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Water and Environmental Engineering
Department of Chemical Engineering

Hydraulic Modeling of Open Stormwater System in Augustenborg, Sweden

Master's Thesis by

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Picture on front page:

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1. A creek through a public park in Augustenborg Eco-City, Malmö, Sweden.

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List of Abbreviations

DHI	DHI Water – Environment – Health
CRS	Cross-section (of an open channel in MOUSE engine)
MOUSE	Model for Urban Sewers
SWMM	Storm Water Management Model
UHM	Unit Hydrograph Method
RDI	Rainfall Dependent Infiltration
ESRI	Environmental Systems Research Institute

Summary

The open stormwater systems have became an increasingly acceptable solution to handle the stormwater in urban areas, since these solutions are more sustainable help to reduce and retard the flow. Therefore, it is important to be able to model such systems in order to evaluate their performance and justify such solutions compared to the traditional practices.

This study aims toward modeling open stormwater systems using a modeling software, MIKE URBAN which is widely used for modeling water distribution systems and collection systems in urban areas. For this purpose a study area is considered in southern Sweden in the city of Malmö called the Augustenborg Eco-City. Augustenborg is an urban residential area is covering a total area of 32 hectares, consisting mostly of residential apartment blocks. In most parts of Augustenborg the stormwater is handled by an open stormwater system. The system consists of various open canals, swales, green-roofs, ponds, detention areas and green spaces.

The open stormwater system is modeled in MIKE URBAN using the available information from the construction companies and the information in published literature. The model uses the MOUSE computational engine consisting of a rainfall-runoff model and a hydraulic network model. To calibrate the model, the discharge in two locations in the system was measured over a period of about 3 months in year 2009.

Three simulation scenarios are considered in this study. The first scenario simulates the current weather conditions in the open stormwater system using measured rainfall data in years 2007 and 2008. The second scenario represents a comparison between the open system and a comparable hypothetical conventional storm sewer system, where the current weather conditions are simulated in both models and the comparison is based on the discharge hydrograph in specific locations in the system. Similar to the second scenario, the third scenario is also a comparison between both systems, but extreme rainfall conditions are simulated using a synthetic storm event and the comparison is based on node flooding.

The results showed that the open stormwater system in Augustenborg has a lag time of 15 minutes up to one hour depending on the system configuration. This was longer by 5 minutes to 40 minutes than that simulated in the conventional system. The simulated discharge rate in the open system was about 40%-50% of that simulated in the conventional system.

Simulating a 50 years storm of duration 20 min showed a more severe flooding in the conventional system, while the open system had a larger capacity to handle such storm.

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Aza Shukri

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Contents

1. Introduction

1.1. Background

The expansion of urban development results in an increase in the stormwater runoff, which means larger stormwater flow volumes to be handled by the sewer network. The problem is severer when the new development is connected to an existing network that is not designed for that increase in flow volume, especially in combined sewer systems, where many problems are expected, such as basement flooding in buildings in extreme weather conditions.

This increase of stormwater flow into sewer systems can be tackled in various ways. One way of achieving that in a sustainable manner is to develop an open stormwater system in the urban areas, to allow early (on-site) handling of a portion of the stormwater before reaching the sewer system.

An open stormwater system can consist of any combination of facilities that contribute in reducing the flow to the sewer system, through infiltration, storage, detention and slow transport of the stormwater (Stahre, 2006). Examples of these facilities are ponds of various types, vegetated buildings roofs, canals of different types (both vegetated and lined), constructed wetlands, existing natural landscape. (Stahre, 2008)

These solutions have proved useful in different projects in Sweden (Stahre, 2008); however the evaluation of the hydraulic performance of these solutions is a complicated task because of the complex nature of the facilities implemented in these systems. Needless to say that the hydraulic performance is of importance to evaluate existing systems and justify the future suggested projects.

1.2. Objectives

The aim of the study is to describe the hydraulic performance of the open stormwater solutions implemented in a study area, Augustenborg Eco-City. This is achieved by modeling the system using the computer modeling system MIKE URBAN, and evaluate the impact of implementing these solutions in comparison to the conventional sewer system.

1.3. Methods

Literature study: The first stage of the study is a literature review. Here, the open stormwater solutions are considered, and information are collected to describe the hydraulic properties of these solutions, focusing mainly on those implemented in the study area.

Data collection: Available information is collected, such as site plans, construction details of the solutions, and topography information. The discharge from selected critical points in the system was measured along with the corresponding rainfall.

Modeling and simulation: Using the knowledge acquired from the two preceding stages, a computer model of the stormwater system in Augustenborg and an equivalent conventional pipe network are constructed using the MOUSE engine in the modeling software, MIKE URBAN. The current conditions and extreme weather conditions are simulated and the evaluation is based on the discharge rate and lag time (response time).

1.4. Limitations

1. The elevation data for some parts of the open stormwater system were not available; these represent the bottom and ground elevations for each node in the system. The elevations for those parts were interpolated manually from neighboring points of known elevations.
2. The lack of significant observed flow data prohibited a conclusive validation of the model.

2. Study Area Augustenborg Eco-City

Augustenborg is an urban area in the city of Malmö in southern Sweden, see Figure 2-1. It consists mainly apartment blocks that were originally built in the 1950's, covering an area about 32 hectares. Among the other buildings and services in Augustenborg is a school, a public parking area, a workshop, and a house for elderly people. The settlement is inhabited by around 3000 people (eco-guide.net, 2006).

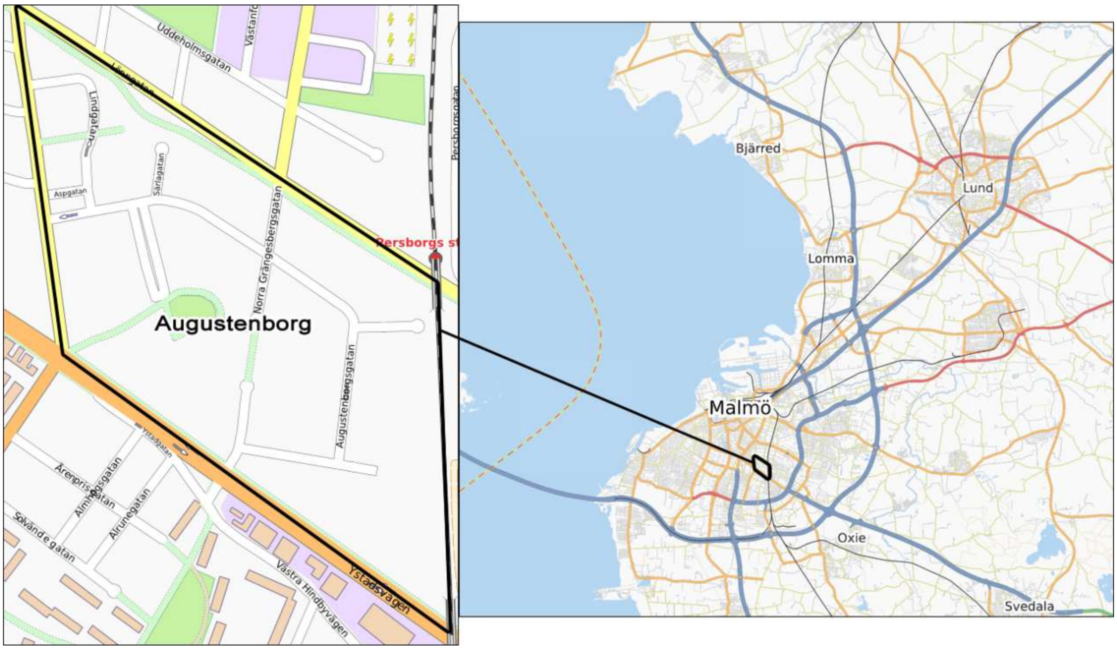


Figure 2-1: A map over Malmö showing Augustenborg Eco-City (OpenStreetMap, 2010).

During the four decades following the construction, the settlement deteriorated and suffered from various problems, among them was the basements flooding during heavy rainfall periods, since the wastewater was handled in a combined sewer system (Stahre, 2008).

Therefore, starting at 1998, the owner company MKB started working on the renovation of the settlement in an ecological manner to restore the social and economical status of the settlement, and thus the Augustenborg Eco-City was developed. The renovations were completed by 2005 (Stahre, 2008).

Many ecological solutions were introduced into the area, and the one of interest here is the stormwater handling system, were the combined sewer system in most parts of Augustenborg was replaced by an open stormwater system to handle the stormwater before entering the existing sewer system in order to solve the basements flooding problem.

The main open stormwater facilities built in Augustenborg can be summarized into three categories (Stahre, 2008):

1. Local infiltration on
    a. Green roofs: various types of green roofs in the Augustenborg Botanical Roof Garden and extensive type green roofs on few public buildings such as the home for elderly people.
    b. Green areas: there are large vegetated areas in Augustenborg, mainly lawn areas, which serve as infiltration facilities and delay runoff.

    c. Permeable parking areas consisting of a gravel layer mechanically stabilized using a grid of polyethylene.
    d. Vegetated waterways.
2. Storage and detention
    a. Ponds: many ponds of different shapes and functionalities are distributed in the area.
    b. Temporary storage facilities in case of excessive rainfall are built such as the amphitheatre in the schoolyard.
3. Slow transport of storm water in different drainage corridors, these are:
    a. Vegetated canals and swales.
    b. Canals with energy dissipaters such as the cube canal, “water drop” gutters and canals with wetlands.

Figure 2-2 shows various examples of the open stormwater solutions in Augustenborg.

(a)

(b)

(c)

(d)

(e)

(f)

Figure 2-2: Shows examples of open stormwater solutions in Augustenborg. ***(a)*** *An extensive green roof on top of a building.* ***(b)*** *A meandering creek in one of the parks.* ***(c)*** *A wet pond used to handle the water locally.* ***(d)*** *The amphitheatre in the school used as a temporary storage.* ***(e)*** *A swale in the east of Augustenborg, serves for both slow transport and infiltration.* ***(f)*** *Open drain with flow obstacles "water drop gutter".*

The old stormwater system in Augustenborg was not entirely replaced with the open drainage system, therefore the stormwater is still being handled in three different ways. As shown in Figure 2-3, the open drainage system handles the water in the northern and southern parts separately (A1 and A2), each part ends with a detention pond that connects eventually to the pipe network. The water from the streets and some other areas are handled by the pipe network (area B in Figure 2-3). The rest of the stormwater is handled locally in an open drainage system which ends in local ponds (area C in Figure 2-3), only a part of it joins the pipe network, indicated as area A3 in Figure 2-3.

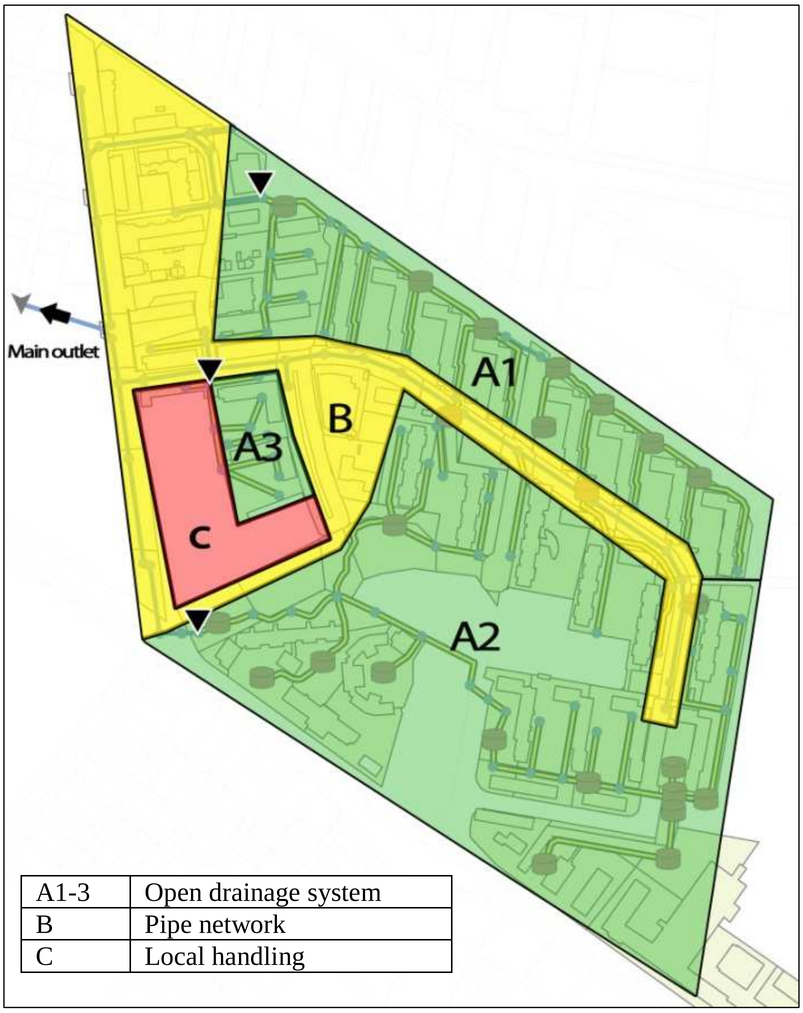


Figure 2-3: Stormwater handling methods in Augustenborg. The triangle symbols show the connection points of the open drainage system to the pipe network.

3. Hydraulic Description of Open Stormwater Solutions

3.1. Introduction

This chapter describes the hydraulics of the important open stormwater system components common in open drainage systems mentioned previously, focusing on the aspects relative to the subsequent modeling of the solutions.

The components discussed here are the open channels, ponds and green roofs.

3.2. Open Channels

Open channels, including canals, creeks and swales, in open drainage systems are mainly used to slow transport the stormwater, for the purpose of delaying the peak runoff discharge. They also contribute to infiltration, particularly swales and creeks, as well as the aesthetic value of such open conduits (Stahre, 2006).

In stormwater systems the characteristics of flow in open channels and pipes changes with time due to the variation of the rain intensity, therefore the flow is considered to occur under non-steady conditions, i.e. the velocity and the depth of the flow changes over time.

To simulate the unsteady, gradually varied flow conditions, various hydraulic models are suggested by French (2007) based on approximate solutions of the St. Venant equations, which consist of the continuity equation

$$\frac{\partial y}{\partial t} + y\frac{\partial u}{\partial x} + u\frac{\partial y}{\partial x} = 0$$

Equation 3-1

where

y = depth of flow (m)
t = time (s)
u = average velocity of flow (m/s)
x = longitudinal distance (m)

and the one-dimensional conservation of momentum equation

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial y}{\partial x} - g\left(S_x - S_f\right) = 0$$

Equation 3-2

where

S_x = slope of the channel in longitudinal direction
S_f = friction slope
g = acceleration of gravity (9.81 m/s^2)

However, because of the complexity of these equations, numerical methods with the aid of computers are often used for the solution.

