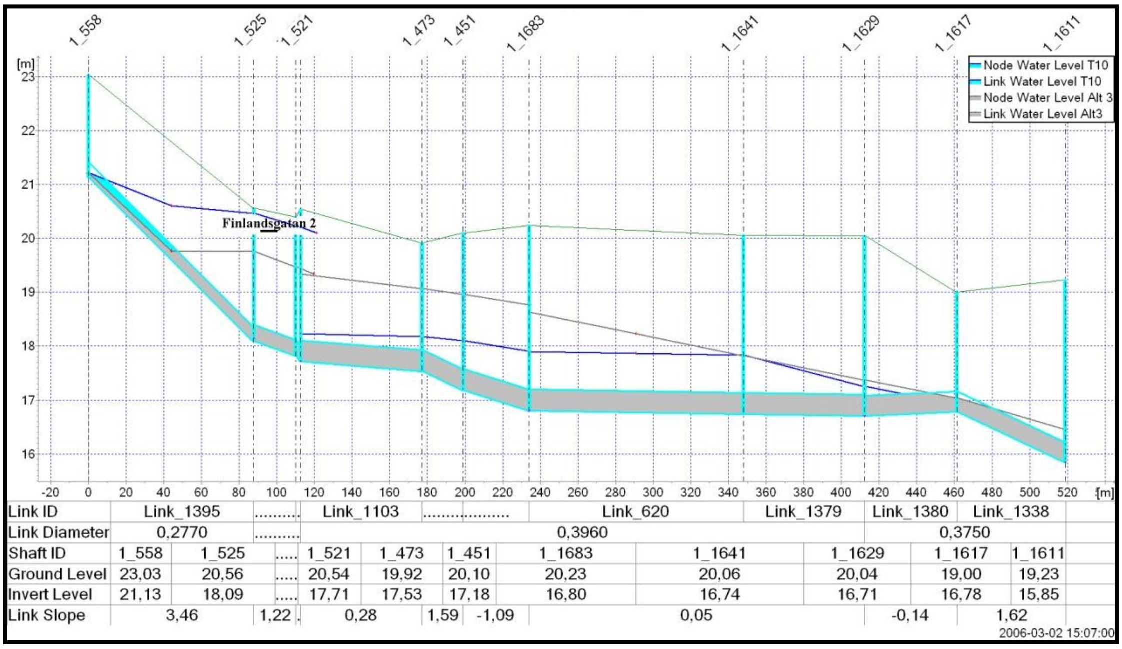
Situation map



Finlandsgatan- Alternative 1



Finlandsgatan- Alternative 3

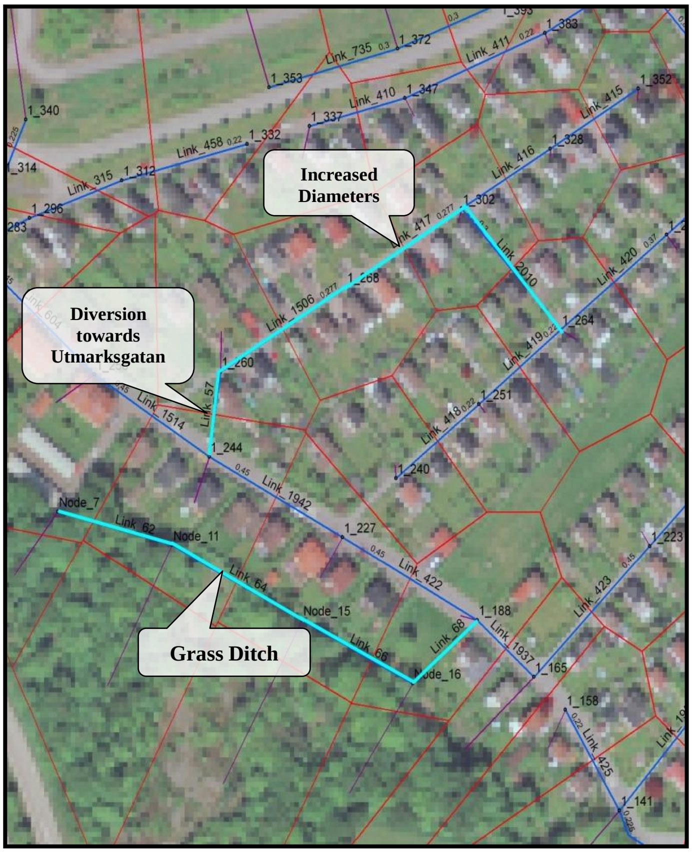


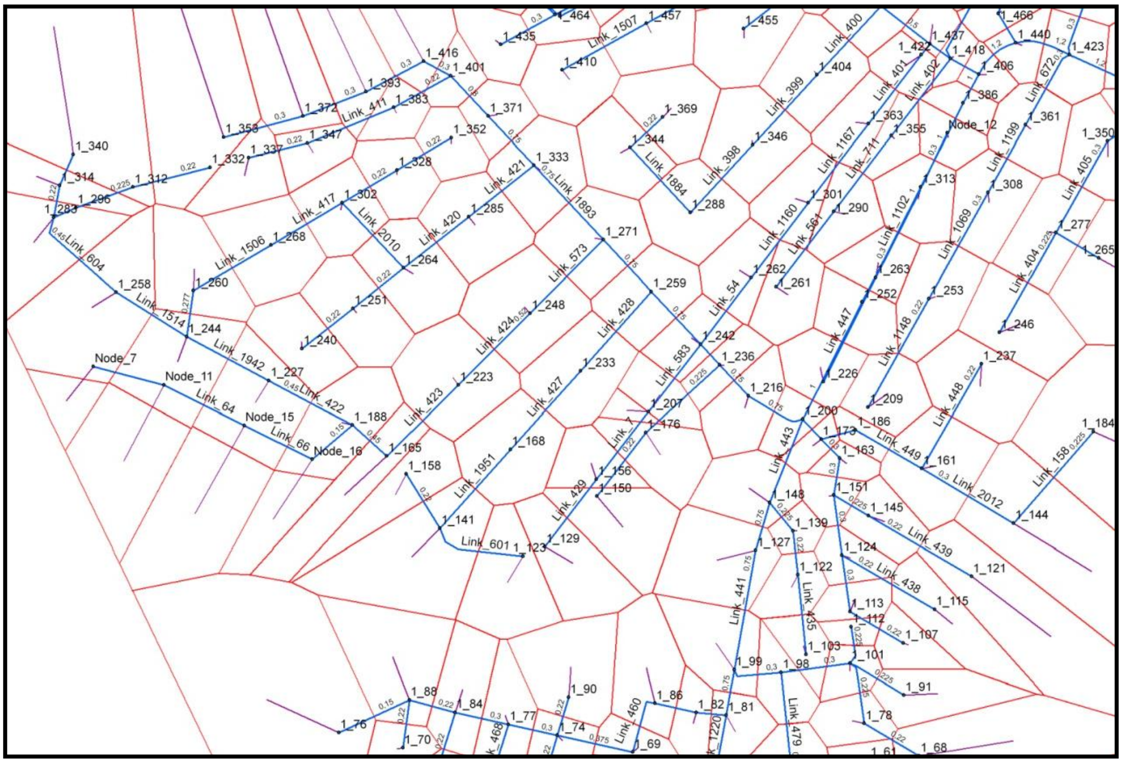
Diameters

Link	Slope (%)	Maximun Discharge (m^3/s)	Diameter Colebrook (mm)	PP_Urib (Inner) (mm)
1135	3.03	0.006	100	277
1103	0.28	0.014	175	396
1091	1.59	0.03	175	396
1381	1.09	0.036	200	396

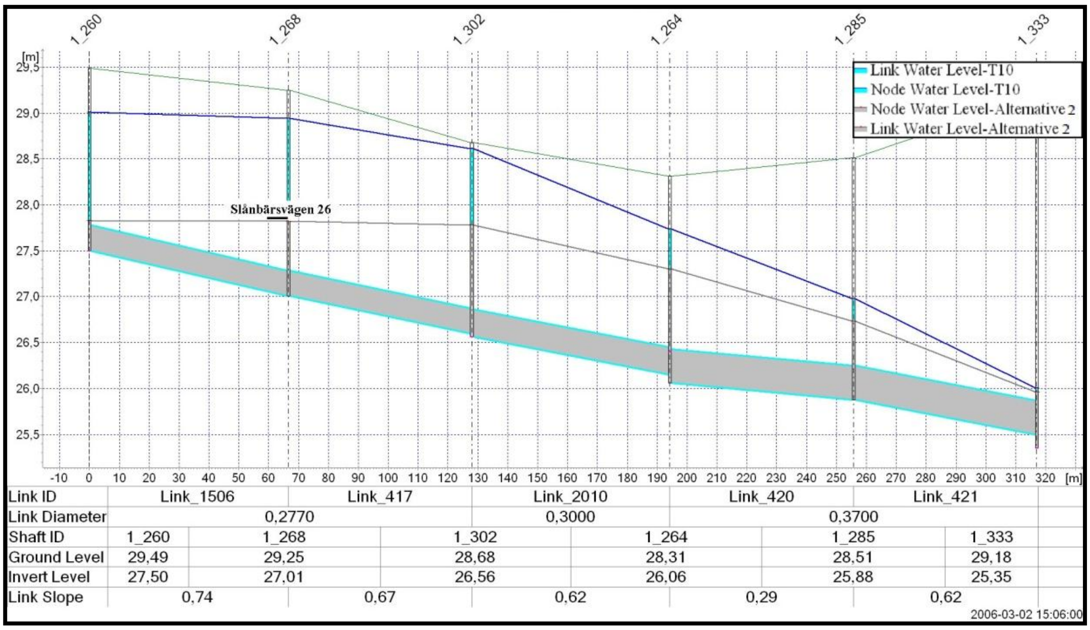
Appendix 7. Results of the network modifications in Slånbärsvägen

Situation map

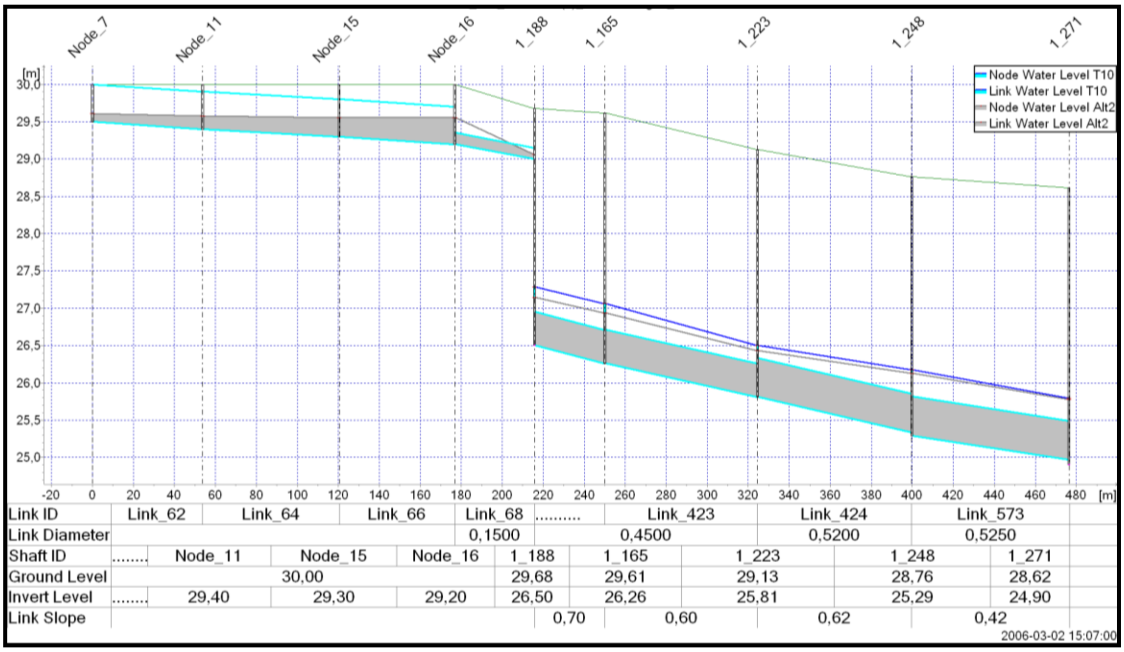




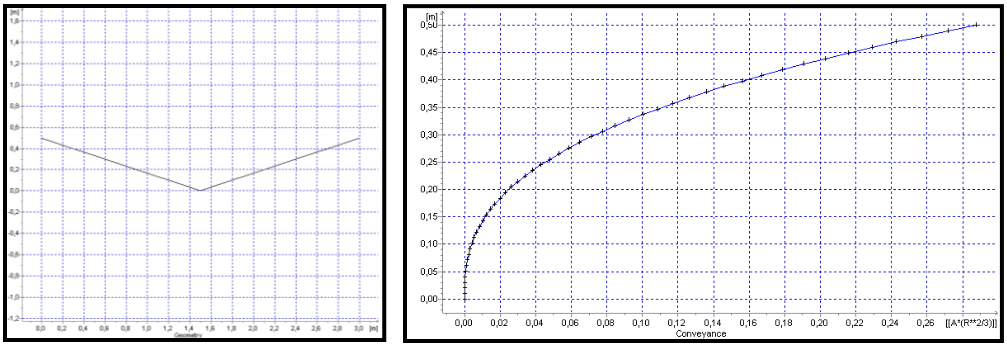
Slånbärsvägen 26-Alternative 2



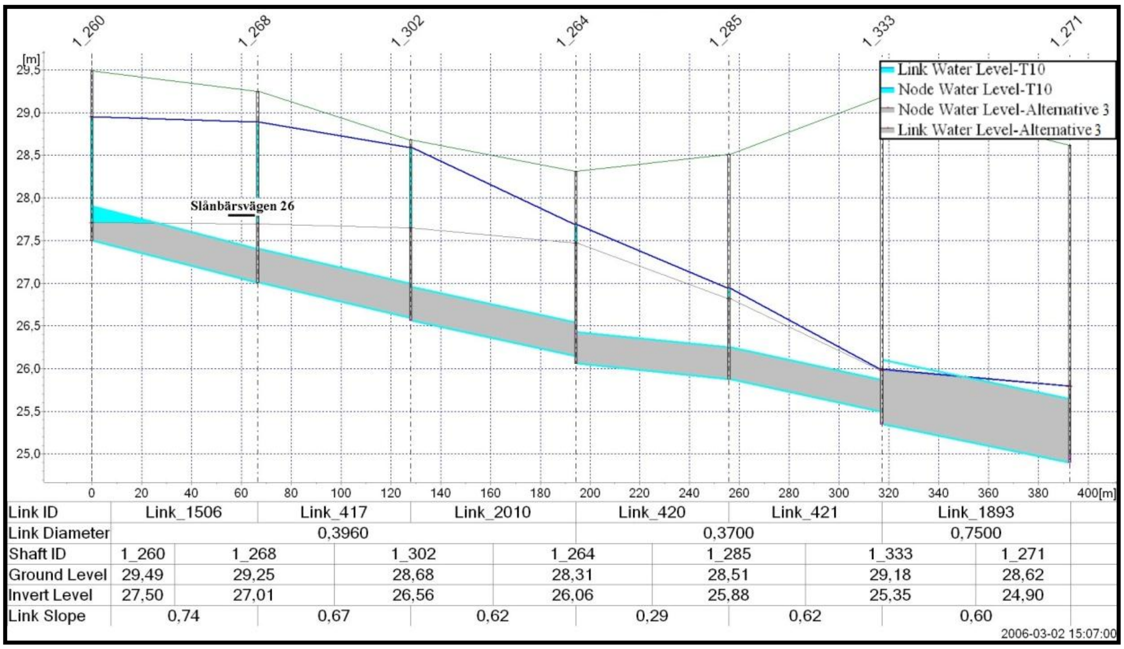
Grass ditch



Grass ditch characteristics (n=0,04)



Slånbärsvägen 26-Alternative 3



Diameters

Link	Slope (%)	Maximun Discharge (m^3/s)	Diameter Colebrook (mm)	PP_Urib (Inner) (mm)
1506	0.74	0.026	280	396
417	0.67	0.058	300	396

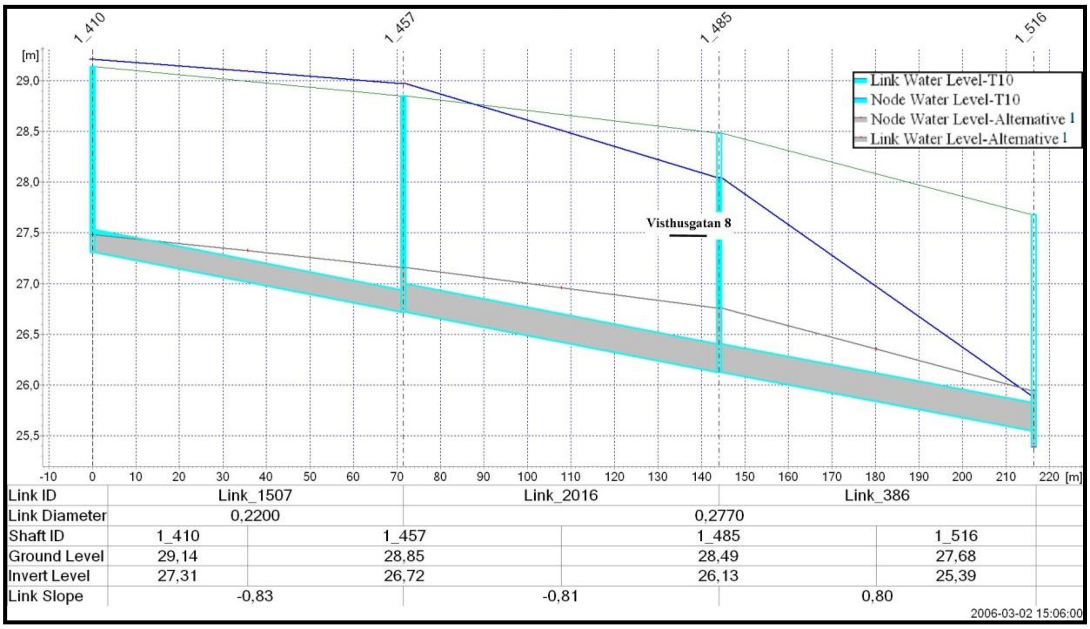
Appendix 8. Results of the network modifications in Visthusgatan

Situation map

Diameters

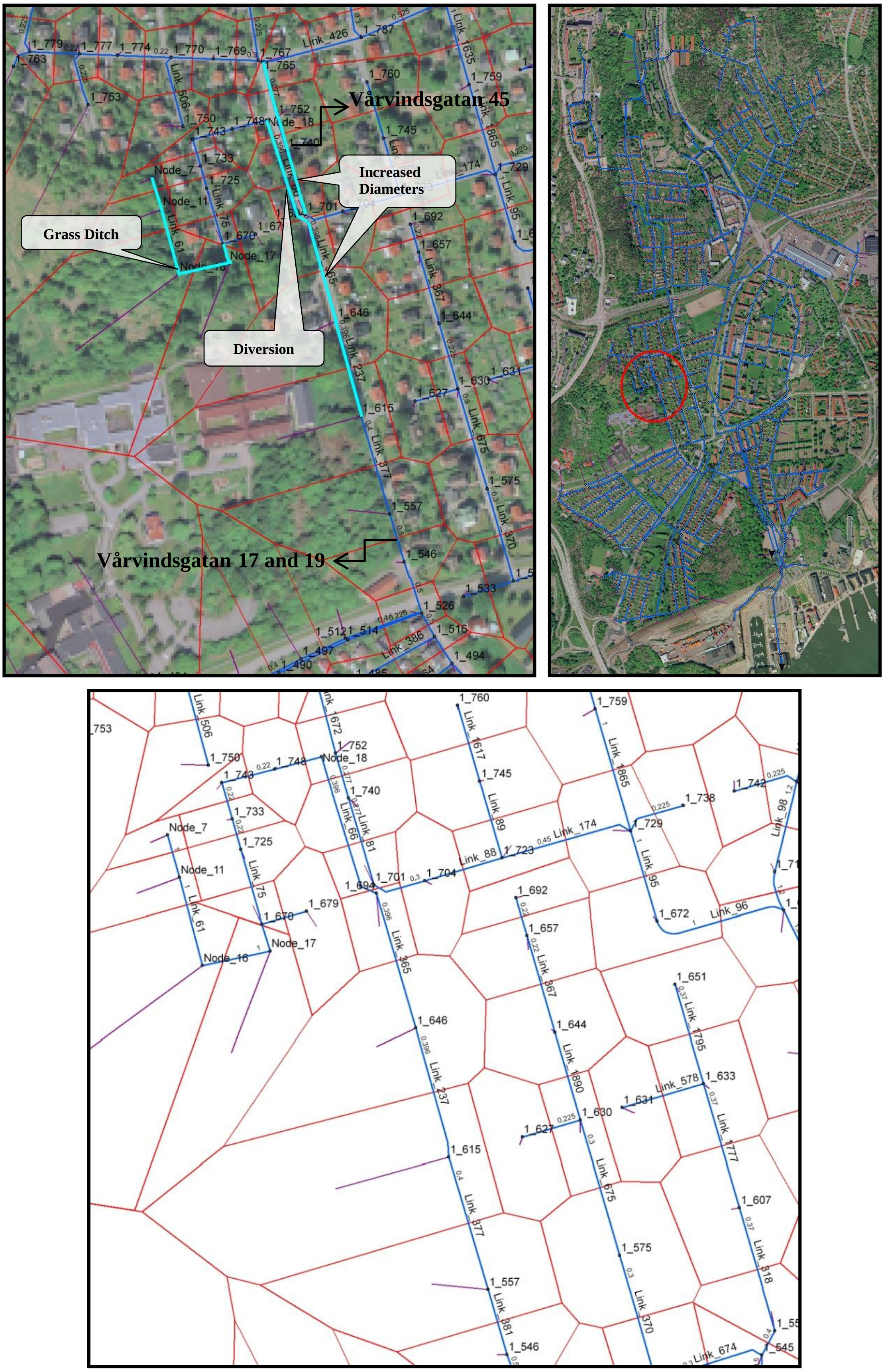
Link	Slope (%)	Maximun Discharge (m^3/s)	Diameter Colebrook (mm)	PP_Urib (Inner) (mm)
1507	0,83	27,7	175	220
2016	0,81	56,4	250	277
386	0,8	81,07	275	277

Visthusgatan



Appendix 9. Results of the network modifications in Vårvindsgatan

Situation map

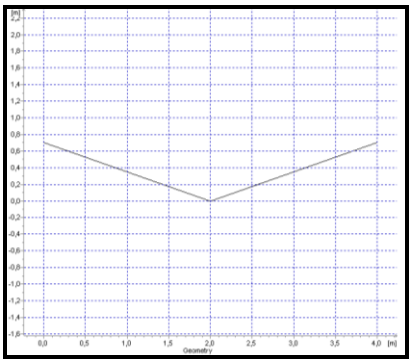


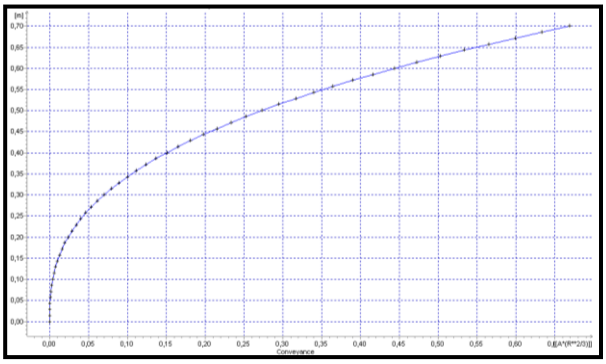
Diameters

Link	Slope (%)	PP_Urib (Inner) (mm)
1672	0.98	277
80	0.66	277
81	1.03	277
365	0.51	396
237	0.58	396

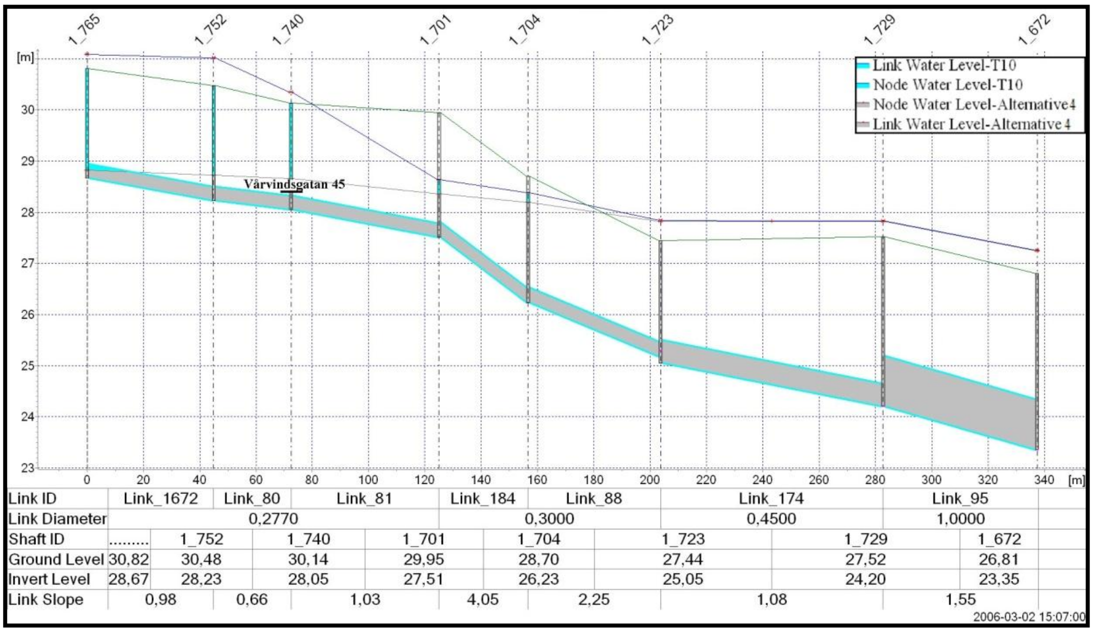
NOTE: In this case, the required diameters were calculated directly with the simulations, since the results obtained with Colebrook were not satisfactory

Grass ditch characteristics (n=0,04)