The friction slope S_f is dependent on the slope of the wave and the depth of flow and it is usually estimated from Manning or Chezy resistance equations (French, 2007).

The Manning equation in the SI units system is defined as

$$Q = MAR^{2/3}\sqrt{S_f} \qquad \textit{Equation 3-3}$$

where

Q = flow rate (m^3/s)
A = flow area (m^2)
R = hydraulic radius (m)
M = Manning resistance coefficient

and the friction slope is estimated from Manning equation as

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

the usage of the absolute value of the velocity $|u|$ together with the velocity u (instead of u^2) is to make sure that frictional resistance S_f always oppose the flow motion (French, 2007).

Perhaps the estimation of the resistance coefficient of the channel, such as Manning coefficient M is the most challenging task in solving these problems, because in most cases it is difficult to measure the resistance directly, since in open stormwater solutions different materials and combination of materials both natural and manmade are used for lining the channels.

A technique used to measure the resistance coefficient is to simulate steady and uniform flow conditions, so that the Manning equation is directly applicable to the channel (French, 2007). This can be done for example for a set of flow data that can be considered steady, or simulating these conditions for manufactured channel units in the laboratory.

In the absence of such data, there are other methods available to estimate the roughness coefficient, usually based on values measured in previous successful designs. Table 3-1 shows Manning roughness coefficient for selected types of conduits common in stormwater handling systems (French, 2007).

Table 3-1: Example values of roughness coefficient (M) based on values given by (French, 2007).

Material	**Manning (*M*)**
Concrete (rough)	68
Concrete (smooth)	85
Plastic	80
Grass (lawn)	20
Cement mortar (neat)	90
Masonry	40
Rubble	30

3.3. Storage Ponds

In urban areas, ponds are used to retard the flow or runoff by storing a certain volume and thus reducing the peak flows downstream, which helps preventing erosion of the receiving stream or reduce the load on the treatment plants. Ponds can be designed to infiltrate the water and are also useful to reduce the pollutant through settling particles (Akan, 1993).

There are many properties that are involved in the hydraulic design of a pond, depending on the purpose intended for a particular pond; these properties will have more or less significance on its performance. Shilton & Harrison (2003) discussed many of these properties, and those of importance to the smaller storage ponds in urban areas can be summarized as

- Flow rate;
- Inlet properties such as size, shape, placement and orientation;
- Outlet properties;
- Geometry of the pond and any flow obstructions exist, such as baffles or vegetation.

The hydraulic design of ponds start with finding the hydraulic retention time (HRT), which is calculated as

$$t_n = \frac{V}{Q_{Design}}$$ *Equation 3-5*

where t_n is the theoretical hydraulic retention time (days);
V is the volume of the pond (m^3);
Q_{Design} is the design flow rate (m^3/d). (Shilton & Harrison, 2003)

To compute the outflow from a pond, given the inflow hydrograph and the pond characteristics it is possible to use the routing equation below (Akan, 1993)

$$I - O = \frac{dS}{dt}$$ *Equation 3-6*

where I is the inflow rate (m^3/s)

O is the outflow rate (m^3/s)

S is the volume of water in the pond (m^3)

and t is the time (s)

The pond characteristics are normally given by the stage-discharge and stage-storage relations observed in a pond (Akan, 1993).

3.4. Green Roofs

Green roofs are known to contribute retention and detention of the stormwater. In Augustenborg Botanical Roof Garden it is seen that green roofs reduce about 50% of the yearly runoff (Stahre, 2008).

Commonly there are two types of green roofs; extensive and intensive green roofs. The extensive ones consist of a thin layer of soil covered by plants that can tolerate draught periods. Intensive green roofs consist of a much thicker soil layer and the plants differ from those used on extensive roofs, larger plants and trees can be planted on those (Stahre, 2008).

Villarreal & Bengtsson (2005) studied the response of an extensive sedum green roof to individual rain events through a laboratory procedure and linear programming methods. They investigated the roofs under different conditions; different roof slopes and rainfall scenarios were considered.

The composition of the soil in the test model was 5% crushed limestone, 43% crushed brick, 37% sand, 5% clay and 10% organic material. In conclusion an average unit hydrograph was constructed for such green roofs (Figure 3-1), which shows the runoff from a green roof in response to 1 mm effective rainfall. The unit hydrograph is used to estimate the runoff amount resulted from any individual rain event. For example, for a rain event of depth 8 mm, the runoff depth after 2 minutes will be 8×0.275=2.2 mm, referring to Figure 3-1.

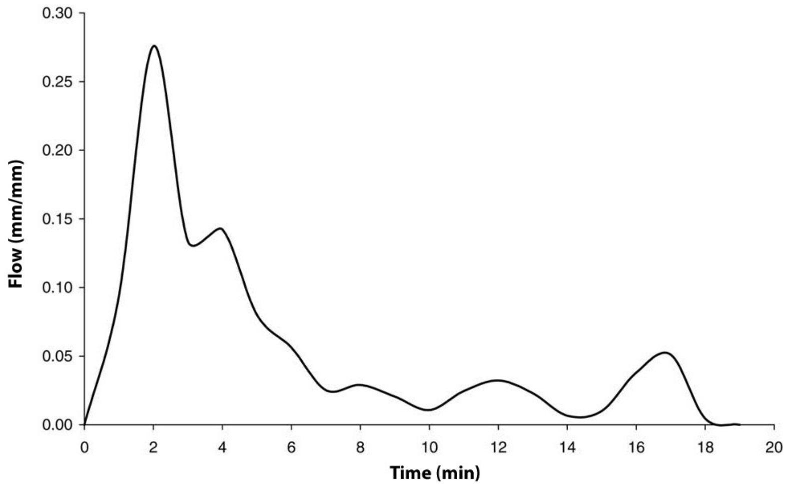


Figure 3-1: Average unit hydrograph (1 mm) (Villarreal & Bengtsson, 2005).

It was also found that the slope of the roof does not change the shape of the hydrograph within the tested range between 2º and 14º; however it affects the retention ability of the green roofs in dry conditions, as retention ability decreases with steeper roof slopes. In dry conditions, 12-16 mm is required to initiate the runoff from the green roof (Villarreal & Bengtsson, 2005).

4. MIKE URBAN Basics

4.1. Introduction

In this chapter, some basic principles of the modeling software used in this study are discussed; highlighting the parts relative to the modeling of the open stormwater solutions mentioned previously and the adjustments required.

4.2. MIKE URBAN MOUSE

MIKE URBAN is a commercial software developed by DHI Group; it is a tool for modeling various urban water distribution and collection systems. The part of interest here is the collection system, in particular collection of stormwater. The software provides two engines for modeling collection systems, the MOUSE engine and the SWMM engine, where the first one is chosen for this study.

The MOUSE engine can be used for modeling hydrological as well as hydraulic modeling of the collection systems. The hydrological model simulates urban storm runoff i.e. rainfall-runoff simulation (MOUSE hydrological model). The result of this simulation can then be fed to the collection network as a hydraulic load to carry out the hydraulic computations (MOUSE pipe flow model) which computes water volumes, levels and velocities in the system. Figure 4-1 illustrates the interaction between the hydrological modeling and hydraulic simulation in the collection network (DHI Software, 2009a).

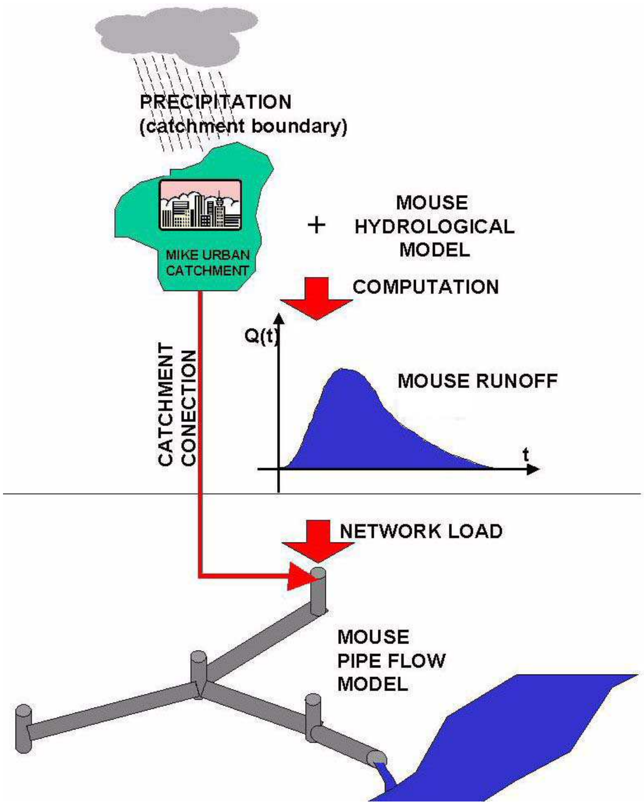


Figure 4-1: Illustrates the concept of modeling a collection system in MOUSE consisting of a hydrological and a hydraulic model (DHI Software, 2009a).

Building the rainfall-runoff model involves defining the following (DHI Software, 2009a)

- *Catchments*.
- *Catchments connections* to the hydraulic network.
- The *hydrological models* to be used to generate runoff.
- Precipitation.

The runoff computations are then carried out and the resulted runoff volume is applied as the hydraulic load for the collection network, which in turn is defined as (DHI Software, 2009a)

- *Nodes* and *structures*, which include *manholes*, *basins*, *storage nodes*, and *outlets*.
- *Links* (*Pipes* and *canals*).
- Other elements such as *pumps*, *orifices*, *gates*, *weirs* and, *stormwater inlets*.
- Additional hydraulic loads.

Figure 4-2 shows the basic elements in a MOUSE collection network.

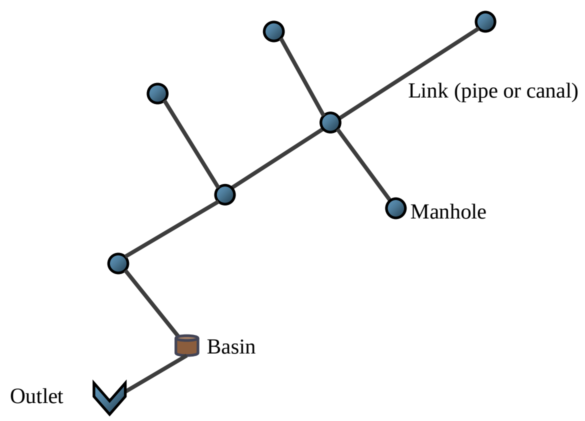


Figure 4-2: Illustrates a basic collection hydraulic network in MOUSE.

4.3. Rainfall-Runoff Modeling

This section deals with the first part of the model, which is the rainfall-runoff model. Here, the necessary terms are introduced and defined.

4.3.1. Catchments

The surfaces that contribute to the runoff to the drainage system are defined as MIKE URBAN catchments; the catchments are common to both computational engines (MOUSE and SWMM). A catchment is defined mainly by its area, other optional parameters such as the general slope, location and an additional flow can be defined if necessary.

4.3.2. The Hydrological Model

The MOUSE engine allows to choose among several hydrological models to simulate the surface runoff, these are (DHI Software, 2009a)

- Time-Area method
- Kinematic Wave
- Linear Reservoir
- Unit Hydrograph Method (UHM)
  And a continuous hydrological model, MOUSE RDI.

Each of these models has its own parameters, to use any of them; the corresponding parameters must be defined for each catchment. The selection of a hydrological model depends on surfaces types and the available information on different surfaces. The time-area model is used for Augustenborg model, since it requires less data than the other models.

The parameters required to define the time-area model as described by DHI Software (2008) are:

Impervious area: is the percentage of catchment area contributing to actual runoff.

Initial loss: is the precipitation depth required to initiate the surface runoff, represents the wetting and storage. It occurs only once per rainfall event.

Hydrological reduction factor: a reduction factor accounts for any water losses due to evapo-transpiration, imperfect imperviousness and other losses.

Time-area curve: defines the shape of the catchment, whether it is rectangular, divergent or convergent. It determines the relation between the flow time and the corresponding sub-area of the catchment. For example, a divergent catchment shape means that the sub-areas of the catchment are larger towards the outlet point. The default pre-defined curves are shown in Figure 4-3.

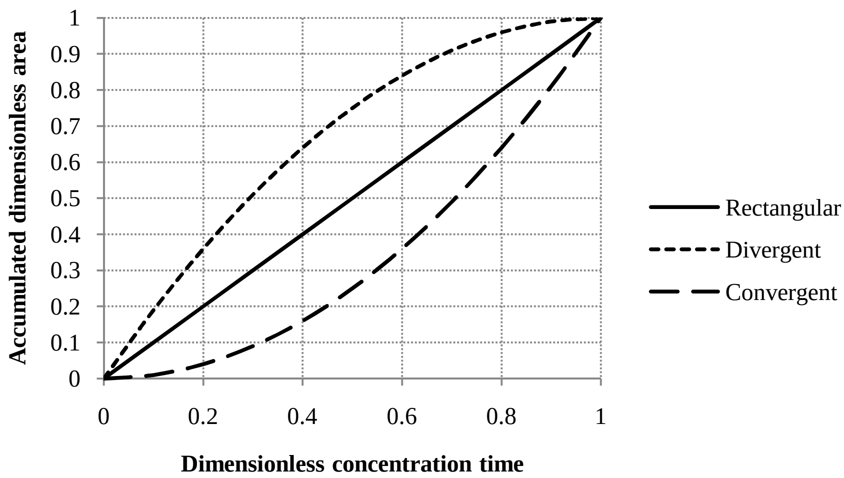


Figure 4-3: Pre-defined time-area curves in MIKE URBAN (DHI Software, 2008).

Concentration time: is the time required for a water drop to travel from the farthest point on a catchment to the outflow point. Here calculated as the travel time from the centroid of the catchment area to the connected node center.

4.3.3. Precipitation

Measured rainfall data or synthetic rainfall events can be used as catchment loads in the rainfall-runoff model; they can be imported or entered manually.

If the rainfall series include more than one rainfall event, the dry periods between these events are determined when the intensity of rainfall is zero and there is no more runoff from the catchments. Then the "initial loss" depth stored starts drying out at a constant rate until the next rainfall event occurs (DHI Software, 2008).

4.3.4. Connection to the Collection Network

The catchments are connected to the corresponding nodes in the collection system to transfer the simulated runoff water volume to the network to perform the hydraulic simulation (DHI Software, 2008).

4.4. Hydraulic Network Modeling

The hydraulic network modeling includes the definition of the geometric properties of the links and nodes, the hydraulic frictional losses, and any additional flows.

Networks consist basically of links (pipes or canals) connected together with nodes (manholes or basins) as starting points or junctions. A network typically ends with one outlet or more, that can be considered the final receiving water for example. It can also be connected to any of the other structures, such as pumps, weirs, orifices and gates or stormwater inlets.