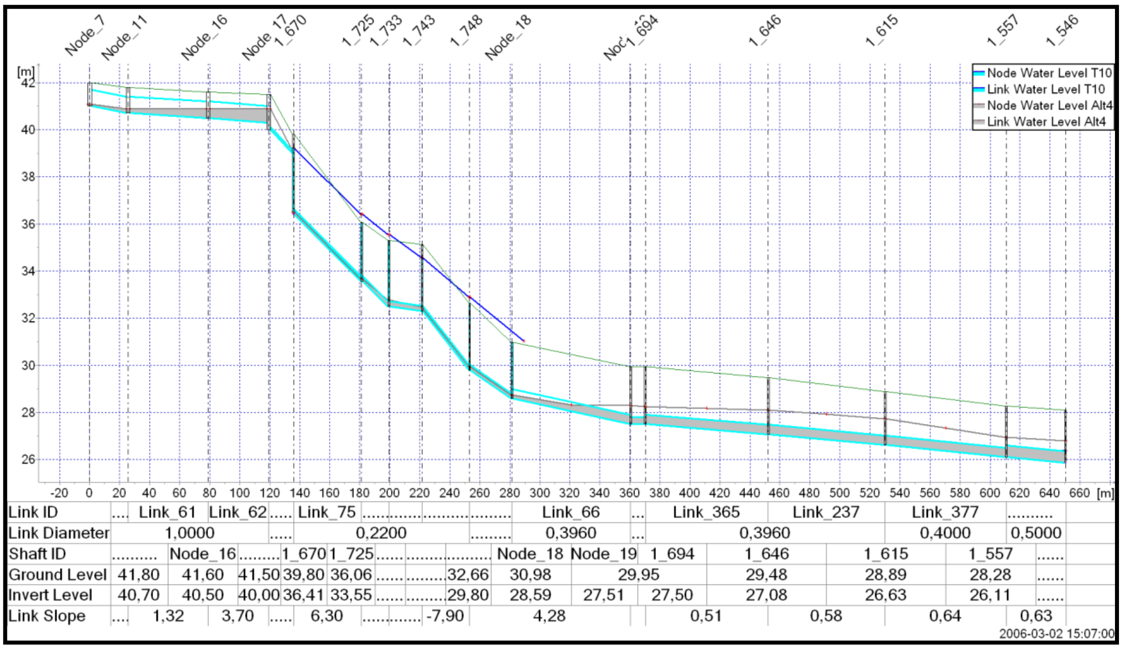




Vårvindsgatan 45

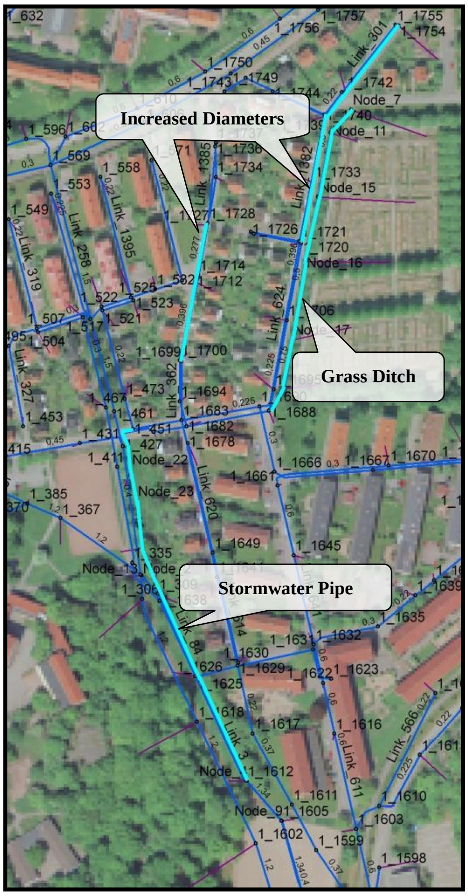


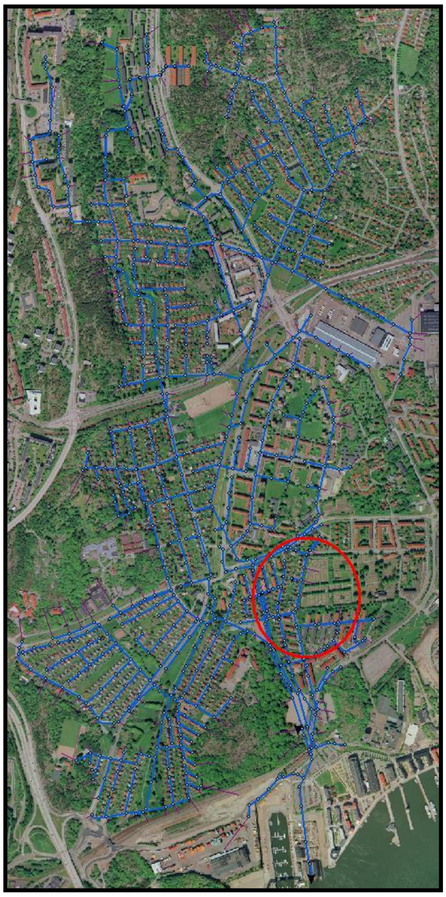
New diversion –Vårvindsgatan towards main line in Bräcke

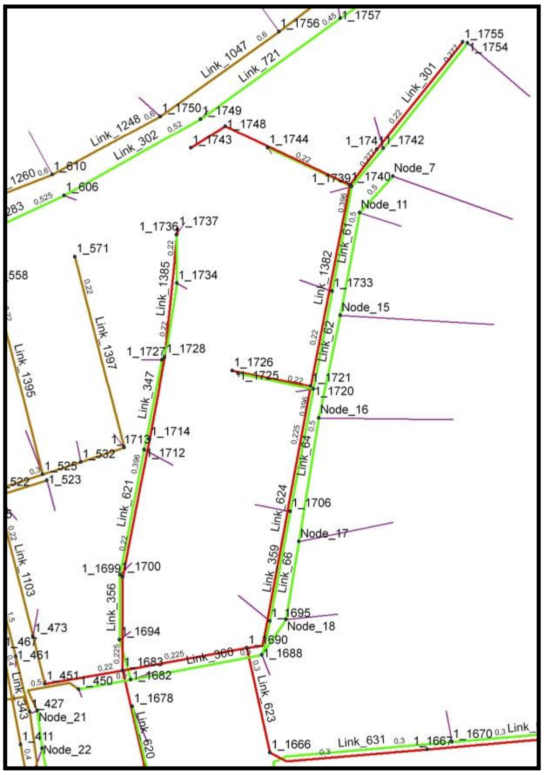


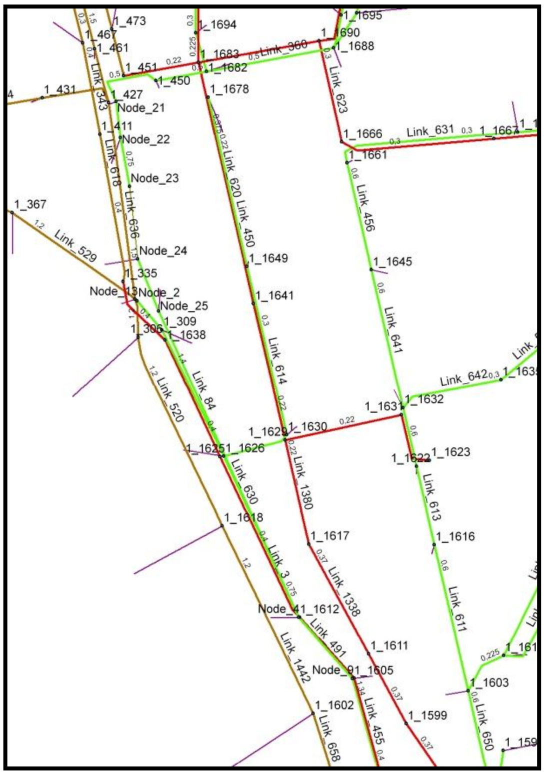
Appendix 10. Results of the network modifications in Pilegårdsgatan-Prebendegatan

Situation map

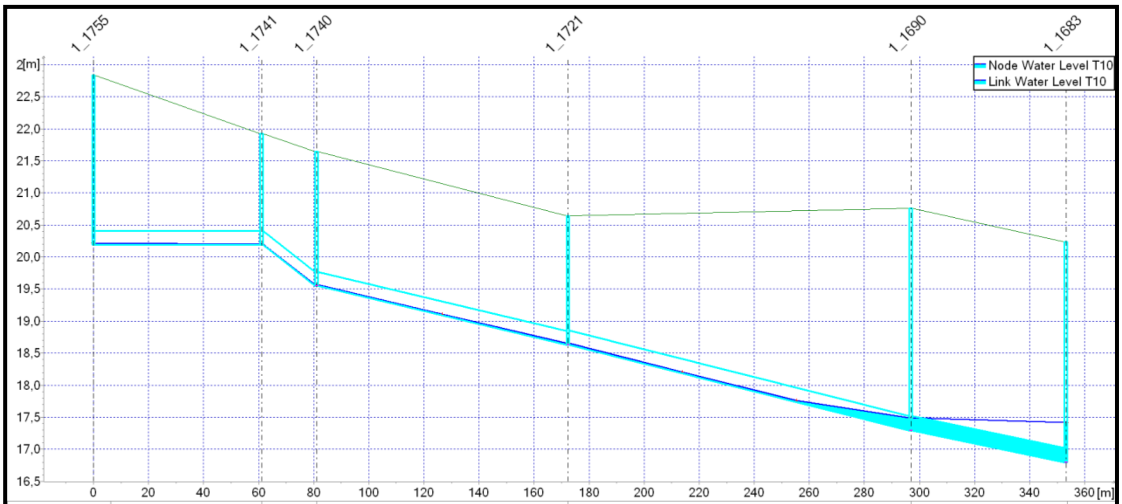








Wastewater pipe in Pilegårdsgatan



Link ID	Link_301	....	Link_1382	Link_624	Link_494	
Link Diameter	0,2200			0,2250		
Shaft ID	1_1755	1_1741	1_1740	1_1721	1_1690	1_1683
Ground Level	22,83	21,92	21,64	20,64	20,76	20,23
Invert Level	20,19		19,55	18,63	17,29	16,80
Link Slope	0,00	3,21	-1,01	1,08	0,87	

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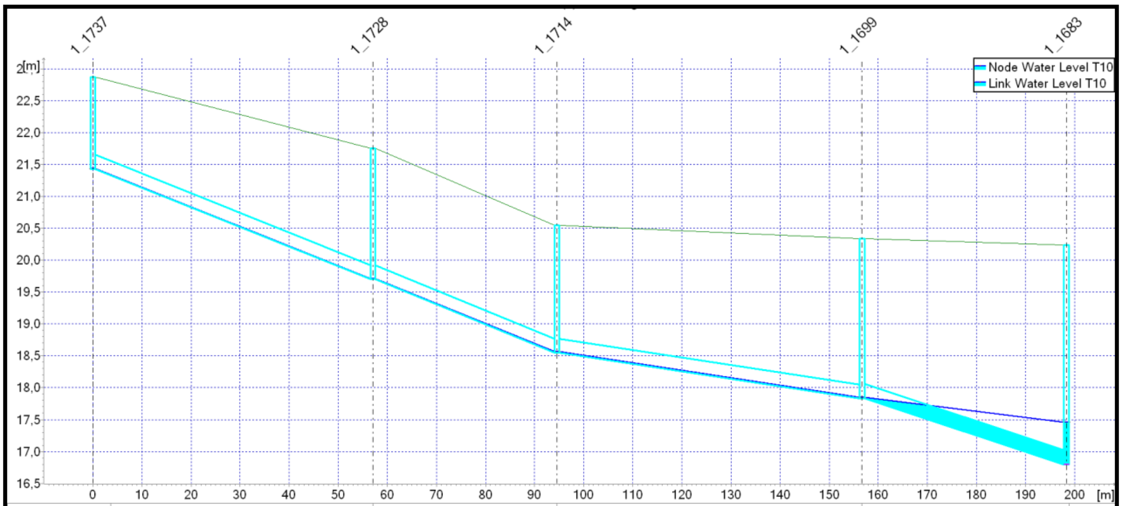
Wastewater pipe in Prebendegatan



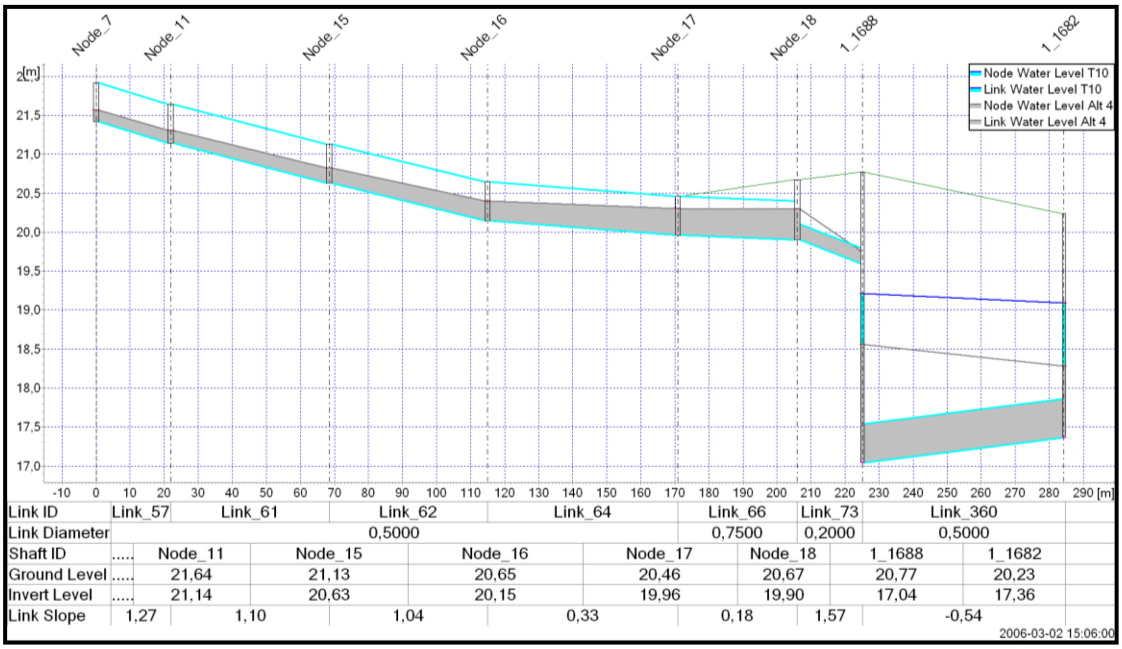
Link ID	Link_1385	Link_349	Link_1355	Link_362	
Link Diameter	0,2200			0,2250	
Shaft ID	1_1737	1_1728	1_1714	1_1699	1_1683
Ground Level	22,87	21,75	20,55	20,34	20,23
Invert Level	21,43	19,70	18,55	17,83	16,80
Link Slope	-3,03	3,07	1,16	2,47	