In the following sections, the different elements constituting a hydraulic network relevant to this study are explored, and the required parameters are discussed.

4.4.1. Manholes

In the MOUSE model a manhole is geographically defined by x and *y* coordinates. The shape of the manhole can only be cylindrical, and the geometry is defined by diameter, invert level and ground level. By default, pipes or canals are connected to manholes so that the invert level of the canal is equal to the invert level of the manhole. There are different methods available to simulate the head loss in a manhole (DHI Software, 2009b).

4.4.2. Pipes and Canals

The geometry of a link (pipe or canal) in the drainage system is defined by its length and cross-section. The length can either be defined by a straight line or a poly-line between two nodes (DHI Software, 2009b).

For pipes, the cross-section can be chosen from the pre-defined cross-sections such as the circular, rectangular or other standard shapes, alternatively a custom cross-section (CRS) can be defined. While for a canal, a CRS is necessary to define its cross-section (DHI Software, 2009b).

There are different methods to define a CRS in MIKE URBAN, tabulated height and width (H, W open) data method is used in this study. An illustration of this method is given in Figure 4-4. Canals of uniform cross-sections are well represented with this method, but for canals of irregular cross-sections throughout the channel length, an average CRS can be assumed when the difference is not very large.

H	W
H_1	W_1
H_2	W_2
H_3	W_3

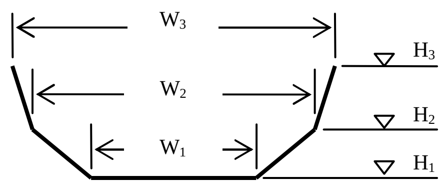


Figure 4-4: Definition of the channel cross-section using the "H, W open" tabulated data.

The hydraulic frictional losses are accounted for by defining the *material* of the link, this implies determining the coefficient used in the velocity equation used to compute losses. Depending on the equation chosen, different coefficients are required, for this study Manning equation is chosen, i.e. Manning number (M=1/n) is defined for the links.

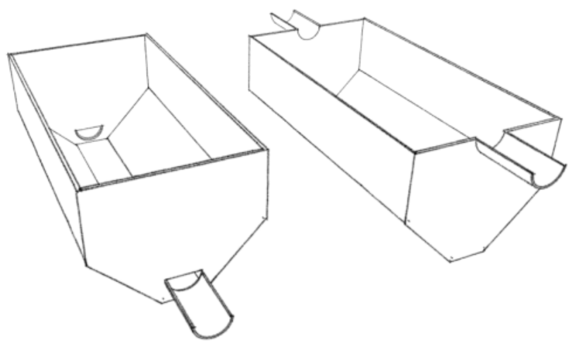
The infiltration from a link can be simulated in a simplified approach, by assigning a constant negative network load per link length.

During the simulation, if the water level in a canal exceeded the cross-section height, the simulation will stop with an error. It is possible to configure the MOUSE engine to extrapolate the cross-section at a specified angle by a specified factor, or adjust the cross-sections manually to have a depth larger than the maximum expected water level in order prevent flooding.

4.4.3. Basins

Ponds can be simulated in MOUSE using the basin nodes, since basins can represent nodes with a specified volume that can retain water. A basin is defined using tabulated values of the surface area and cross-sectional area perpendicular to flow direction at different depths of the basin.

Similar to the case of manholes, in MOUSE the links are connected to the basin at the invert level by default, which is the case for a dry pond, but to model a wet pond, the links invert level must be raised above the bottom level to the desired height manually, this is illustrated in Figure 4-5.



(a) (b)

Figure 4-5: An illustration showing a basin with different links configurations to model (a) dry ponds, (b) wet ponds.

Similar to the procedure used for channels, simple infiltration can be simulated for ponds by assigning a constant negative network load, which simulates an outflow from the system.

5. Building the Model

This chapter describes the method used to build the stormwater model of Augustenborg, starting with defining the hydrological and hydraulic model followed by the sensitivity analysis to decide the significance of model parameters and finally the calibration of the model.

The initial values of the parameters used are mostly based on the discussions in the previous chapters or based on the default values provided in MIKE URBAN, however some of them are changed later in the calibration stage.

The hydrological model (the rainfall-runoff model) is first discussed followed by the hydraulic network model.

5.1. Rainfall-Runoff Model

The rainfall-runoff modeling starts with defining the geometric properties of the catchments, in which the catchments polygons were readily available in the form of ESRI shape files provided by VA SYD, Malmö. The polygons were originally categorized into different classes, such as buildings, blocks, roads, parks, etc, but were further categorized and subdivided in MIKE URBAN based on aerial photos and architectural plans to account for different roof types and land uses. The catchments and the land-use categories are shown in Figure 5-1.

Only areas connected to the stormwater system in Augustenborg are connected to the model, the few areas that are connected to the existing combined sewer system or handled locally are not connected, since there is no interference from these areas on the stormwater system.

The catchments connections to nodes were made to agree with the conditions in reality as reasonable as possible, and the water is assumed to travel from the centroid of the catchment to the node center, during a time equal to the time of concentration.

Figure 5-1: The MIKE URBAN catchments and land-use categories.

5.1.1.Hydrological Model (Time-Area method) Parameters

The hydrological model chosen was the Model A, the time-area model. The parameters required for this model are described in section 4.3.2. In the following sections the values used for each parameter are listed.

Imperviousness: The impervious area percentage of the surfaces is given in Table 5-1 below, obtained from (Svenskt Vatten, 2004) with adjustments of some values to suit the land-use categories of the catchments in Augustenborg.

Table 5-1: The percentage of impervious area for different types of surfaces in urban areas (Svenskt Vatten, 2004).

Surface Type	Impervious Area %
Tile roofs	90
Concrete and asphalt	80
Residential areas with veg. and concrete	35
Sand covered park areas	20
Extensive green roofs	15
Lawns	10
Intensive green roofs	5

Initial loss: The default value in MIKE URBAN of 0.6 mm is set for all the catchments as an initial value. For the extensive greenroofs a value between 12-16 mm can be set, but the default value is kept for simplicity since the total percentage of greenroofs in Augustenborg model compared to the other surfaces is about 4.6%.

Hydrological reduction: The default value of 0.9 was assumed for the initial model state.

Time-Area curve: The default time-area curve corresponding to a rectangular catchment was kept (see Figure 4-3).

Time of concentration: The time of concentration for each catchment was calculated automatically in the program based on the distance between the catchment centroid and the connected node center, and the mean surface velocity. The mean surface velocity was set to the default value of 0.3 m/s.

5.1.2.Precipitation:

The rainfall event used for the sensitivity analysis and calibration of the model was measured by VA SYD during the period July to October 2009, see Figure 5-2. The location of the rain gauge was in the study area.

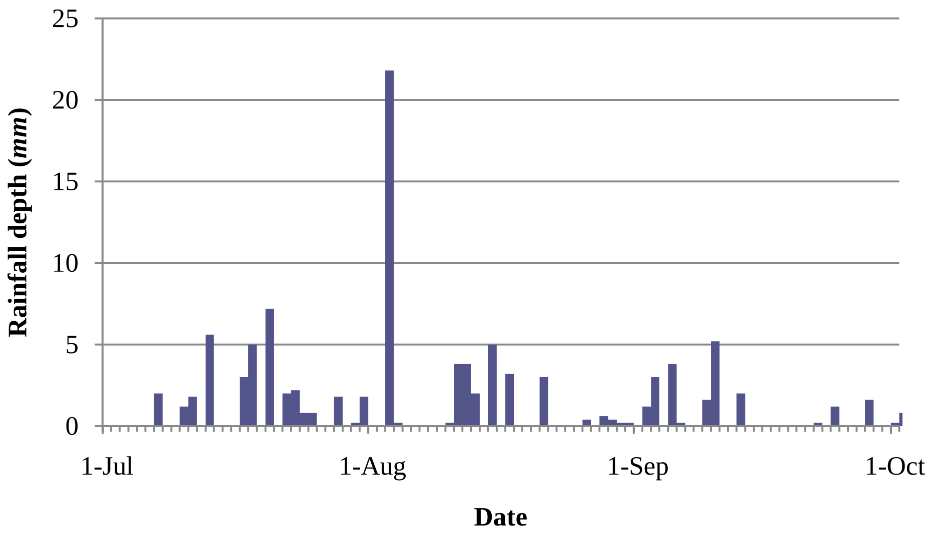


Figure 5-2: Measured total daily rainfall depth over Augustenborg for the period July 2009 to October 2009.

5.2. Hydraulic Network Model

The hydraulic network modeling includes defining the geometric and hydraulic properties of the nodes and links in the system.

The alignment of the existing stormwater and combined sewer network were readily available as ESRI Shape files, provided by VA SYD, Malmö, and where imported to the model, therefore the discussion will be focused on the open drainage system.

The open drainage system was drawn in MIKE URBAN according to the site plan of Augustenborg shown in Figure 5-3. The dimensions of the irregular channels and ponds were either taken from the maps or measured directly on the site. The elevations of the different points of the system were taken from measurements provided by VA SYD as well, but the missing elevations were interpolated from the known elevations.

5.2.1. Links (pipes and canals)

The layout of the open drainage system was based on the site plan of Augustenborg. The lengths and alignment where taken from the drawings. However, not all of the drains where included in the network, the smaller and shorter drains were omitted for simplification also because the computations are inaccurate for links shorter than 10 meters in MOUSE engine.

The Manning roughness coefficients for different types of channels were set based on Table 3-1, while for channels of composite cross-sections, consisting of more than one material, an average coefficient was estimated.

For channels with flow obstacles, such as the "*water drop gutters*" shown in Figure 2-2 (f) for example, the flow resistance in the model is increased through adjusting the material parameters to give an approximate effect of the obstacles.

To simulate the infiltration a constant negative load was assigned to swales and grass channels in parks where infiltration is expected. The amount of infiltration was set to 8mm/hr (VAV, 1983 cited in Thysell, 1997).

Figure 5-3: An overview map over Augustenborg (Green Landscaping AB).

5.2.2. Nodes (manholes and ponds)

The inlet points and junctions in the open drainage system were modeled as circular nodes of diameters equal to the width of the largest connected link. The geometry of the ponds was based mostly on the site plan. The head loss in the nodes is calculated using the Engelund formula.

The layout of the hydraulic network is shown in Figure 5-4.

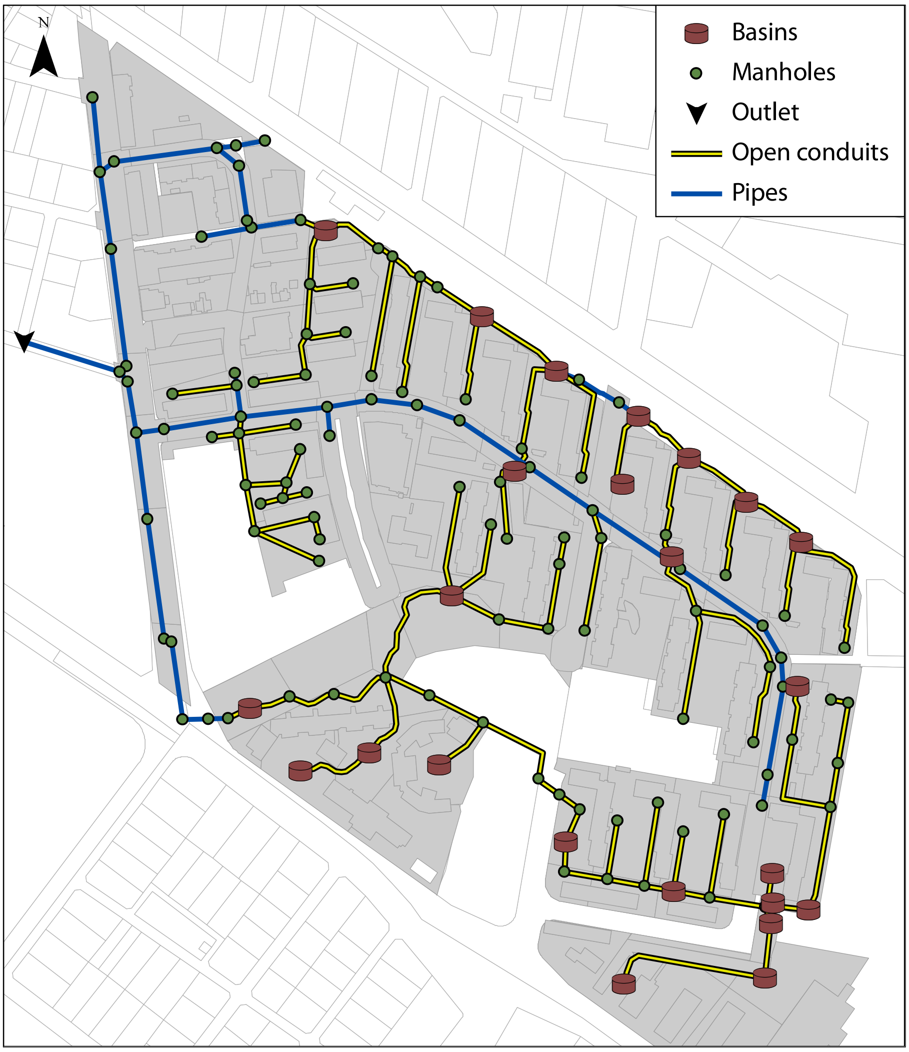


Figure 5-4: The layout of the hydraulic network of Augustenborg in MIKE URBAN.

5.3. Assumptions

For simplification purposes, unavailability of information or model limitations, assumptions were made in the modeling process. These assumptions are listed in this section.

System outlet: In the points where the water leaves the system, it is assumed that the receiving waters are big enough to omit backwater effects.

Infiltration: Because there was no information available on the infiltration rate and capacity, a constant infiltration rate of 8 mm/hr was used for links and some ponds.

Small links: Some of the shorter stormwater drains were not included in the hydraulic to simplify the modeling.

5.4. Sensitivity Analysis

Since many parameters are involved in the model, it is necessary to reduce the number of parameters that will be altered in the calibration stage to simplify the process. To determine which parameters are more significant, a sensitivity analysis is carried out.

The procedure is to change one parameter at a time within a range and then the simulation is performed for a certain time period. The resulted discharge hydrograph from each test is compared to the initial hydrograph obtained by using the initial parameters, here called the "*base*" values.

The range limits for each parameter was chosen to about double of half of the base value whenever permissible. The range limits for some parameters were changed during the analysis to further investigate the model behavior to these changes.

The parameters that are included in the analysis, along with the base values and the test range considered are listed in Table 5-2.

Table 5-2: Parameters included in sensitivity analysis, the base values, and the test range.

Parameter	Base values	Lower range	Upper range
Impervious area %	as per Table 5-1	½ base values	2× base values (max 100%)
Initial loss (*mm*)	0.6	0.3	2
Mean surface velocity (*m/s*)*	0.3	0.05	1.0
Hydrological reduction	0.9	0.3	1.0
Time-area curve	rectangular	convergent	divergent
Roughness coefficient M	as per Table 3-1	½ base values	2× base values
Infiltration rate in links (*mm/hr*)	8.0	4.0	16.0

*The mean surface velocity is used by MIKE URBAN to compute the time of concentration.

The discharge is observed in the links at points where the open stormwater system connects to the pipe network, denoted as monitoring points in Figure 5-5 which represent only the discharge from the open drainage system.

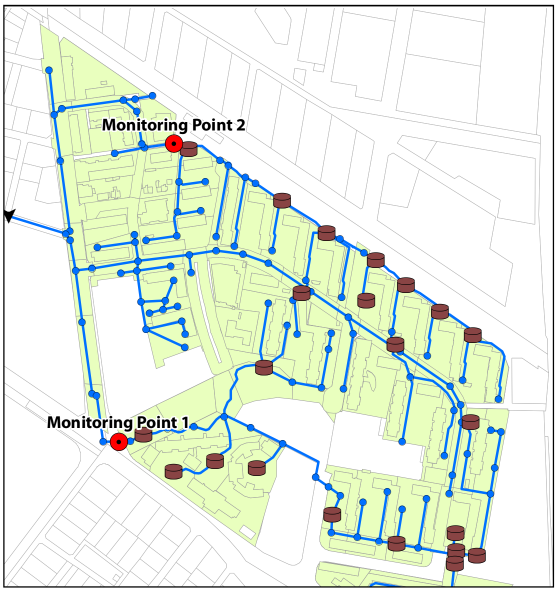


Figure 5-5: The location of monitoring points.

The result of the sensitivity analysis for each parameter is discussed below:

Impervious area %: The imperviousness of the surfaces, as expected had a well pronounced effect on the discharge hydrograph, the larger impervious areas result in higher peaks which implies a higher runoff volume as shown in Figure 5-6.

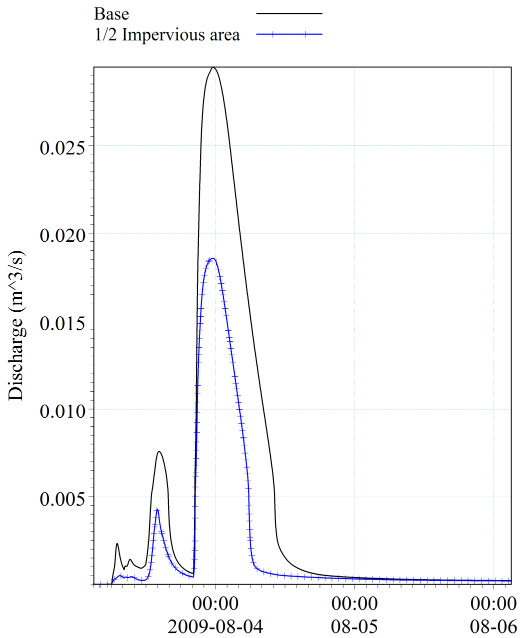


Figure 5-6: The impervious area % reduced to the half.

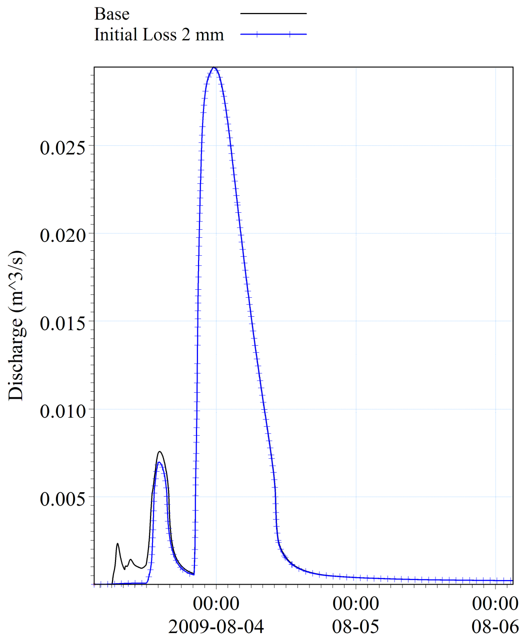


Figure 5-7: Effect of initial loss on the runoff.

Initial loss: Since the initial loss is a constant value that is subtracted from the rainfall depth at the beginning of the event, it can have a large or small effect depending on its value. In the range tested, it had a small effect on the discharge and the effect was pronounced at the beginning of the rainfall event as shown in Figure 5-7.

The mean surface velocity: This parameter is used by MIKE URBAN to compute the time of concentration for the catchments, using the catchment processing wizard. Changing this parameter did not result in a significant change (Figure 5-8), velocities lower than 0.05 m/s give very long concentration times, while velocities higher than 1.0 m/s give a time of concentration around 0. This is because the catchments defined in this model are relatively small and their centroids are close to the collection points.

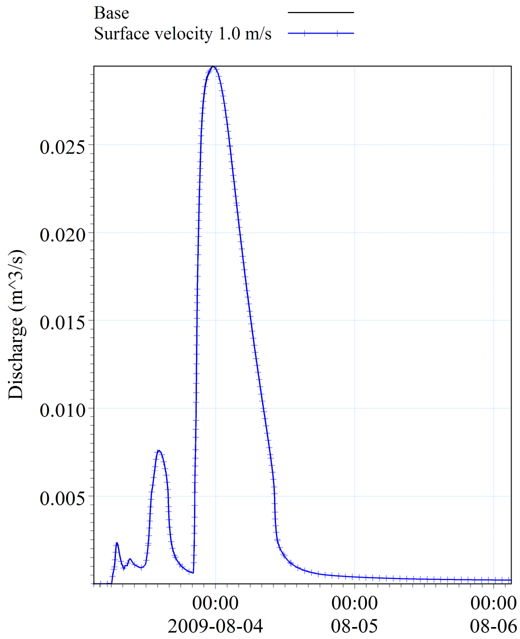


Figure 5-8: No significant change in the discharge hydrograph associated with changing the surface velocity.

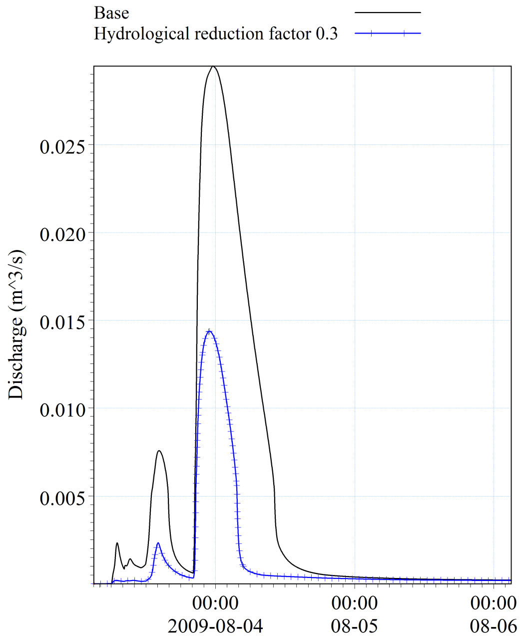


Figure 5-9: The effect of the hydrological reduction factor.

The hydrological reduction factor: This parameter has a similar effect on the runoff volume as the impervious area % has; this is noticed on the flow peaks in the discharge hydrograph. This factor has only a small change in response time was noticed which is shown in Figure 5-9.

The time-area curve: Changing the time-area curve had no noticeable effect on the discharge hydrograph when the pre-defined curves where selected (those being rectangular, divergent and convergent curves), possibly because of the small area of the individual catchments.

Manning roughness coefficient: As for the hydraulic network parameters, the Manning roughness coefficient is of most importance. Lower values of the roughness coefficient produce lower discharge peaks with slower response. Figure 5-10 shows the longer lag time resulted from assigning a lower Manning number.

Links infiltration rate: The infiltration rate from links has a moderate effect on the total discharge volume depending on the assigned infiltration rate, because it is defined as a constant rate per link length rather than a variable dependant on rainfall and soil conditions (Figure 5-11).

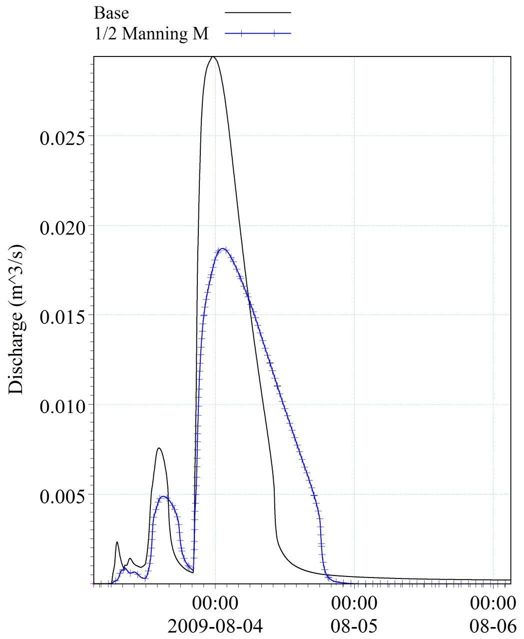


Figure 5-10: Effect of reducing the Manning coefficient to the half for the links in the system.

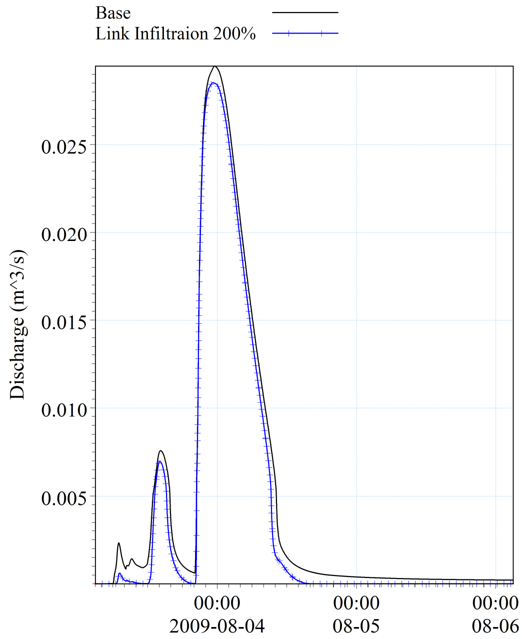


Figure 5-11: Increased infiltration rate in the links results in overall runoff volume.

Summary of Sensitivity Analysis

The sensitivity analysis results are summarized in Table 3-1.

Table 5-3: The summary of sensitivity analysis, showing the effect of each parameter on model results.

Parameter	Effect
Impervious area %	High effect on discharge volume
Initial loss	Low to moderate
Mean surface velocity	Very low
Hydrological reduction factor	High effect on discharge volume
Time-area curve	Very low
Manning roughness coefficient	High effect on lag time and discharge rate
Infiltration rate	Moderate effect on total discharge

5.5. Calibration

In order to get a better fit of the model to the reality, it is important to calibrate the model to get the simulated discharge as close as possible to the measured discharge, which is achieved by adjusting the hydrological and hydraulic parameters of the model accordingly.

The data used for calibration were discharge measurements, measured at two locations of the system, which are the points where the open drainage system joins the stormwater pipe network in Augustenborg. These are labeled monitoring point 1 and 2 in Figure 5-5.

The measurements were taken from 20-July to 02-October 2009, and the rainfall for the same period was measured. As shown in Figure 5-2, there was only one rainfall event of an effective depth to calibrate the model against, which was on 03-August. In the rest of the observation period there was either no discharge or very low discharge which did not give any conclusive comparison. Therefore the model was calibrated against that rainfall event, but validation was not possible because of the lack of data.

Because of the location of observation points, it was only possible to perform the sensitivity analysis and the calibration for the open drainage system, since the flow measurements were only available for those two monitoring points. It was assumed that the parameters obtained from the calibration suit well the rest of the model parts.

In the calibration process the correction of the high peaks was prioritized over the lower peaks, since they are more important for the overall evaluation of the system. The lower measured discharges (smaller than 5 *l/s*) were always simulated higher in the model because there is always a minimum water depth assumed in the links by the computational engine to make the calculations. The result of the calibration at monitoring point 2 is shown in Figure 5-12.

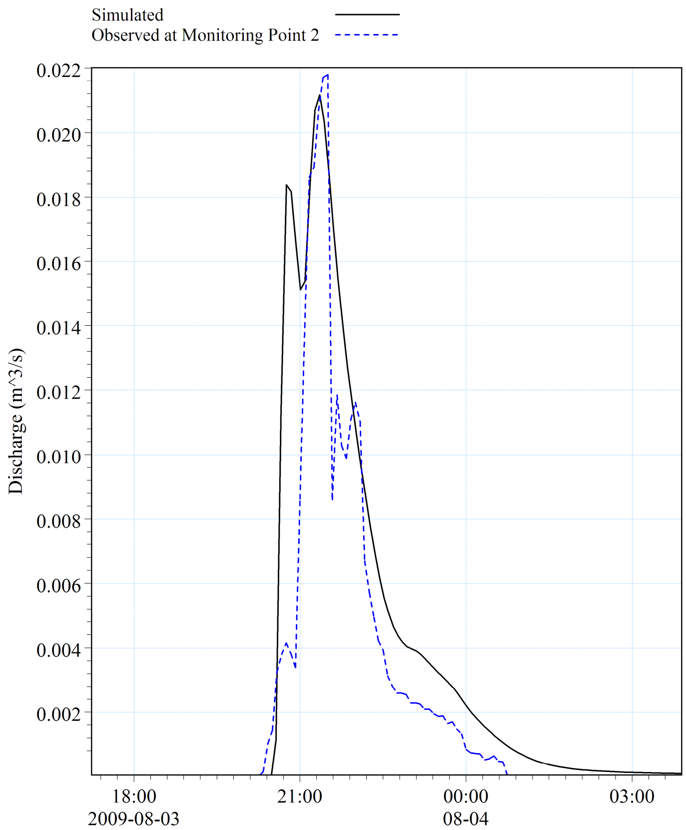


Figure 5-12: The simulated flow at monitoring point 2 compared to the measured flow.

At monitoring point 1a lot of smaller discharges were recorded by the measuring equipment, but not simulated in the model, probably because of an error in the equipment or some external flow conditions, for example a cross-over from a sewer pipe. The calibrated discharge from this monitoring point is shown in Figure 5-13.