2006-03-02 15:03:00

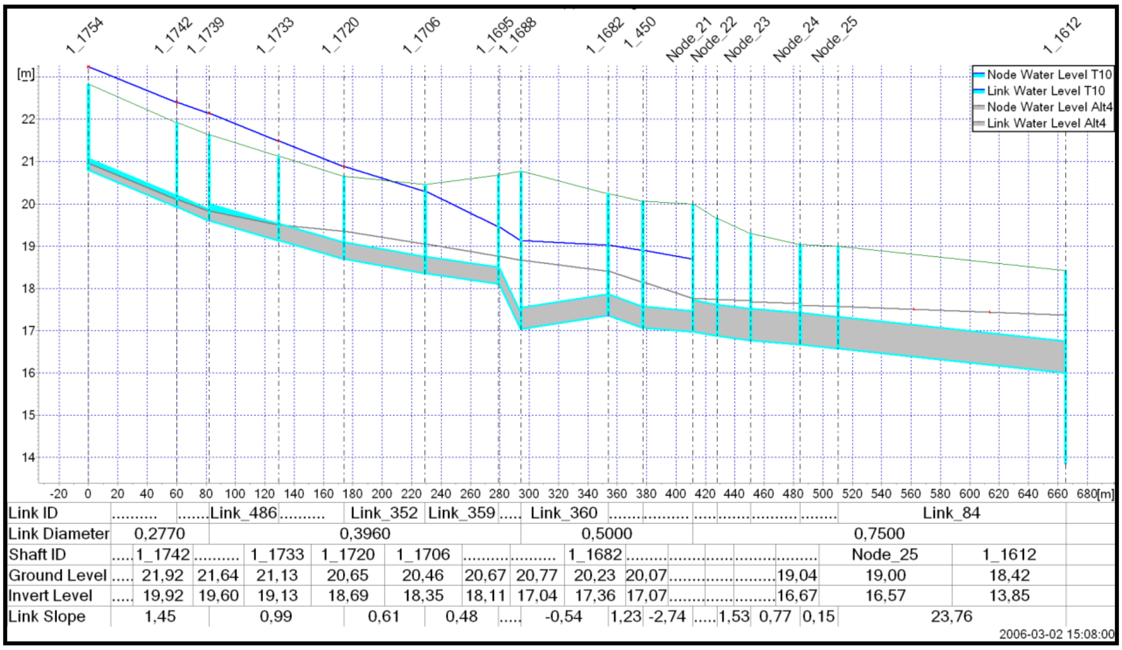
Diameters

Link	Slope (%)	Maximun Discharge (m^3/s)	Diameter Colebrook (mm)	PP_Urib (Inner)(mm)
552	1,45	0,072	240	277
719	1,45	0,11	260	277
486	0,99	0,14	350	396
353	0,99	0,15	350	396
347	3,04	0,087	230	277
621	0,57	0,11	350	396

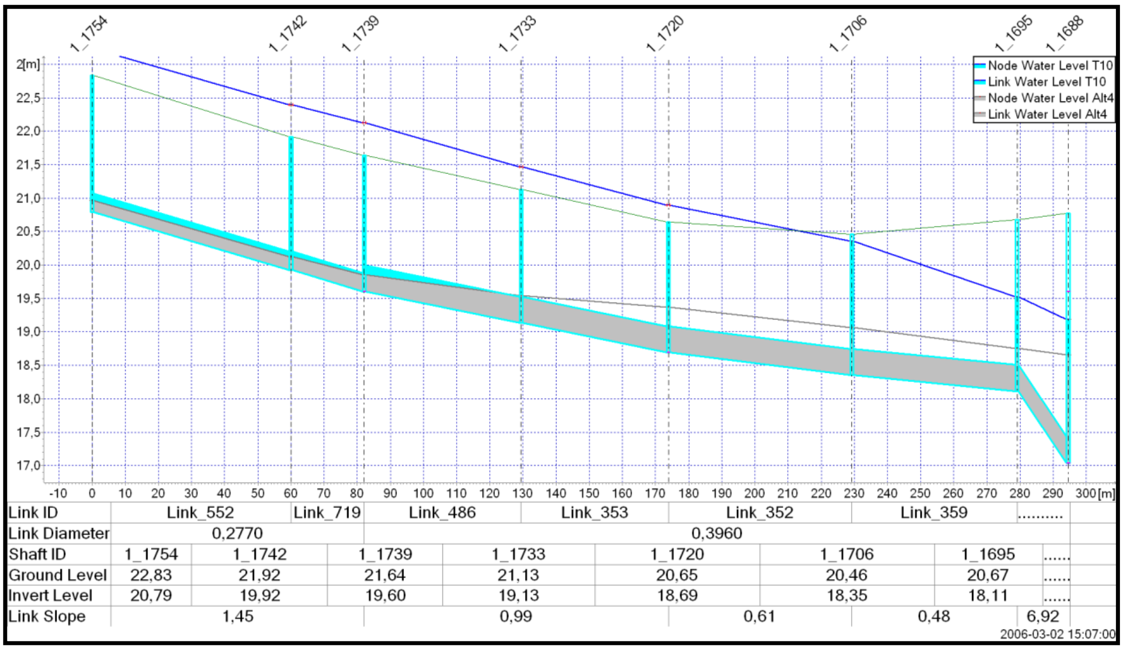
Grass ditch



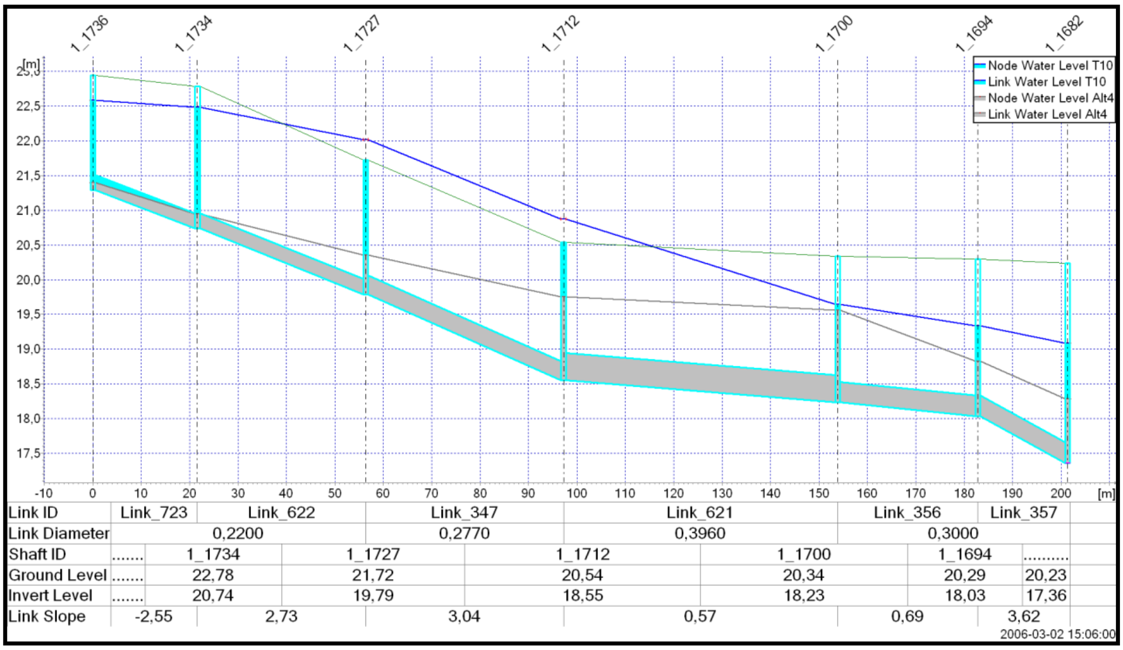
New stormwater pipe



Pilegårdsgatan



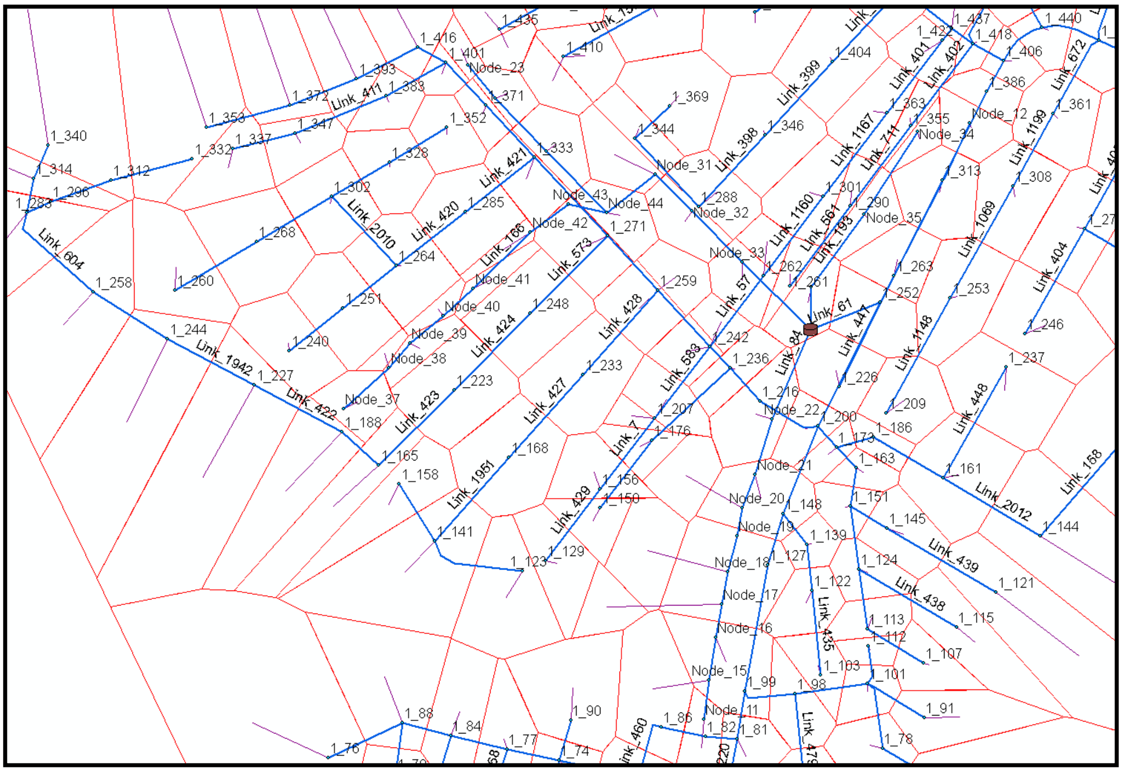
Prebendegatan



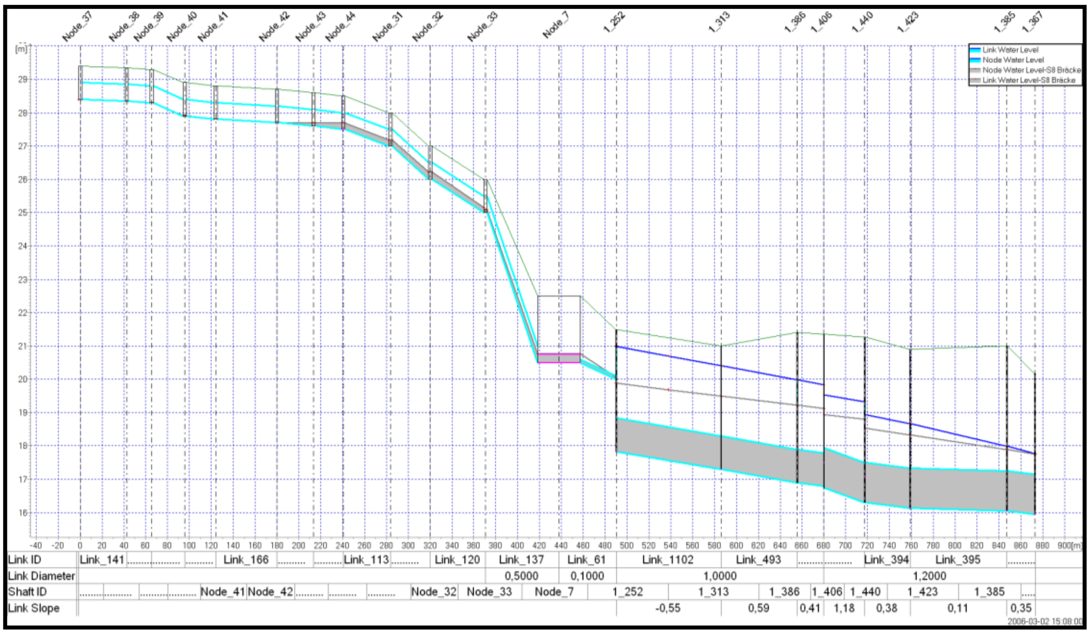
Appendix 11. Results of the network modifications with sustainable drainage systems

S7- Bräcke. Situation map

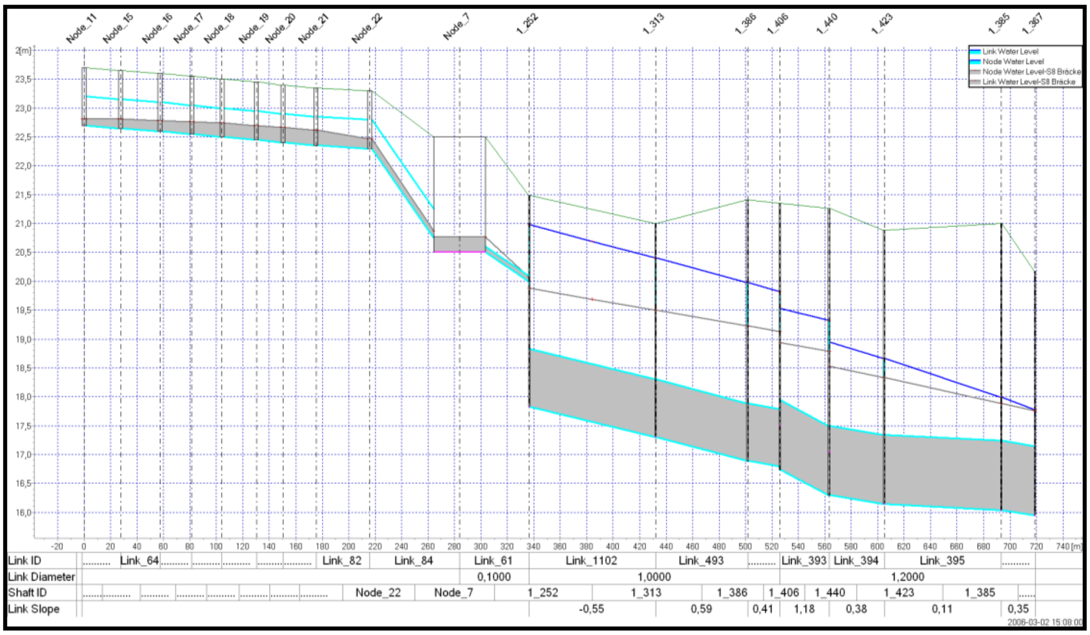
Dentention Pond and grass ditch branches



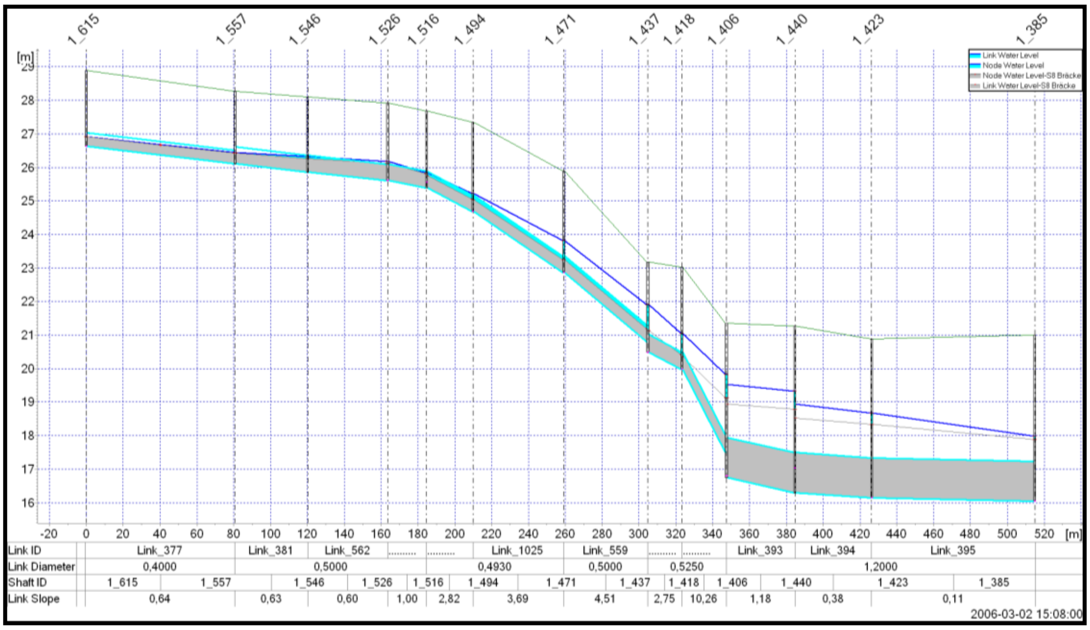
Grass ditch (north) and Bräcke line



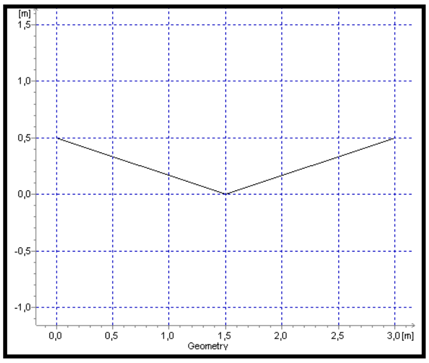
Grass ditch (south) and Bräcke line

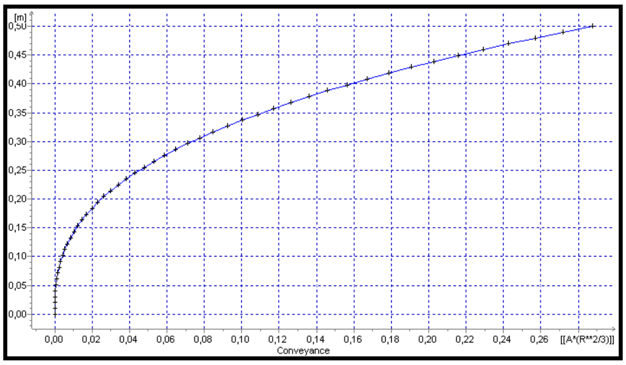


Influence of grass ditches in Vårvindsgatan line

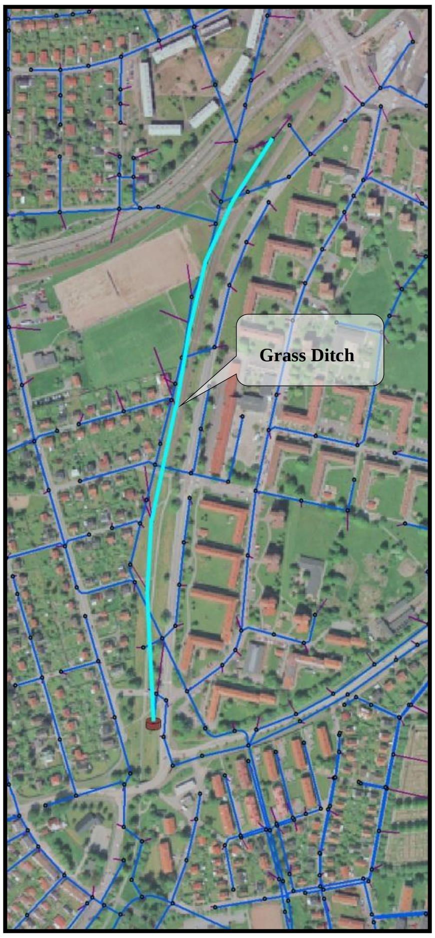


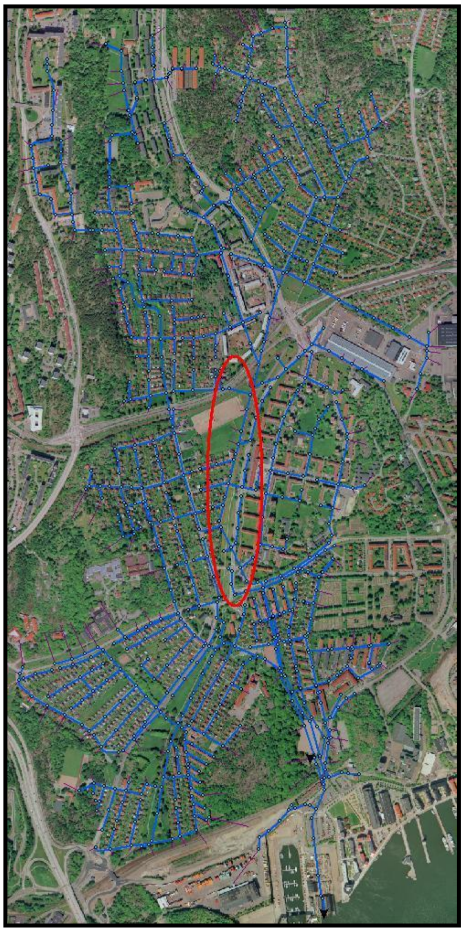
Grass ditch characteristics (n=0,04)