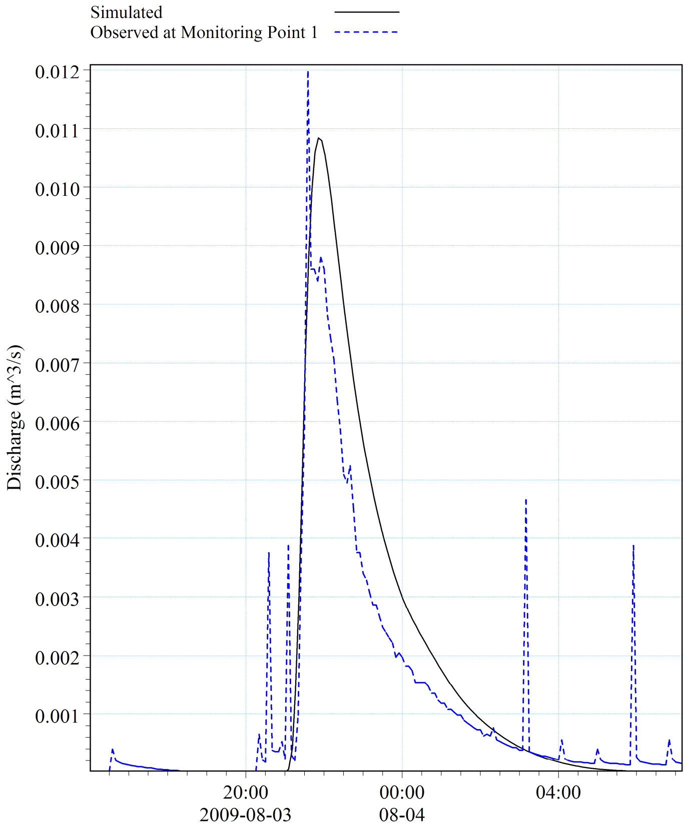


Figure 5-13: The calibration result in monitoring point 1.

Summary of Calibrated Parameters

Post the calibration the hydrological reduction factor was 0.55 and the initial loss was 3.0 millimeters. The mean surface velocity was set to 0.6 m/s and the calculated time of concentration varied from 0 to 8 minutes depending on the area of the catchment.

A summary of the impervious area % values for the different surface types are given in Table 5-4 and the values for individual catchments are given in Appendix D. The Manning coefficients for the different canals are shown in Table 5-5 and the full details for individual links are given in 0. The other parameters remained unchanged as the result of the sensitivity showed that they have a small effect on the model results.

Table 5-4: The impervious area percentage for the different surface types after calibration.

Surface type	Impervious Area%
Tile roofs	53.33 - 93.71
Concrete and asphalt	7.2 - 78.75
Residential areas with veg. and concrete	4.5 - 7.88
Sand covered park areas	2.7 - 3.0
Extensive green roofs	9.45 - 16.54
Lawns	2.7
Intensive green roofs	3.15 - 3.50

Table 5-5: The manning number (M) for the different canal types after calibration.

Material ID	Manning's Number (M)
Cement mortar (smooth)	100
Concrete (smooth)	85
Concrete (water drop gutters)	85
Stone	85
Cube Canal	80
Plastic	80
Cement mortar	77
Concrete canal with stone walls	65
Boulders	60
Macadam dike with grass sides	55
Grass	10

6. Simulation Scenarios and Results

6.1. Introduction

This chapter describes the different simulation scenarios covered in this study and displays their results. The scenarios are:

1. The existing open stormwater system in Augustenborg under the current weather conditions. This scenario aims to show how the system reacts to a rainfall event and form the basis for comparison in the next scenario.
2. The open stormwater compared to a conventional stormwater handling system consisting of a network of pipes and manholes without any open stormwater solutions. The comparison is done on the bases of the lag time and the discharge rate.
3. The last scenario simulates extreme weather conditions based on a synthetic storm for different return periods and storm durations. The open stormwater system is compared to the conventional system under these conditions to investigate flooding in both systems.

6.2. Scenario 1: The open stormwater system under current conditions

In this scenario, the open stormwater system response to rainfall is demonstrated under the current weather conditions represented by a measured rainfall event. A longer lag time is desired to have less severe flow conditions downstream.

The maximum daily rainfall was chosen from the measurements recorded in Augustenborg during the period 2007-2008. The event chosen on July 5th, 2007 is shown in Figure 6-1. The total rainfall depth of the event was 72 mm.

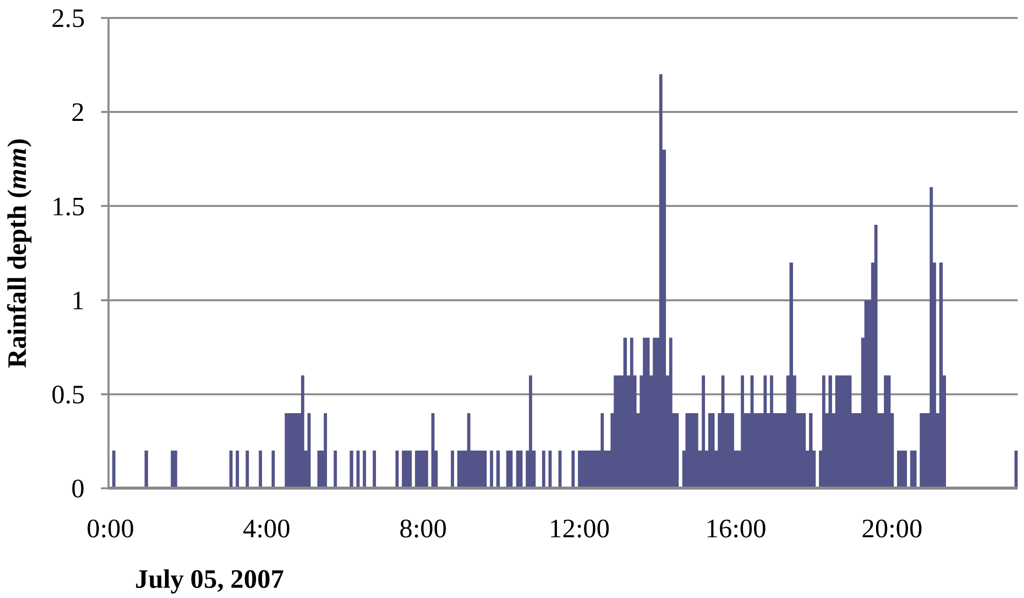


Figure 6-1: The rainfall event used in scenario 1 and 2 (5-min interval).

The simulation was run for the selected rainfall event and the discharge hydrograph at selected locations of the open stormwater system was observed, in order to examine the discharge lag time from the rainfall.

By observing the flow peaks at monitoring point 2 shown in Figure 6-2, it is seen that the response to the rainfall is delayed about 15-35 minutes depending on the conditions in the system. The response is faster when there is already water filling the hydraulic network, while it is slower when the network is empty, because of the time needed to fill the empty parts of the network. The longer delay at the beginning of the simulation is due to the fact that the precipitation is small and most of it is lost initially by surface wetting and infiltration.

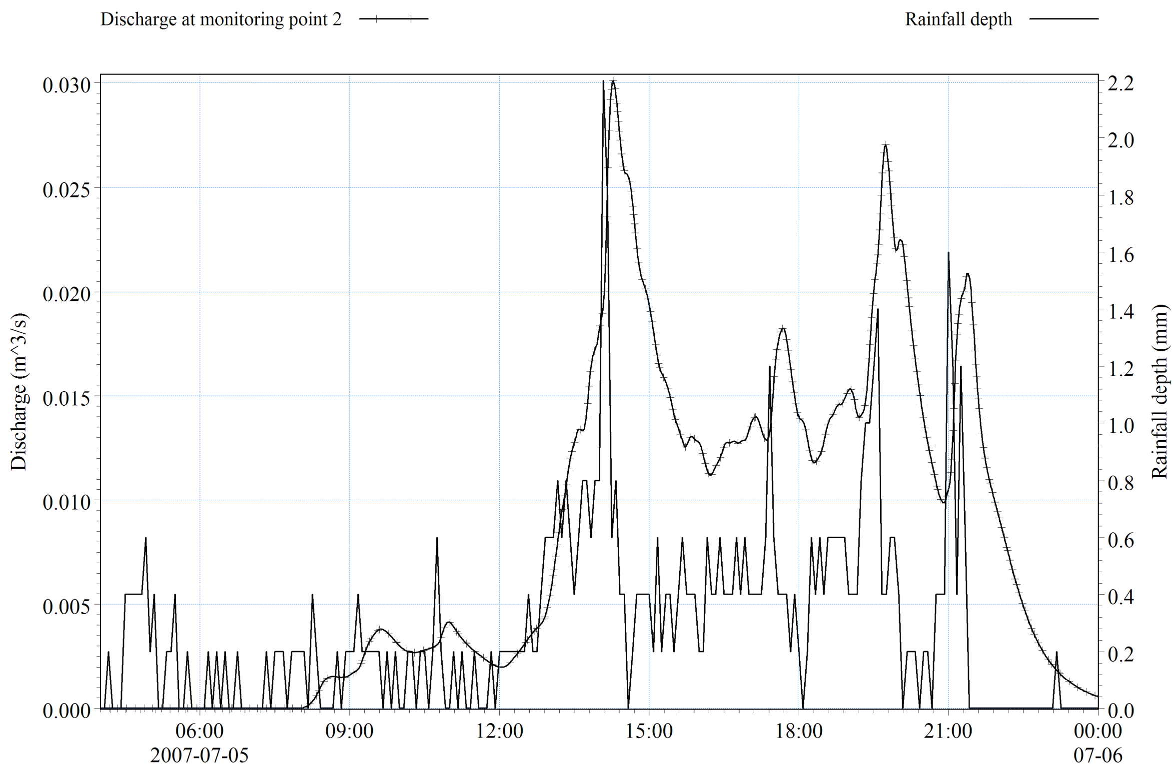


Figure 6-2: The discharge from monitoring point 2 (left axis) plotted against the rainfall depth (right axis, 5 minutes resolution).

At monitoring point 1 the response is slower than that observed at monitoring point 2, which can take up to an hour.

There was no flooding detected in any parts of the system as a result of this rainfall event, since the water level in the nodes and links did not exceed the ground level.

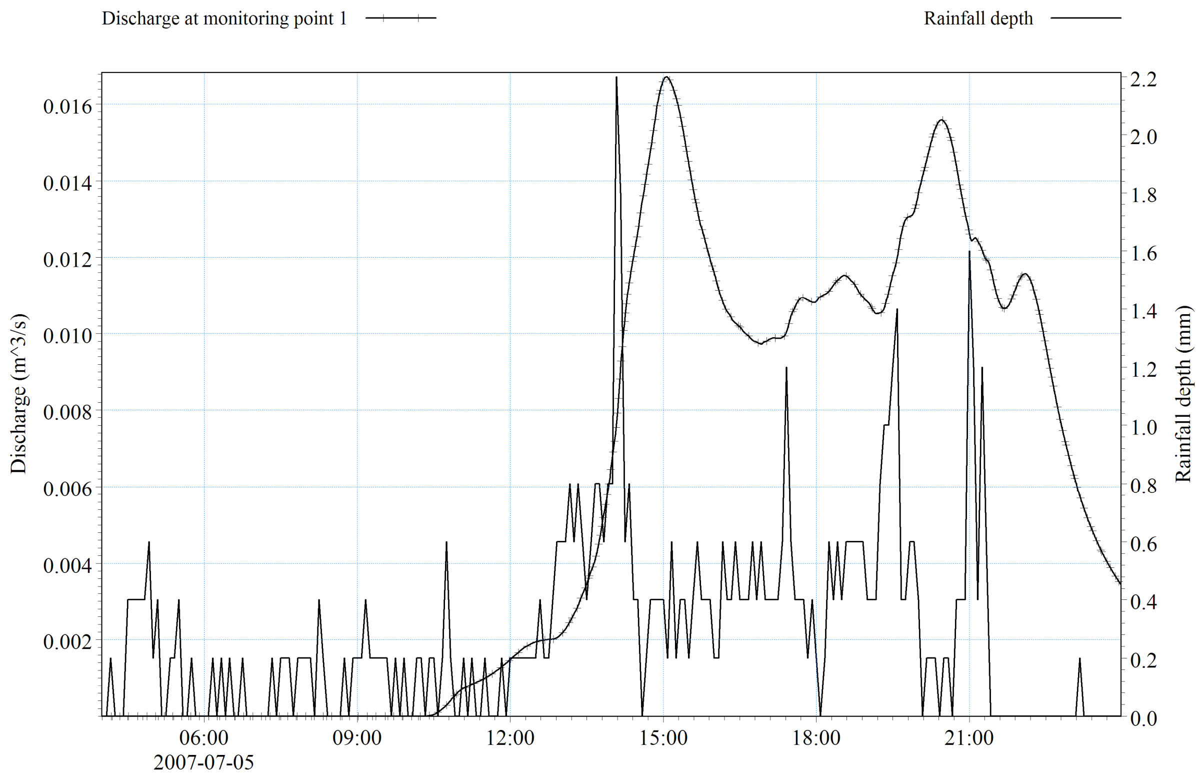


Figure 6-3: The discharge from monitoring point 1 (left axis) plotted against the rainfall depth (right axis, 5 minutes resolution).

6.3. Scenario 2: The conventional stormwater system

This scenario aims at evaluating the efficiency of the open stormwater solution in comparison to a conventional pipe system. For this purpose another model is built to resemble a conventional stormwater system in Augustenborg.

In the beginning the idea was to model the previously existed combined sewer system in Augustenborg before the renovations, but because the renovations were carried out along a long period of time, the original plans were not available.

Therefore it was not possible to model the old combined sewer system. Instead, the same layout of the open drainage system was used to model the conventional system by replacing the open drains with pipes, ponds with circular manholes and no infiltration was considered.

Figure 6-4 shows the hydraulic network layout of the conventional system, also showing the monitoring points at which the discharge was measured.

The catchments and land-use categories are kept identical; therefore the rainfall-runoff model is common to both models. In this simulation the same rainfall event as in the previous scenario is used to compare both systems.