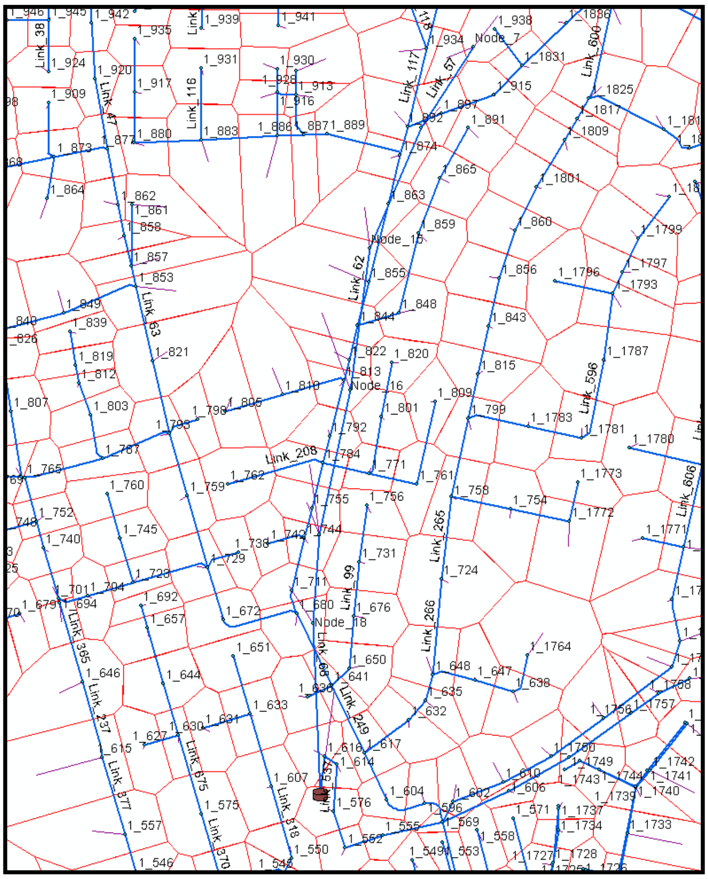




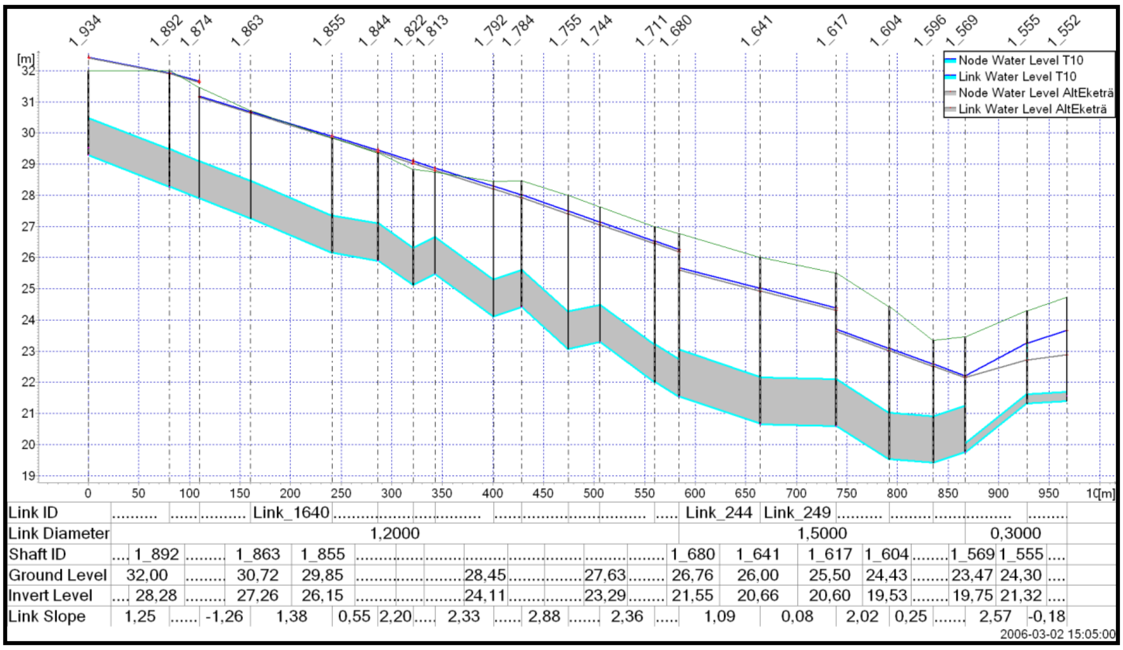
S8- Eketrägatan. Situation map



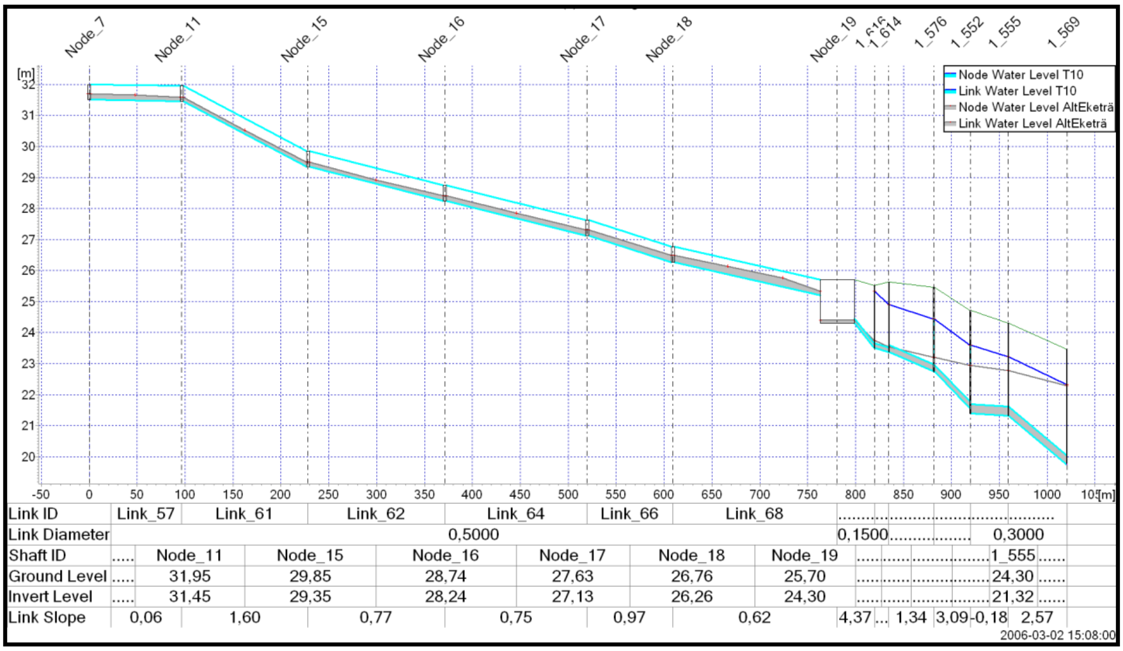




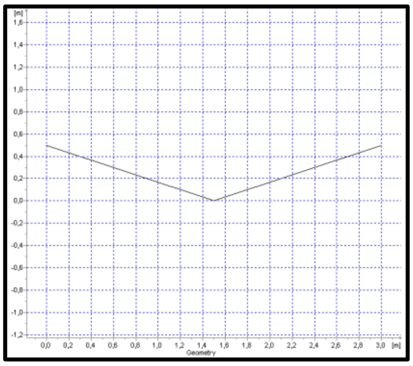
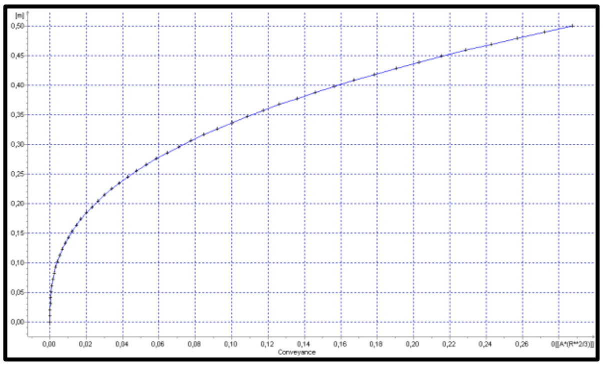
S8- Eketrägatan. Influence of the grass ditch in the main line



S8- Eketrägatan. Grass ditch performance

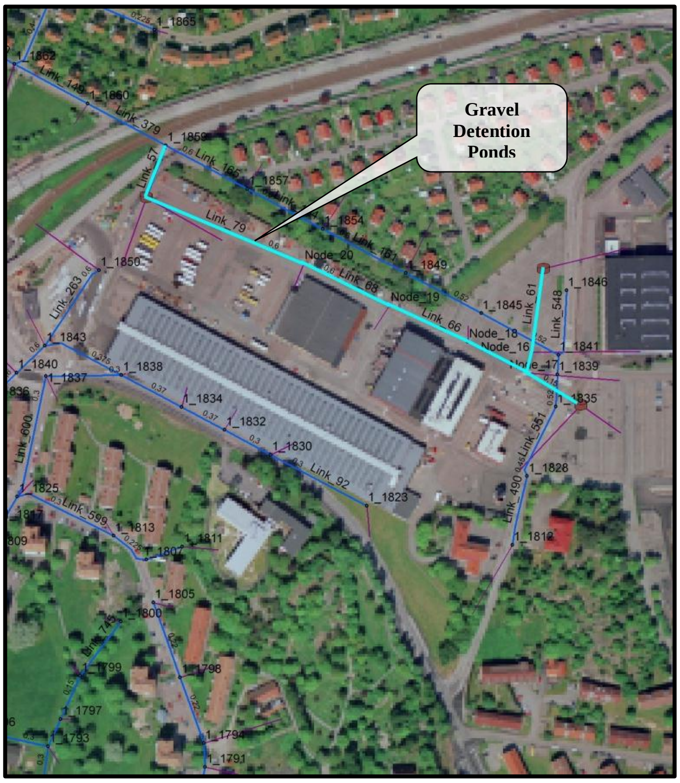


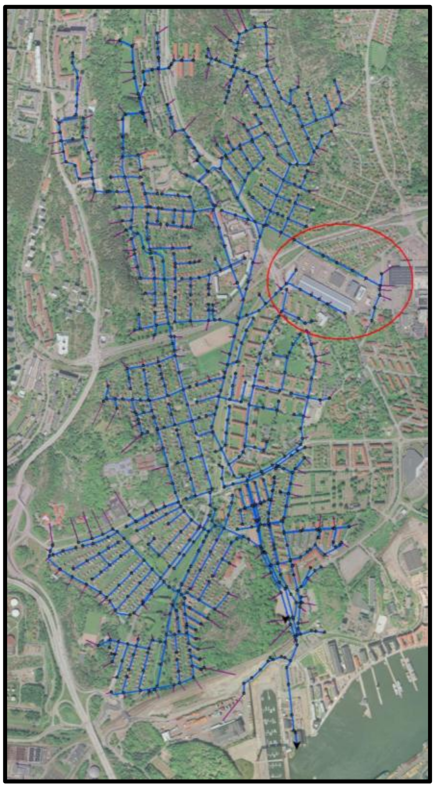
Grass ditch characteristics (Manning coefficient n=0,04)

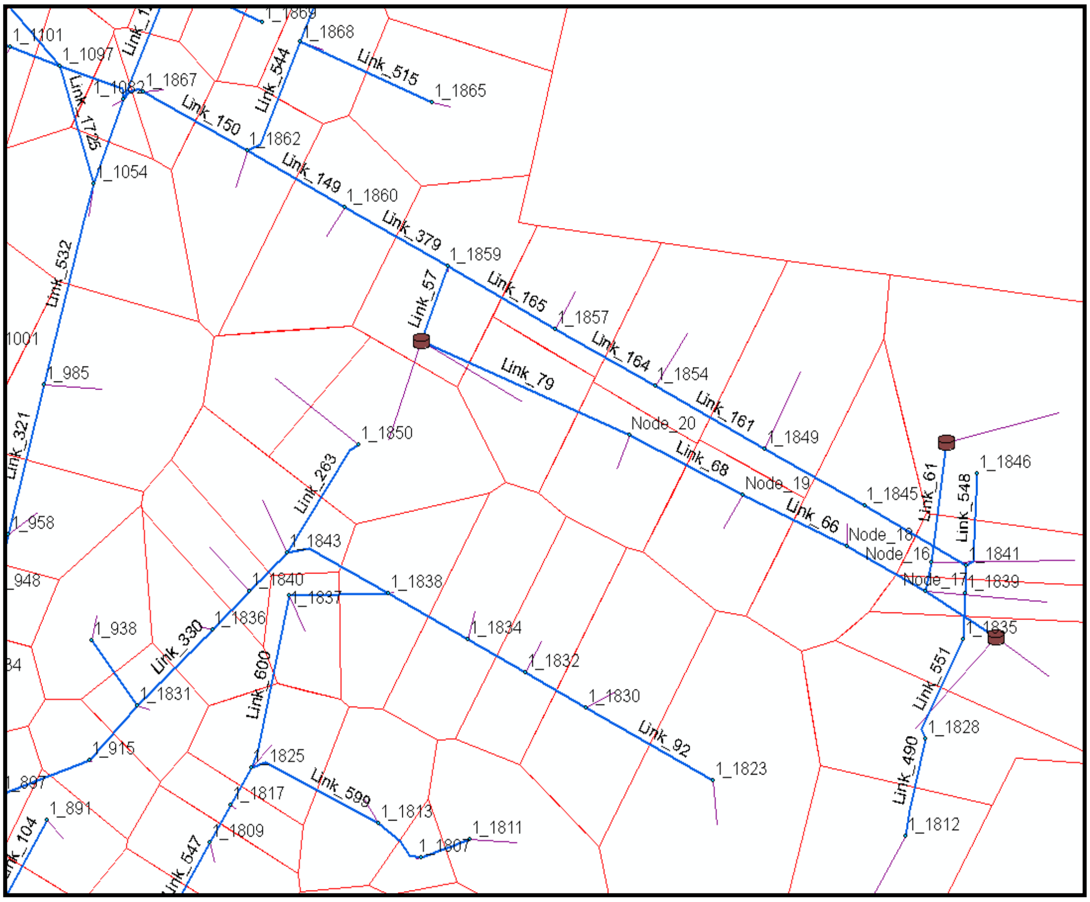



S9- Volvo

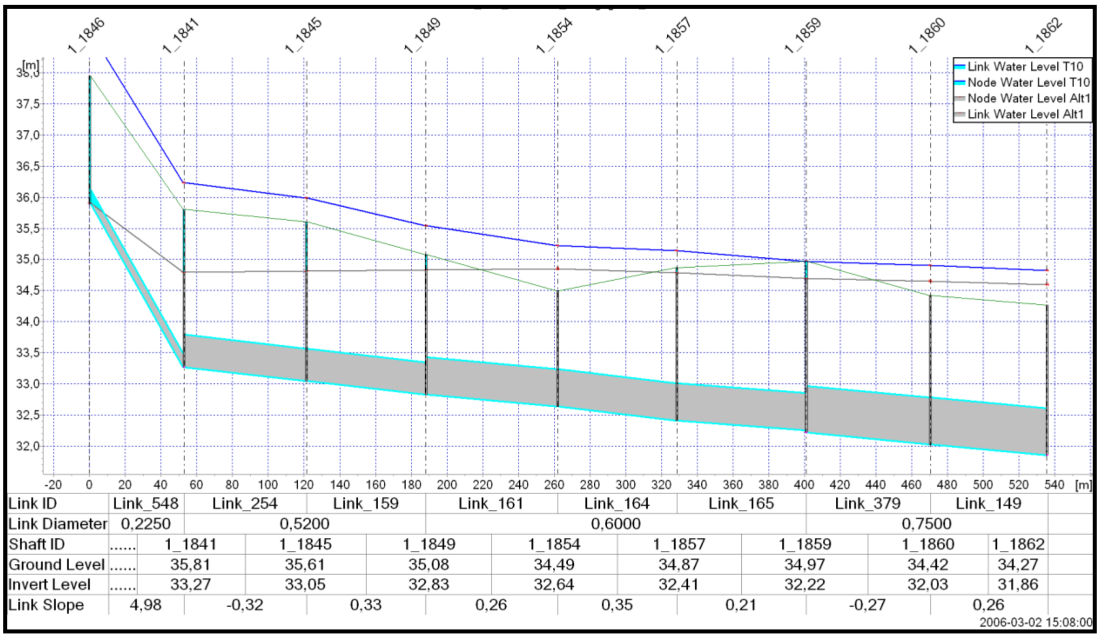
Situation map



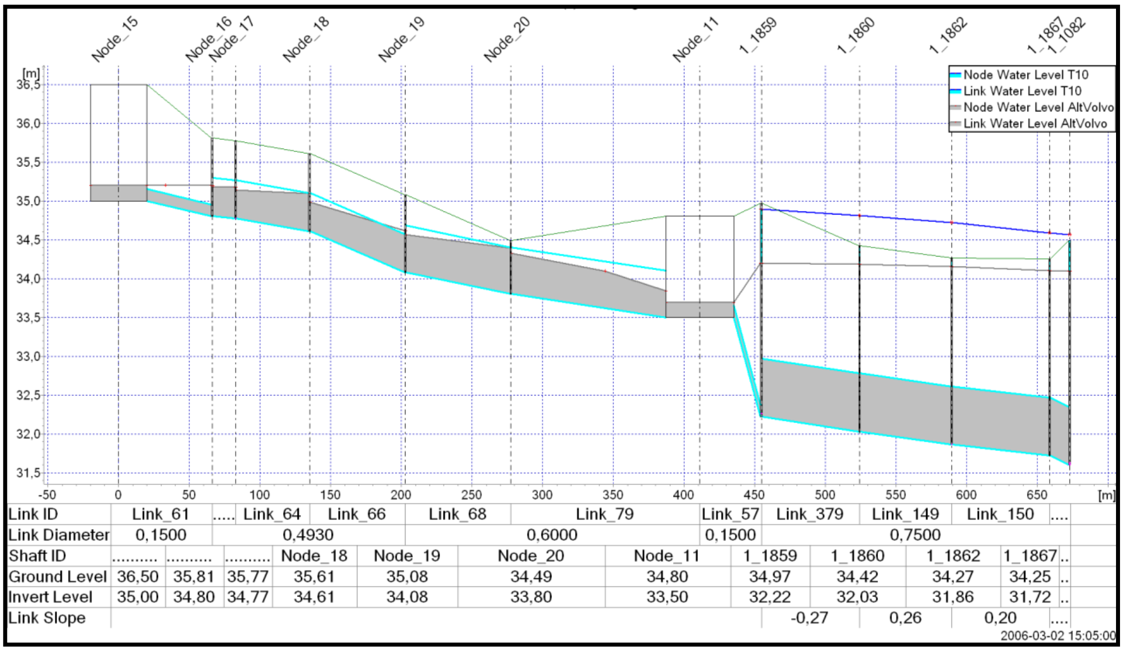




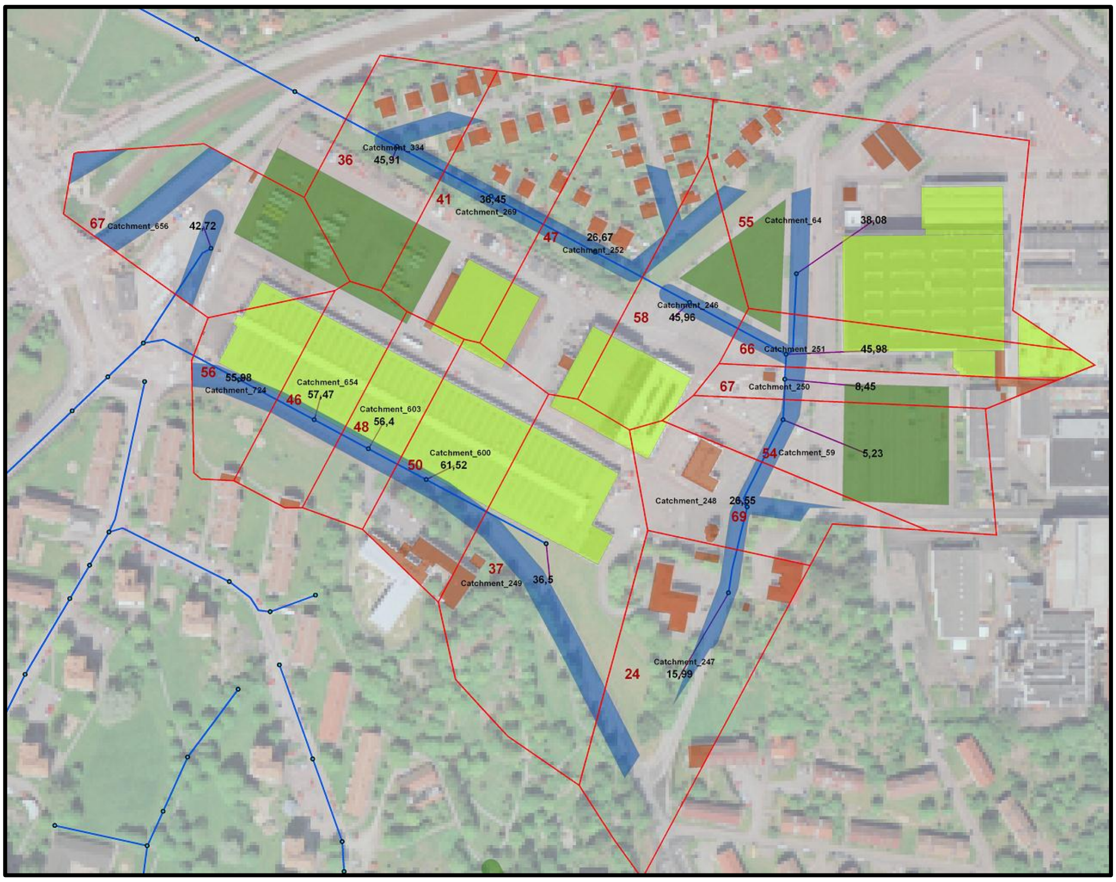
Northern pipeline at Volvo



Gravel tanks



Imperviousness values

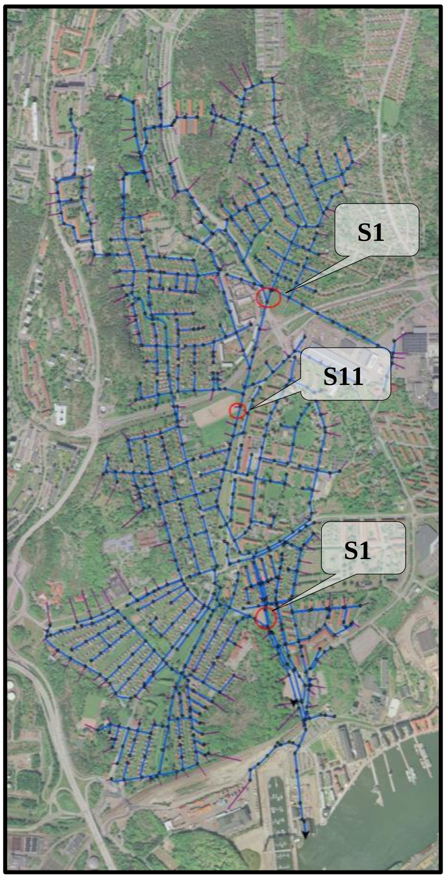


Catchments	Total Area	Imperviousness	Roof Area	Roof Imp.	Road Area	Road Imp.	Green Roof Area	Green Roof Imp.	Green Parking Area	Green Parking Imp.	Green Area	Grass Imp.	Street area	Street Imp.	Modified Imperviousness
59	10760	5%	7	0,9	521	0,8	10	0,5	4030	0,1	0	0,1	6191	0,8	53%
64	24040	38%	1010	0,9	1086	0,8	7880	0,5	1610	0,1	3897	0,1	8557	0,8	54%
269	12000	36%	1086	0,9	864	0,8	1400	0,5	1190	0,1	5050	0,1	2410	0,8	41%
246	8690	46%	62	0,9	1314	0,8	2430	0,5	900	0,1	807	0,1	3177	0,8	58%
247	15490	12%	1352	0,9	1599	0,8	0	0,5	0	0,1	12539	0,1	0	0,8	24%
248	7780	27%	532	0,9	1113	0,8	320	0,5	20	0,1	1136	0,1	4658	0,8	69%
249	19780	36%	359	0,9	2073	0,8	4450	0,5	0	0,1	10417	0,1	2482	0,8	36%
250	3980	8%	283	0,9	246	0,8	10	0,5	780	0,1	0	0,1	2662	0,8	67%
251	5930	46%	154	0,9	816	0,8	2410	0,5	170	0,1	0	0,1	2379	0,8	66%
252	13730	27%	1285	0,9	1392	0,8	1450	0,5	0	0,1	5977	0,1	3626	0,8	47%
334	9330	46%	632	0,9	1086	0,8	0	0,5	2770	0,1	3182	0,1	1660	0,8	36%
600	8970	62%	598	0,9	951	0,8	3800	0,5	0	0,1	2314	0,1	1307	0,8	50%
603	5420	56%	19	0,9	595	0,8	2380	0,5	20	0,1	1471	0,1	935	0,8	47%
724	5820	56%	36	0,9	833	0,8	2270	0,5	0	0,1	1045	0,1	1635	0,8	565%
654	6610	57%	68	0,9	687	0,8	2630	0,5	370	0,1	1764	0,1	1090	0,8	45%
656	12510	43%	22	0,9	2531	0,8	230	0,5	2310	0,1	0	0,1	7417	0,8	66%

Note: Areas in m^2

Appendix 12. Results of the network modifications with reservoirs

Situation map



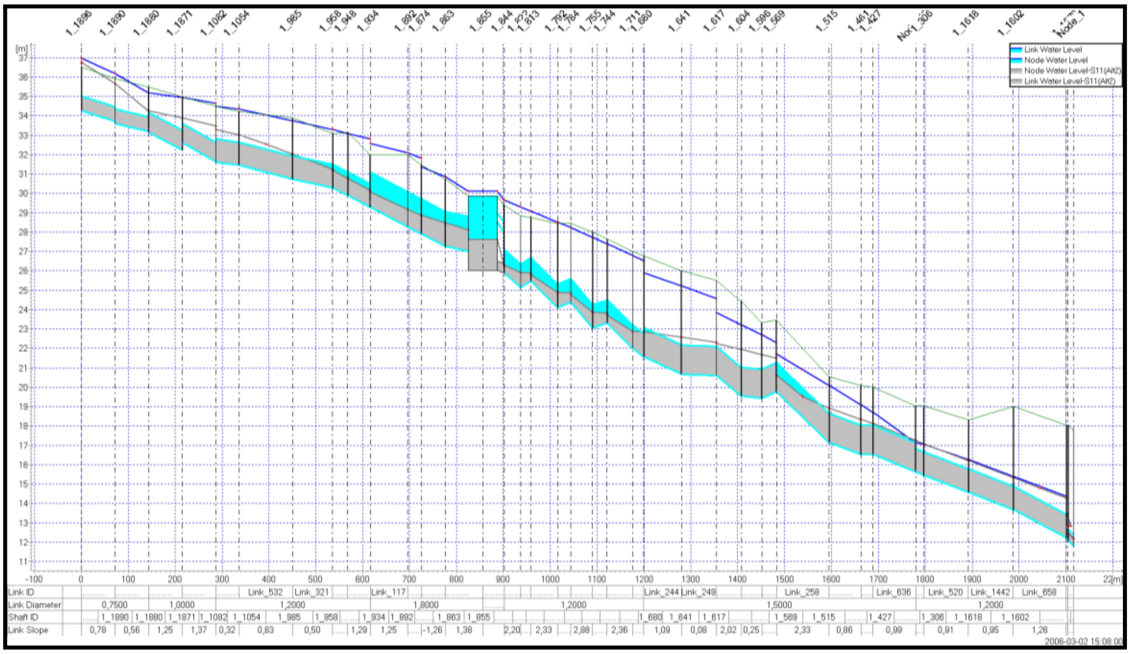
Continuous rain (T5)		T5		S10-Alt1		S10-Alt2		S10-Alt3		S11		S12	
Pipe filling	<0	252	33%	327	39%	336	40%	277	33%	310	37%	294	35%
	>0	587	67%	512	61%	503	60%	562	67%	529	63%	545	65%

Block rain (BlockT5)		BlockT5		S10-Alt1		S10-Alt2		S10-Alt3		S11		S12	
Pipe filling	<0	319	38%	369	44%	376	45%	277	33%	369	44%	336	40%
	>0	520	62%	470	56%	463	55%	562	67%	470	56%	503	60%

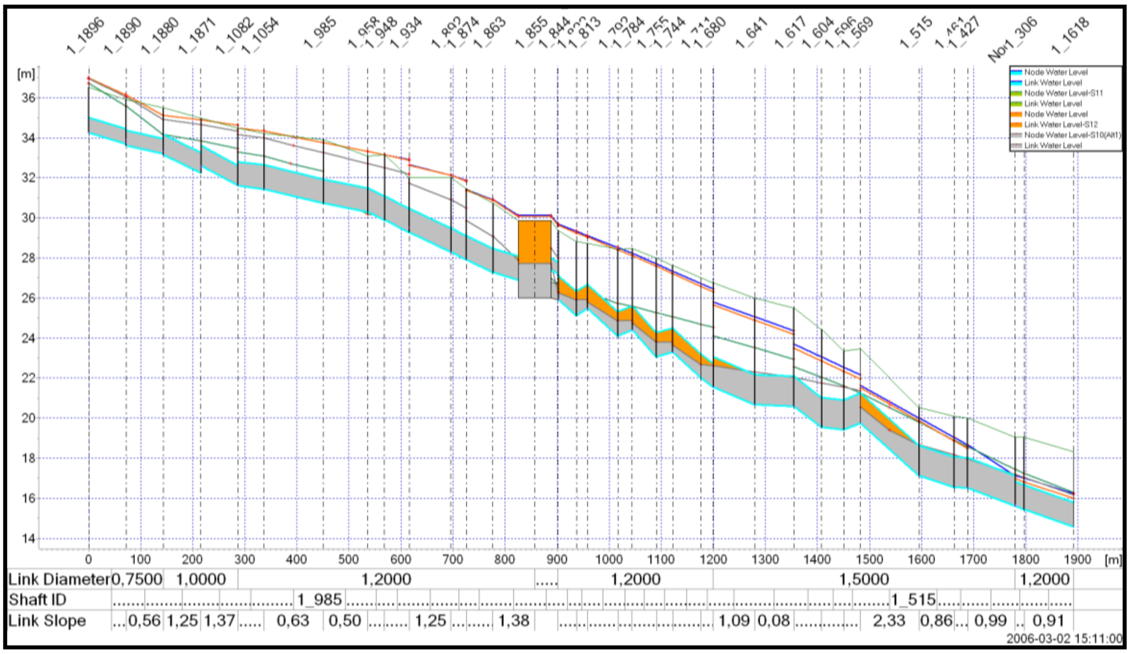
Continuous rain (T10)		T10		S10-Alt1		S10-Alt2		S10-Alt3		S11		S12	
Nodes flooding	< -1,5 m	371	44%	439	53%	454	54%	433	52%	416	50%	392	47%
	-1,5- 0m	276	33%	249	30%	258	31%	251	30%	286	34%	261	31%
	>0	189	23%	148	18%	124	15%	152	18%	134	16%	183	22%
Pipe flooding	< -1,5 m	302	36%	376	45%	390	46%	365	44%	343	41%	323	38%
	-1,5- 0m	276	33%	259	31%	275	33%	265	32%	309	37%	263	31%
	>0	261	31%	463	24%	174	21%	209	25%	187	22%	253	30%

Block rain (BlockT10)		BlockT10		S10-Alt1		S10-Alt2		S10-Alt3		S11		S12	
Nodes flooding	< -1,5 m	426	51%	504	60%	519	62%	504	60%	500	60%	456	55%
	-1,5- 0m	242	29%	198	24%	208	25%	198	24%	221	40v	231	27%
	>0	168	19%	134	16%	109	13%	134	16%	115	14%	149	18%
Pipe flooding	< -1,5 m	360	43%	444	53%	459	55%	445	53%	503	60%	389	46%
	-1,5- 0m	252	30%	210	25%	230	27%	209	25%	221	26%	241	29%
	>0	227	27%	185	22%	150	18%	185	22%	115	14%	209	25%