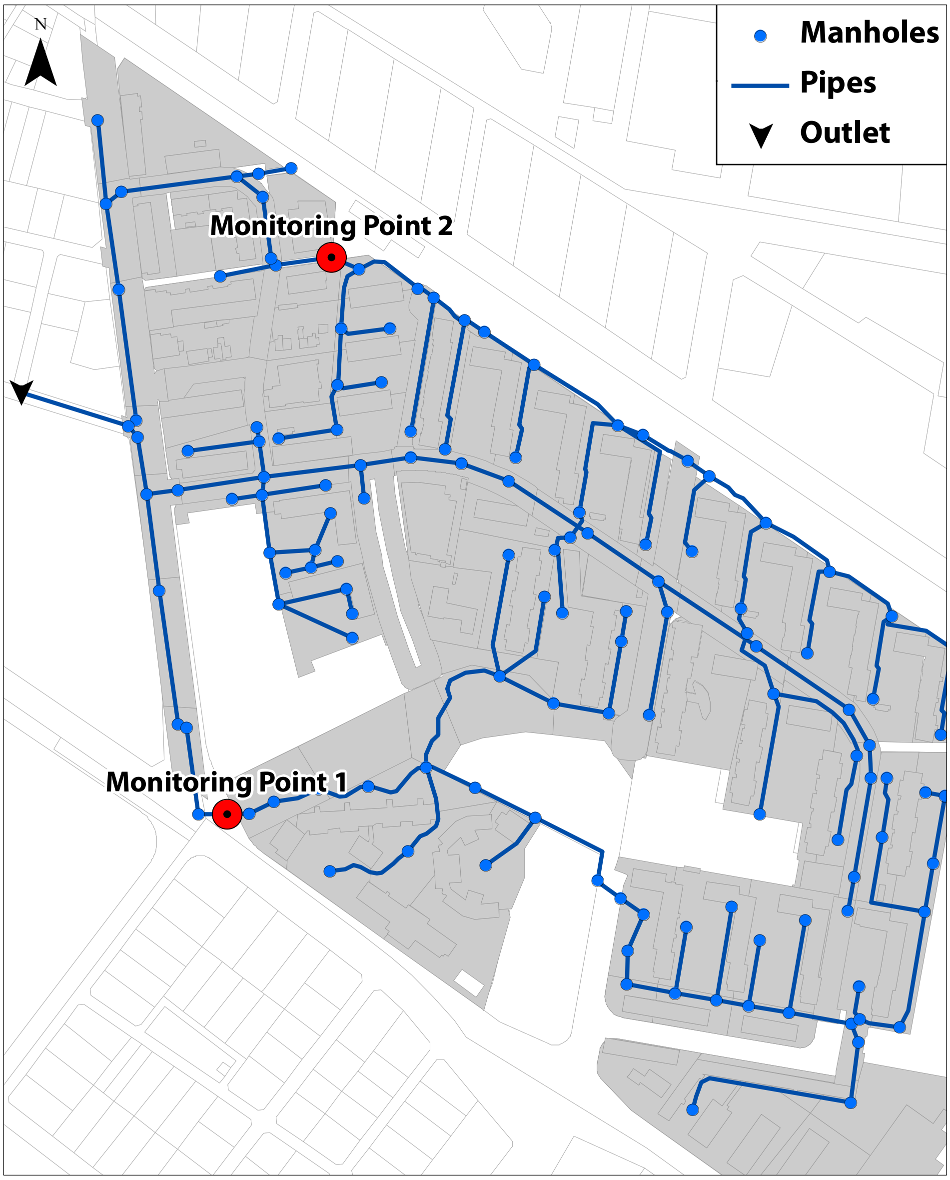


Figure 6-4: Pipe network model used in Scenario 2 and the location of monitoring points.

At monitoring point 2 it was noticed that the discharge rate in the pipe network was approximately double the discharge rate in the open drainage system as shown in Figure 6-5. This indicates the longer lag time resulted from the open solutions, however the peak timing difference between the two systems was only about 5 to 10 minutes, the pipe network being the faster one.

The pipe network response to smaller precipitation is also evident, which is clear at the beginning of the simulation. This is not seen in the open drainage system mainly because of the infiltration in the links and the volume stored in the ponds.

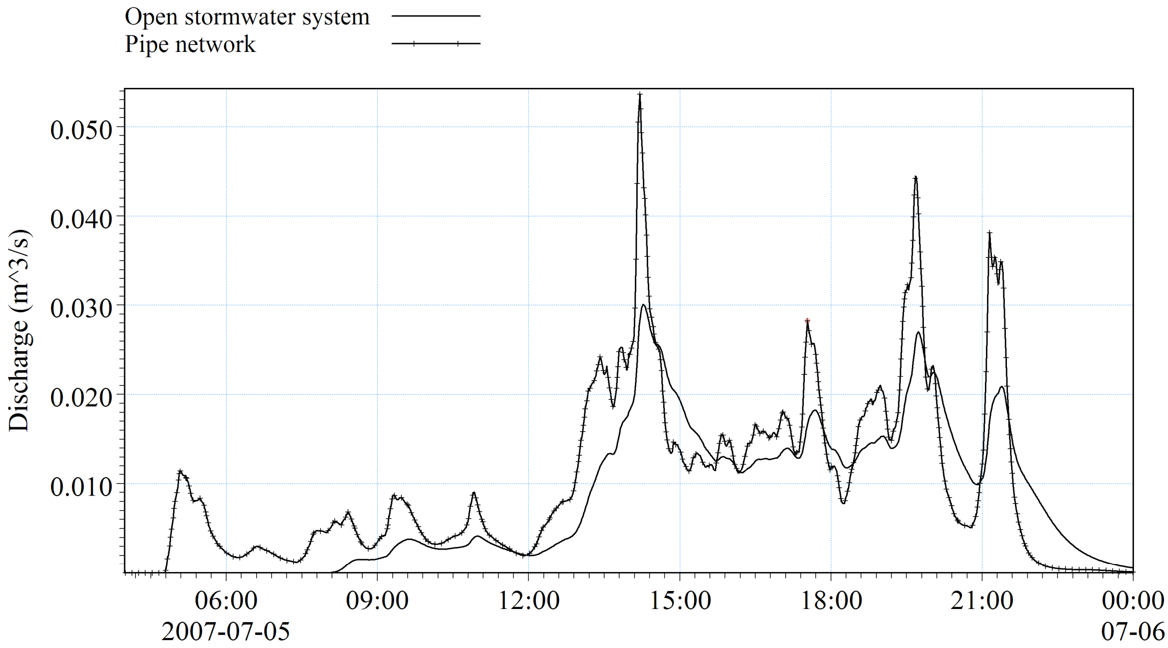


Figure 6-5: The simulated discharge at monitoring point 2 simulated for the open stormwater system compared to the discharge from the pipe network system at the same location.

The discharge in the open drainage system at monitoring point 1 shows a more pronounced effect of the open stormwater solutions. The smoother discharge hydrograph shown in Figure 6-6 indicates longer lag time in the open system, because of the larger number of ponds in this area. The peaks appear about 40 minutes later in the open stormwater system compared to the pipe network with a discharge rate of about 2.5 times lower. At this monitoring point less water volume is leaving the open system compared to the pipe system due to the infiltration along the water path in the links and nodes.

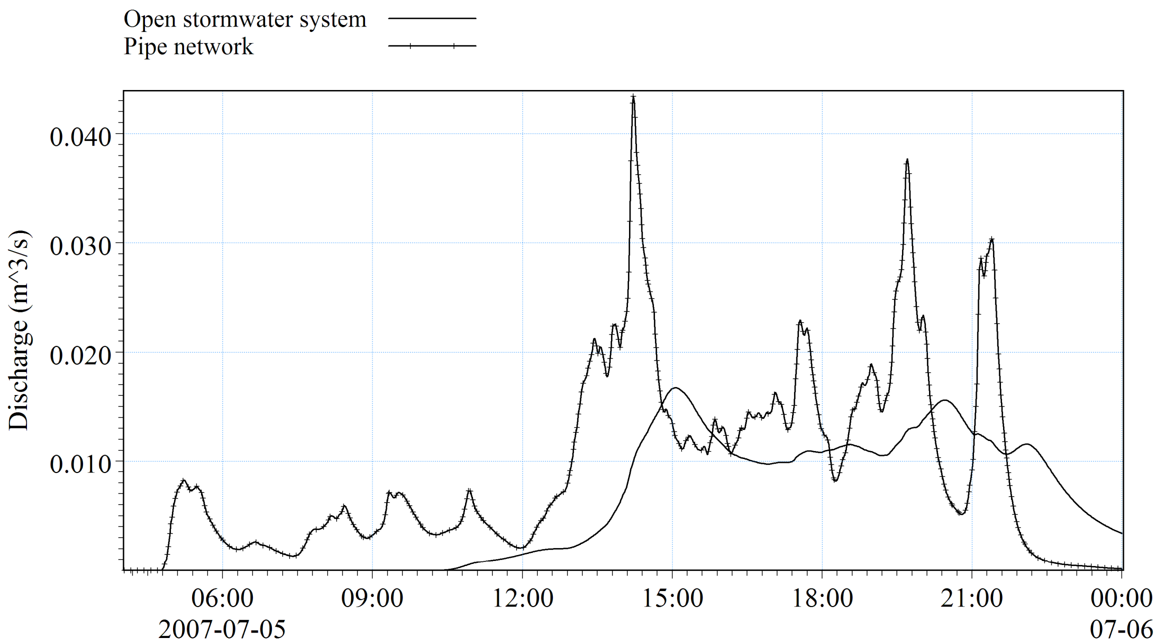


Figure 6-6: The simulated discharge at monitoring point 1 is plotted for both systems. The open drainage system exhibits a more evened out discharge hydrograph.

6.4. Scenario 3: Extreme rainfall conditions

To evaluate the performance of the open stormwater system, it is necessary to consider extreme rainfall conditions in order to spot any flooding risks. It is also of interest to find out which of the two systems considered handles flooding better. Therefore the extreme rain conditions will be simulated in both systems.

The rainfall duration-intensity curves were computed for Malmö city using Dahlström formula (Appendix A) for different return periods, these are shown in Figure 6-7.

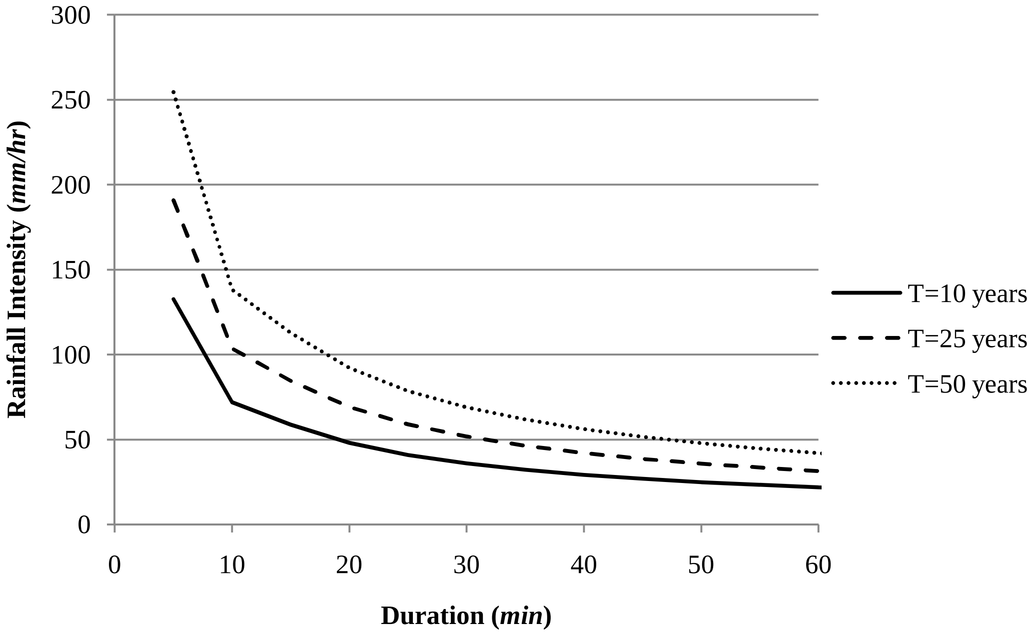


Figure 6-7: Rainfall intensity curves calculated using Dahlström formula for different return periods (T).

To find out which rainfall event is critical for flooding, different intensities over different durations were considered. The return periods taken were 10, 25 and 50 years for durations of 10, 20 and 30 minutes, the corresponding rainfall intensities are given in Table 6-1.

Table 6-1: The rainfall intensities and durations considered for the extreme weather analysis.

Return period (*years*)	Rainfall intensity (*mm/hr*) for duration		
	10 *min*	20 *min*	30 *min*
10	70	50	35
25	105	70	50
50	140	90	70

During the 50 years storm events of 10 minutes duration (140 *mm/hr* intensity) it was noticed that there is flooding at the some inlet points and shallow nodes in both systems. The flooding depth in nodes is calculated as the *water level - ground level*. The flooding in the open stormwater system is shown in Figure 6-8 and the flooding in the conventional pipe network is shown Figure 6-9.

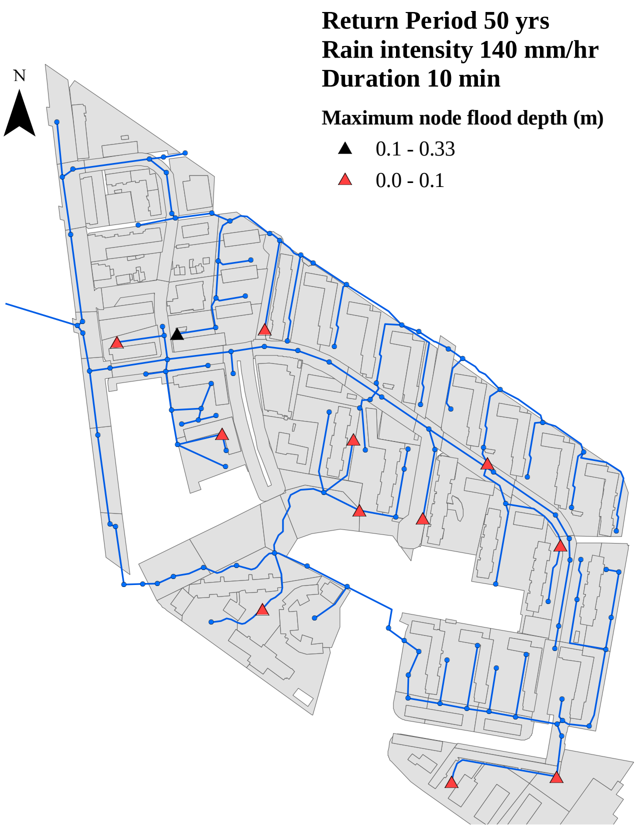


Figure 6-8: The maximum node flooding in the open stormwater system during the simulation period. The flooding depth is calculated as water level – ground level.

Figure 6-9: The maximum node flooding in the pipe network during the simulation period.

The duration of the same 50 years storm event was extended to 20 minutes to investigate the behavior of both systems under more severe conditions. The 140 mm/hr intensity rain this caused more flooding in both systems, but considerably higher in the pipe network.

The flooding in the open system occurred at inlet nodes and extended to the shallow nodes further in the system. The flooded nodes are shown in Figure 6-10. The flooding is calculated as the (water level - ground level) in nodes and the values are maximums for the whole simulation period. The maximum flooding height was about 0.34m above ground level.

Figure 6-10: Maximum node flooding in the open stormwater system caused by the 140 mm/hr intensity rainfall for the duration of 20 minutes.