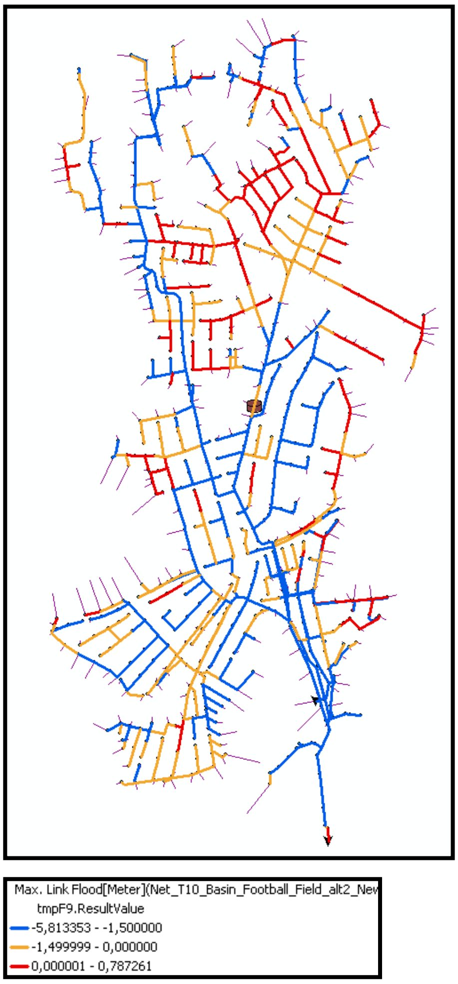
S10-Alt2. Main line

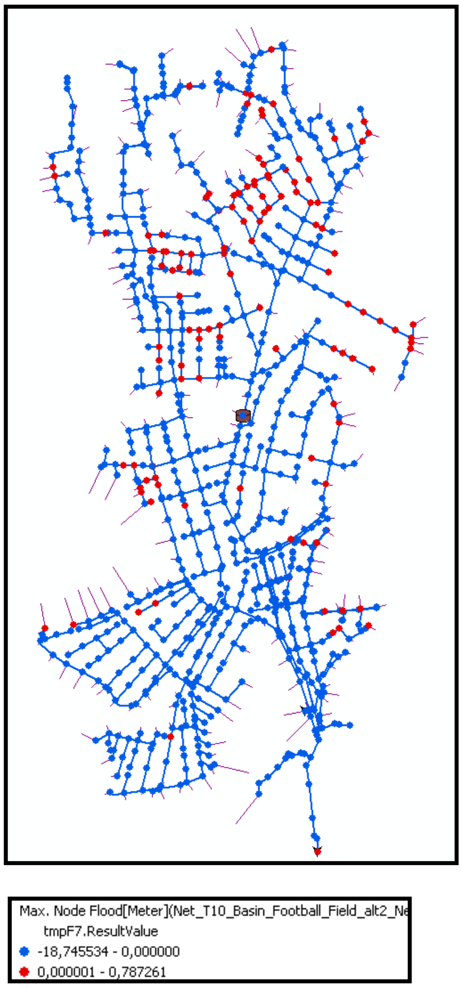


Comparison S10-Alt1, S11 and S12



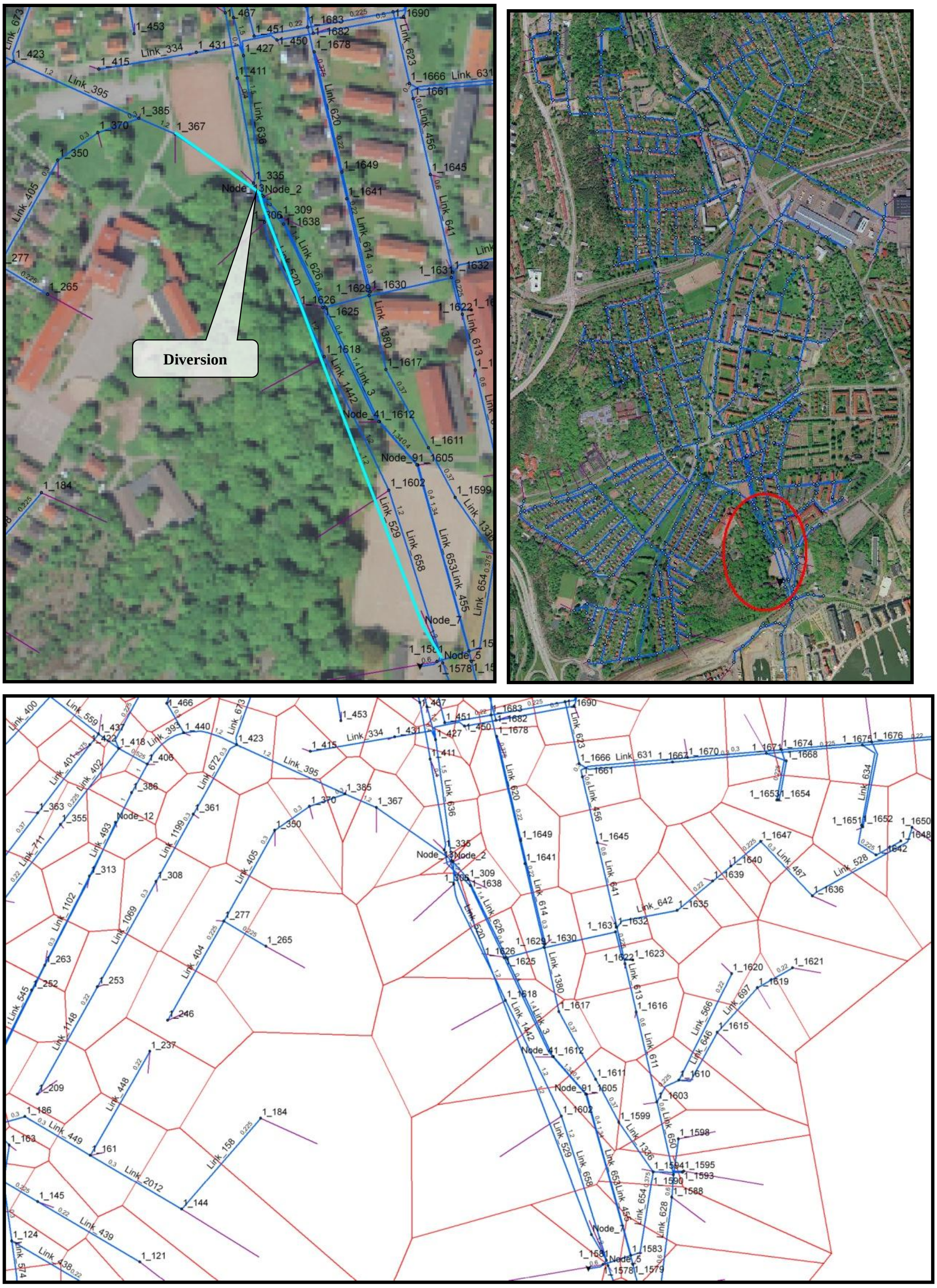
S10-Alt2. Link food and node flood



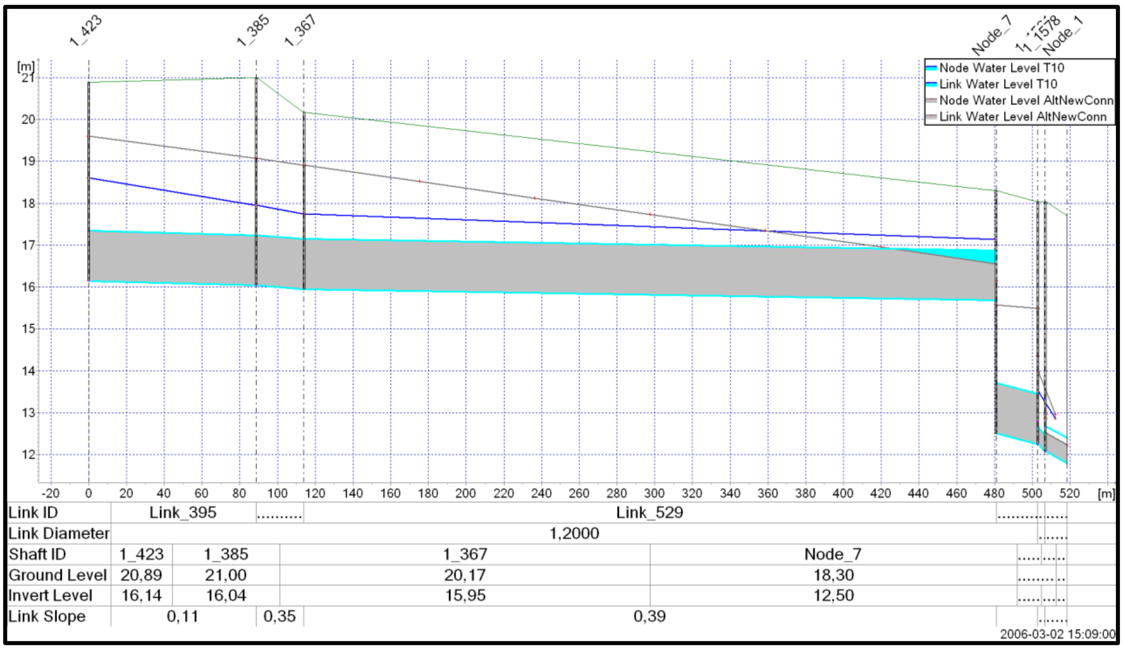


Appendix 13. Results of the network modifications with diversion of pipelines

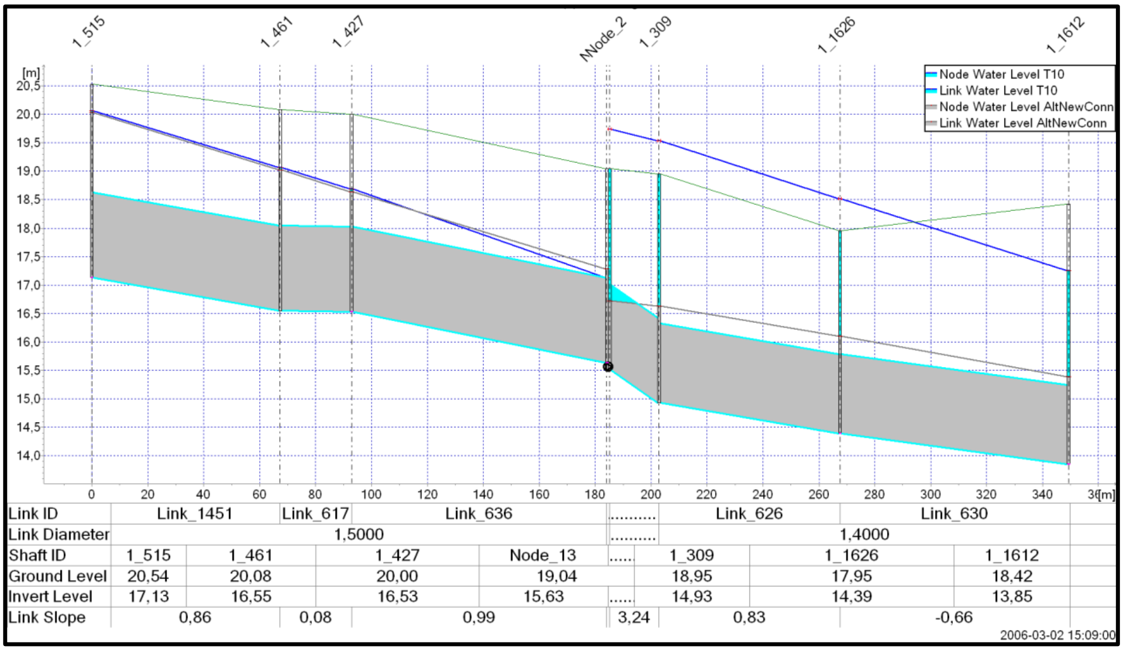
Situation map



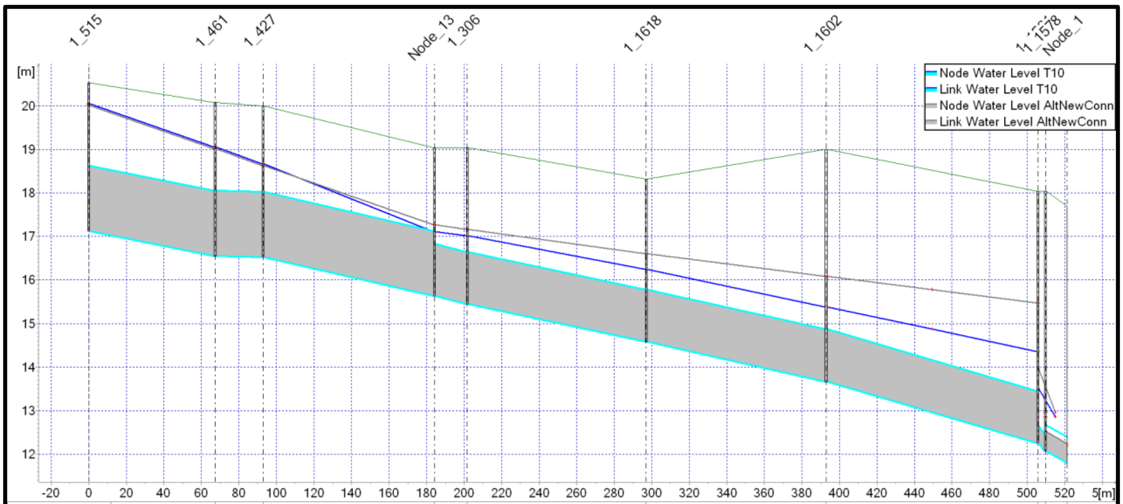
Bräcke line until outlet (T10)



Main line-CSO-Stormwater pipe



Main line-CSO-Combined pipe



Link ID	Link_1451	....	Link_636	....	Link_520	Link_1442	Link_658	......
Link Diameter	1,5000	1,2000	......					
Shaft ID	....	1_461	1_427	Node_13	1_306	1_1618	1_1602	1_1581	.....
Ground Level	....	20,08	20,00	19,04	18,32	19,00	18,04	..	
Invert Level	....	16,55	16,53	15,63	15,45	14,58	13,67	12,25	.....
Link Slope	0,86	0,08	0,99	1,03	0,91	0,95	1,26	......	

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