The situation was different in the pipe network, as the flooding was more extensive and of larger depth. The maximum flooding depth was about 2 m as shown in Figure 6-11.

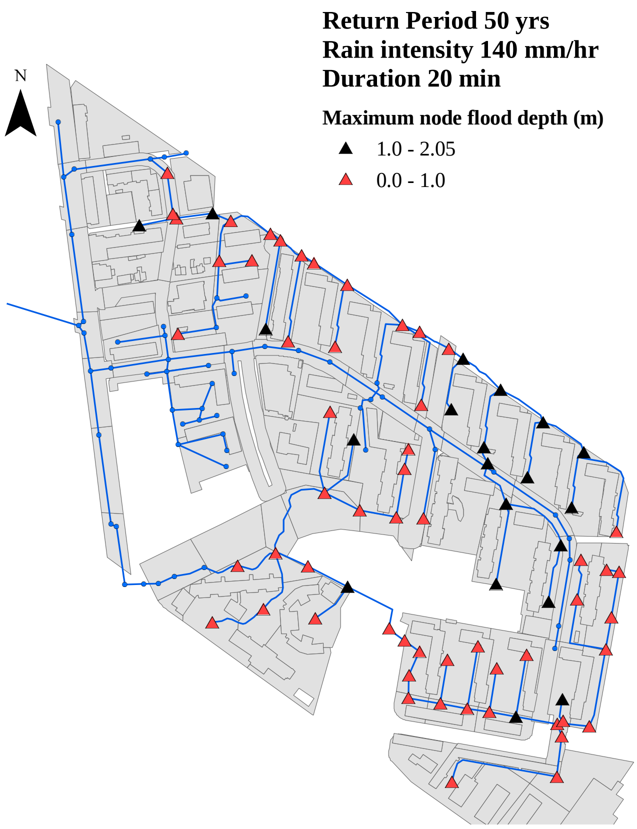


Figure 6-11: The maximum node flooding in the pipe network system caused by the 140 mm/hr intensity rainfall for the duration of 20 minutes.

The discharge at the monitoring points is observed in both systems, and is shown in Figure 6-12 and Figure 6-13

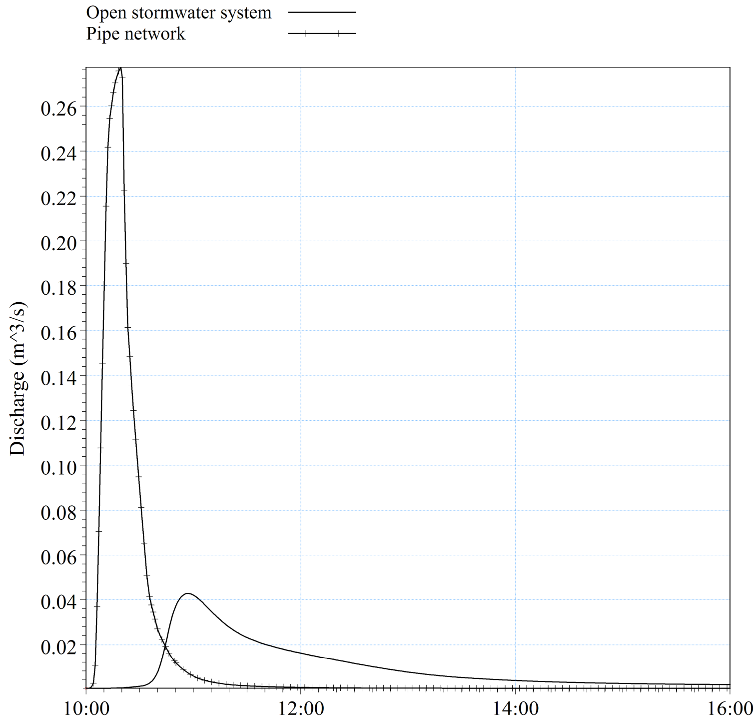


Figure 6-12: Showing the discharge from the open stormwater system and the conventional system at monitoring point 1.

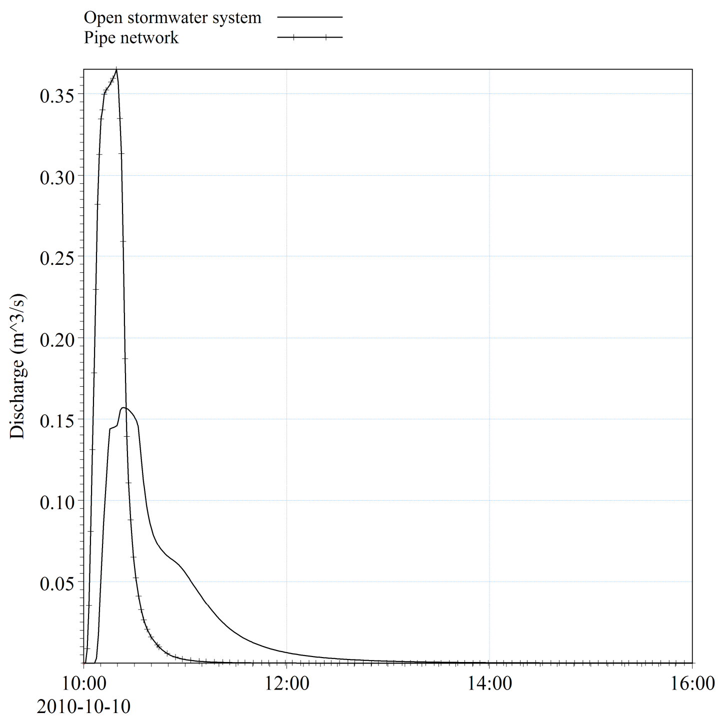


Figure 6-13: The discharge from the open stormwater system compared to the pipe network at monitoring point 2.

7. Discussion

7.1. General

In this chapter the issues related to each stage of the modeling are addressed, and results are discussed and interpreted.

7.2. Model Construction

Quality of input data: The first important consideration is the input data quality, it is essential to use good quality data to obtain usable results from the model. Starting with the geometrical and geographical data, in the case of Augustenborg there were no complete elevation data available, therefore the elevations were assumed or interpolated at many locations, which can alter the slopes of the links and the volume of the ponds for example. Also the geometric properties of the many irregular shaped channels were not available, and the dimensions were estimated from the site plan.

In deciding the level of details to include in the model, the amount of observed data and its distribution plays the major rule, because it is unnecessary to do a very detailed modeling if it is not possible to evaluate and validate the values of the parameters used. As there were observations from only two points in the area available, a lower level of details was preferred.

For example, for channels of composite cross-sections (consisting of more than one material) it is possible to define a section with variable roughness coefficient in MOUSE, but instead an average value was assumed here, since there was no possibility to evaluate the more complex cross-section.

Infiltration: The infiltration in this model was assumed as a constant rate, this can be accurate enough when the dry soil conditions exist, or there is only small amount of rain expected. When the soil is saturated, this assumption can lead to over-estimated infiltration.

Catchments connections: In the hydrological model, it is important to assign the catchments to the nodes reasonably, since it controls the amount of runoff entering the node and the time required for that, and hence will affect the overall response time of the system.

Evapotranspiration: The evapotranspiration in this model was based on a constant fraction included in the hydrological reduction factor. Using an evaporation model as a function of temperature can yield a better fit, especially when the temperature varies significantly within the rainy seasons.

7.3. Calibration

Perhaps the major concern in the calibration process was the lack of observed flow data. There was only one significant rainfall event during the observation time, at which the corresponding flow in the monitoring points was recorded.

With such short period data it is not possible to eliminate any errors or irregularities in the measurement, and it can lead to inaccurate conclusions in the final calibration results. Given that event was recorded at summer time, one expects highly localized rain showers leading to uneven distribution of the rain.

Another problem that arises with the lack of data, especially when the number of the observation points is not adequate, is the equifinality problem. The equifinality means that it is possible that many parameter sets can give the same output from the model, given the limitations of the model and the observed data (Beven, 2003). This can affect the accuracy of the model results. The equifinality effect can be reduced by taking observations at more points, and hence the outflow from an area becomes dependent on a smaller number of parameters which yields a better accuracy.

In Augustenborg (as one may expect in open drainage systems) it was noticed that vegetation grow in the open channels and litter accumulate at times which may have affected the flow observations.

Although it is an important stage of the modeling process, but the validation was not possible because the lack of flow measurements. This of course is a drawback, since it is the basis to judge the conformity of the model to the reality and evaluate the quality of the calibration.

7.4. Scenarios

7.4.1. Scenario 1

The main purpose of the first scenario was to show the behavior of the open drainage system in response to a normal rain event and form the basis for the comparison with the conventional system in the next scenario.

Starting at monitoring point 2, at the beginning of the rainfall period the discharge is not initiated, because the fact the rainfall depth is small and it is lost according to the *initial loss* parameter which accounts for the storage for surface wetting. Refer to Figure 6-2.

The lag time is longer at the beginning of the rainfall when the hydraulic system is empty (links and dry ponds) and it decreases gradually as system fills up with water. Therefore there is a variation from 15-35 minutes in the lag time.

In the MOUSE definition of a basin, a basin is always filled up initially up to the invert level of the lowest link connected to it. This means that there can be some excess water in the system at times, since in reality the water level drops in wet ponds due to evaporation and infiltration in dry periods. This can shorten the lag time in the model to a certain extent.

Examining another location, that is the monitoring point 1, the discharge hydrograph shown in Figure 6-3. It is seen that the lag time is considerably longer than the one observed in the previous case. Mainly because the water path is longer in this case, but also because the ponds along this path are larger and the channels cross-sections are larger, which transport the water more slowly.

It can also be noticed that the calibration at this point adds slightly longer lag time which can be considered when judging the efficiency of that part of the system.

7.4.2. Scenario 2

In this scenario the open stormwater system is compared to an equivalent conventional pipe system. The basis of the comparison was the discharge hydrograph.

The discharge hydrographs observed at monitoring point 2 shown in Figure 6-5 shows that the discharge rate in the pipe system is about the double of that in the open system. The main reason for this difference is attributed to the larger volume of the open system given by the larger links cross-sections area and the detention caused by the ponds.

The other factor is the lower roughness coefficients of the canals, which helps retard the flow and also the peak timing is delayed.

These effects are more clearly seen in monitoring point 1 with even larger difference in the discharge and peaks timing. This is because of the larger volumes of the network causing more retention as mentioned previously. There is also local backwater effect in some locations due to mild slopes. The infiltration in ponds and canals also contributes to this reduction.

One can argue about the fairness of this comparison regarding the way both systems are built. Since in pipe networks the designer would want to take the shortest paths, the fewest bents and turns, and more suitable slopes. While in open drainage systems for stormwater purposes the opposite is preferred, and here both systems are built with similar configurations.

However, a more careful design of the pipe network will most probably result in higher efficiency in terms of draining the stormwater more quickly. Therefore this comparison is still valid.

7.4.3. Scenario 3

Considering the rainfall scenarios in Table 6-1 applied to the model, both systems showed enough capacity to handle such rains with only minor flooding, at which the water level raised only a small amount above the ground level compared to the node depth. This can be due to the fact that the smaller branch links were omitted, as they can retain a certain volume of water.

However, with a more extreme case considered it was possible to distinguish which system handles such conditions better. The open system handled the rainfall event better favored by its higher capacity. In open systems even in flooding cases there is usually more room at the surface which can retain a certain volume of water during high peaks.

It is possible that the pipe network is somehow under-designed in this case, because the standard design procedures were not followed in this part of the model. But the flood height in the manholes gives an idea about the expected amount of water in such cases, even if the depth of a manhole is too small for example.

8. Conclusions

From the model results and the preceding discussion it can be concluded that the open stormwater systems with green solutions do contribute in reducing the peak flows. In this case 50% or more was reduced compared to the traditional pipe system.

The open solutions also delay the peak flow timing, which helps preventing high flow problems downstream.

In more extreme rainfall events, the open solutions handle the flooding better than the conventional systems and flooding is less critical specially when the water path is situated in parks and areas at a safe distance from buildings, as in the case of Augustenborg. Compared to combined systems that can cause considerable damage when flooding occurs, or separate stormwater systems that can flood streets for example.

In this context it can also be of interest to conclude from this experience whether the MOUSE engine in MIKE URBAN used in this study is suitable for modeling the open stormwater systems. One most important issue with the MOUSE engine is flooding, because in open stormwater models it is of interest to know what happens after the water exceeds the ground level. In such cases MIKE URBAN might be used in combination with another model that can simulate surface flooding, or implement another model.

Another concern is the way the engine handles cases when the water level exceeds the height of the cross-section of an open conduit, which is also common to open systems. In such cases the simulation is stopped. The only way to avoid this is to extend the cross-section above any expected water level; however in this case there is no direct way to find out when the water level exceeds the original cross-section height.

There is also the issue of the complexity of ponds in reality versus the simple definition of basins in MOUSE. It might be desired to define more complex configurations of ponds, such as irregular shapes that affect the flow in the pond, or different inlet and outlet conditions.

9. Recommendations and Future Work

To better describe the open stormwater solutions, it is essential to have adequate and good discharge measurements and at more spatially distributed to allow a better calibration and validation.

In such models with large green areas it is useful to use the Rainfall Dependent Infiltration (RDI) to better understand the infiltration. Provided there are enough measurements available (about 2 years of measurements).

It can be interesting to try the different hydrological models, such as the kinematic wave or the unit hydrograph model to find the most suitable model for such projects.

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Appendix A. Dahlström Formula

Dahlström formula for finding the rain intensity for a given city in Sweden (Svenskt Vatten, 2004).

$$i(t_r, Z) = 2.78(a + Z \cdot b)[1 + 0.1\,(t_r - 0.167)/(t_r - 0.157)]{t_r}^{-0.72} \qquad \textit{Equation A-1}$$

Or a simplified form

$$i(t_r, Z) = 2.78(a + Z \cdot b) \cdot c \qquad \textit{Equation A-2}$$

where

$i(t_r, Z)$ = rain intensity for a selected city in Sweden ($l/s \cdot ha$)
Z = regional parameter = 12 for Malmö city
t_r = rain duration (h)
$a, b\ and\ c$ = parameters given in Table A-1and Table A-2

Table A-1: Parameters a, b, and c to be used in Equation A-2.

Return Period (T)		**Constants**	
Months	**Years**	***a***	***b***
12	1	5.38	0.272
24	2	7.53	0.293
60	5	11.63	0.309
100	10	16.12	0.314

Table A-2: Parameter c for different rain durations t_r.

t_r (min)	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**
c	3.62	2.96	2.41	2.06	1.81	1.62	1.47	1.35	1.25	1.17	1.10
t_r (hr)	**1**	**1.5**	**2**	**3**	**4**	**6**	**8**	**12**	**16**	**20**	**24**
c	1.10	0.821	0.667	0.499	0.405	0.303	0.246	0.184	0.149	0.127	0.112

Appendix B. Open Stormwater System Model Data

B-1. Hydraulic Model Data: Links

Figure B-1 shows the layout of the hydraulic network of the open stormwater system. The properties of the links are given in Table B-1.The cross-sections (CRS) for the open conduits are given in Table B-2, and the Material ID’s for the links are given in Table B-3.

Figure B-1: The layout of the hydraulic model of the open stormwater system showing the links in the system. OP denotes an open conduit, SW is a stormwater pipe.

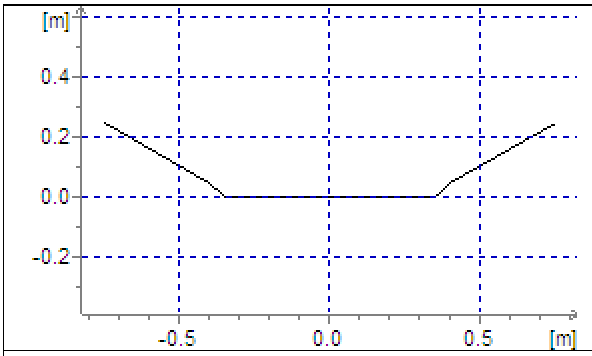
Table B-1: Open stormwater system links details.

Link ID	Dia. (m) or Cross-section	Length (m)	Upstream Level (m)	Downstream Level (m)	Slope %	Material ID
OP001	CRS_P	17.52	13.90	13.85	0.29	Concrete (Smooth)
OP002	CRS_F	71.50	12.60	12.10	0.70	Grass_Macadam
OP003	CRS_P	99.81	13.90	12.76	1.14	Concrete (Smooth)
OP004	CRS_O	23.87	15.49	15.40	0.10	Concrete_Onion
OP005	CRS_Da	28.44	15.34	15.31	0.11	Grass
OP006	CRS_B	6.73	15.31	15.28	0.45	Concrete (Smooth)
OP007	CRS_B	13.21	15.36	15.28	0.61	Concrete (Smooth)
OP008	CRS_O	46.11	15.45	15.20	0.54	Concrete_Onion
OP009	CRS_B	43.25	15.28	14.94	0.79	Concrete (Smooth)
OP010	CRS_B	28.37	14.94	14.86	0.28	Concrete (Smooth)
OP011	CRS_O	65.01	15.60	14.94	1.02	Concrete_Onion
OP012	CRS_Db	33.88	15.37	15.35	0.06	Grass
OP013	CRS_B	28.65	14.84	14.81	0.10	Concrete (Smooth)
OP014	CRS_P	114.38	15.60	14.50	0.96	Concrete (Smooth)
OP015	CRS_P	45.17	14.10	13.90	0.44	Concrete (Smooth)
OP016	CRS_Q	51.77	14.50	14.45	0.10	Concrete (Smooth)
OP017	CRS_P	84.89	14.80	14.50	0.35	Concrete (Smooth)
OP018	CRS_O	46.04	15.45	14.81	1.39	Concrete_Onion
OP019	CRS_L	60.65	14.46	14.26	0.33	Stone
OP020	CRS_Db	36.91	13.40	13.30	0.27	Grass
OP021	CRS_L	68.24	14.24	13.30	1.38	Stone
OP022	CRS_I	101.94	13.80	13.30	0.49	Grass
OP023	CRS_P	85.16	14.10	13.80	0.35	Concrete (Smooth)
OP024	CRS_O	91.15	12.87	11.70	1.28	Concrete_Onion
OP025	CRS_B	22.91	14.86	14.84	0.09	Concrete (Smooth)
OP026	CRS_O	36.35	13.29	12.40	2.43	Concrete_Onion
OP027	CRS_P	17.32	13.45	13.30	0.87	Concrete (Smooth)
OP028	CRS_P	49.64	12.05	11.90	0.30	Concrete (Smooth)
OP029	CRS_S	31.33	12.80	12.40	0.82	Stone
OP030	CRS_O	40.19	12.40	12.00	1.00	Concrete_Onion
OP031	CRS_O	12.37	13.40	12.80	4.85	Concrete_Onion
OP032	CRS_O	27.77	13.22	12.80	1.26	Concrete_Onion
OP033	CRS_O	19.14	13.48	13.40	0.42	Concrete_Onion
OP034	CRS_O	17.46	13.45	13.40	0.29	Concrete_Onion
OP035	CRS_R	22.12	14.25	11.86	10.81	Concrete (Normal)
OP036	CRS_R	72.28	14.75	14.25	0.69	Concrete (Normal)
OP037	CRS_O	79.94	13.40	12.76	0.80	Concrete_Onion
OP038	CRS_P	44.07	12.35	12.00	0.79	Concrete (Smooth)
OP039	CRS_O	67.19	13.09	12.10	1.47	Concrete_Onion
OP040	CRS_O	55.14	13.70	13.29	0.48	Concrete_Onion
OP041	CRS_P	47.75	13.30	13.29	0.03	Concrete (Smooth)
OP042	CRS_E	23.18	14.93	14.90	0.13	Grass_Macadam
OP043	CRS_E	27.15	14.89	14.86	0.11	Grass_Macadam
OP044	CRS_F	51.16	14.00	13.95	0.10	Grass_Macadam
OP045	CRS_O	58.48	14.29	13.95	0.77	Concrete_Onion
OP046	CRS_F	57.45	14.20	14.00	0.35	Grass_Macadam
OP047	CRS_O	65.72	14.80	14.20	0.91	Concrete_Onion
OP048	CRS_O	61.20	15.08	14.50	0.95	Concrete_Onion

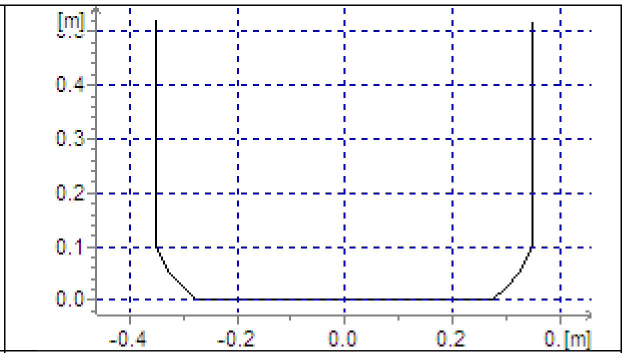
Link ID	Dia. (m) or Cross-section	Length (m)	Upstream Level (m)	Downstream Level (m)	Slope %	Material ID
OP049	CRS_F	55.23	14.50	14.20	0.54	Grass_Macadam
OP050	CRS_B	33.74	14.81	14.76	0.15	Concrete (Smooth)
OP051	CRS_P	21.25	12.08	12.00	0.39	Concrete (Smooth)
OP052	CRS_P	38.88	12.20	11.60	1.54	Concrete (Smooth)
OP053	CRS_Hb	36.86	13.20	13.00	0.54	Grass
OP054	CRS_Hb	32.56	13.00	12.86	0.43	Grass
OP055	CRS_A	125.02	16.15	16.05	0.08	Boulders
OP056	CRS_P	78.97	15.10	14.50	0.76	Concrete (Smooth)
OP057	CRS_Ha	27.19	11.70	11.50	0.74	Stone_Concrete
OP058	CRS_R	93.52	12.35	11.50	0.91	Concrete (Smooth)
OP059	CRS_F	48.29	11.30	11.25	0.10	Grass_Macadam
OP060	CRS_O	65.15	15.45	14.84	0.94	Concrete_Onion
OP061	CRS_F	41.02	12.10	11.90	0.49	Grass_Macadam
OP062	CRS_P	40.25	12.35	12.30	0.12	Concrete (Smooth)
OP063	CRS_Hb	47.60	13.30	13.20	0.21	Grass
OP064	CRS_P	35.34	11.70	11.60	0.28	Concrete (Smooth)
OP065	CRS_Ha	12.75	11.50	11.30	1.57	Stone_Concrete
OP066	CRS_P	32.35	12.30	12.20	0.31	Concrete (Smooth)
OP067	CRS_P	31.53	12.30	12.20	0.32	Concrete (Smooth)
OP068	CRS_Db	47.50	15.38	15.37	0.02	Grass
OP069	CRS_P	13.32	15.40	15.38	0.15	Concrete (Smooth)
OP070	CRS_P	20.78	14.58	14.55	0.14	Concrete (Smooth)
OP071	CRS_P	50.66	14.55	14.50	0.10	Concrete (Smooth)
OP072	CRS_P	38.72	14.50	14.45	0.13	Concrete (Smooth)
OP073	CRS_P	60.43	15.20	15.10	0.17	Concrete (Smooth)
OP074	CRS_Q	18.04	14.40	14.05	1.94	Concrete (Smooth)
OP075	CRS_Q	66.66	14.05	14.00	0.08	Concrete (Smooth)
OP076	CRS_P	67.62	13.87	13.80	0.10	Concrete (Smooth)
OP077	CRS_P	45.93	11.60	10.90	1.52	Concrete (Smooth)
OP078	CRS_B	19.09	14.70	14.58	0.63	Concrete (Smooth)
OP079	CRS_K	71.93	14.50	13.60	1.25	Concrete (Smooth)
OP080	CRS_K	46.43	13.60	13.40	0.43	Concrete (Smooth)
OP081	CRS_Ha	15.60	11.90	11.70	1.28	Stone_Concrete
OP082	CRS_N	47.75	14.96	13.60	2.85	Concrete (Smooth)
OP083	CRS_Da	81.83	15.35	15.34	0.01	Grass
OP084	CRS_P	83.81	15.40	15.35	0.06	Concrete (Smooth)
OP085	CRS_P	42.03	15.50	15.40	0.24	Concrete (Smooth)
SW001	0.3	18.33	13.80	13.60	1.09	Concrete (Smooth)
SW002	0.4	15.17	12.13	11.83	1.98	Concrete (Smooth)
SW003	1	77.57	8.33	8.18	0.19	Concrete (Smooth)
SW004	0.315	24.84	9.11	9.09	0.08	Concrete (Smooth)
SW005	0.3	35.53	13.10	12.90	0.56	Concrete (Smooth)
SW006	0.3	18.83	12.90	12.60	1.59	Concrete (Smooth)
SW007	0.3	20.16	13.80	13.40	1.98	Concrete (Smooth)
SW008	0.4	12.69	12.00	11.00	7.88	Concrete (Smooth)
SW009	0.5	6.05	9.14	9.13	0.17	Concrete (Smooth)
SW010	0.8	91.21	8.58	8.40	0.20	Concrete (Smooth)
SW011	0.6	6.63	8.38	8.35	0.45	Concrete (Smooth)
SW012	0.5	57.92	8.84	8.76	0.14	Concrete (Smooth)
SW013	0.3	22.40	12.93	12.85	0.36	Concrete (Smooth)

Link ID	Dia. (m) or Cross-section	Length (m)	Upstream Level (m)	Downstream Level (m)	Slope %	Material ID
SW014	0.5	42.56	9.12	9.06	0.14	Concrete (Smooth)
SW015	0.5	38.31	9.23	9.16	0.18	Concrete (Smooth)
SW016	0.6	21.85	8.98	8.93	0.23	Concrete (Smooth)
SW017	0.5	58.85	11.86	11.46	0.68	Concrete (Smooth)
SW018	0.3	19.92	11.58	11.50	0.40	Concrete (Smooth)
SW019	0.5	60.08	9.03	8.98	0.08	Concrete (Smooth)
SW020	0.5	65.42	11.41	10.53	1.35	Concrete (Smooth)
SW021	0.315	9.47	9.11	9.09	0.21	Concrete (Smooth)
SW022	0.315	22.86	10.19	9.97	0.96	Concrete (Smooth)
SW023	0.5	35.17	9.61	9.27	0.97	Concrete (Smooth)
SW024	0.6	93.25	8.86	8.73	0.14	Concrete (Smooth)
SW025	0.8	59.94	8.73	8.60	0.22	Concrete (Smooth)
SW026	0.5	34.94	9.25	9.15	0.29	Concrete (Smooth)
SW027	0.16	24.18	13.75	13.67	0.33	Concrete (Smooth)
SW028	0.2	39.10	9.89	9.72	0.43	Concrete (Smooth)
SW029	0.2	18.53	12.84	12.13	3.83	Concrete (Smooth)
SW030	0.4	77.53	12.71	12.37	0.44	Concrete (Smooth)
SW031	0.5	5.96	9.03	8.88	1.50	Concrete (Smooth)
SW032	0.6	80.01	8.98	8.89	0.11	Concrete (Smooth)
SW033	0.225	22.68	8.94	8.88	0.26	Concrete (Smooth)
SW034	0.5	66.59	9.13	9.05	0.12	Concrete (Smooth)
SW035	0.375	60.47	10.01	9.07	1.55	Concrete (Smooth)
SW036	0.6	67.10	8.71	8.60	0.16	Concrete (Smooth)
SW037	0.5	34.84	10.43	9.79	1.84	Concrete (Smooth)
SW038	0.3	69.24	13.65	12.95	1.01	Concrete (Smooth)
SW039	0.5	80.74	12.35	11.88	0.58	Concrete (Smooth)
SW040	0.4	28.38	12.83	12.72	0.39	Concrete (Smooth)
SW041	0.6	9.83	8.41	8.35	0.61	Concrete (Smooth)
SW042	0.8	39.93	8.52	8.43	0.23	Concrete (Smooth)
SW043	0.6	13.38	8.87	8.80	0.52	Concrete (Smooth)
SW044	0.5	22.52	9.04	9.02	0.09	Concrete (Smooth)
SW045	0.225	15.16	8.88	8.83	0.33	Concrete (Smooth)
SW046	0.4	20.86	14.58	14.50	0.38	Concrete (Smooth)
SW047	0.4	41.07	14.45	14.00	1.10	Concrete (Smooth)
SW048	0.3	42.41	16.00	15.60	0.94	Plastic
SW049	0.3	20.95	11.10	9.23	8.93	Concrete (Smooth)

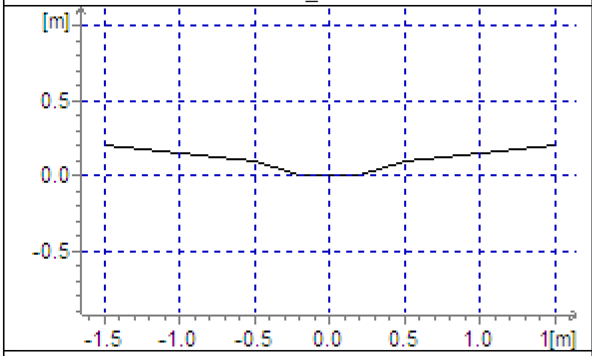
Table B-2: The cross-sections (CRS) of the open conduits in the hydraulic system.



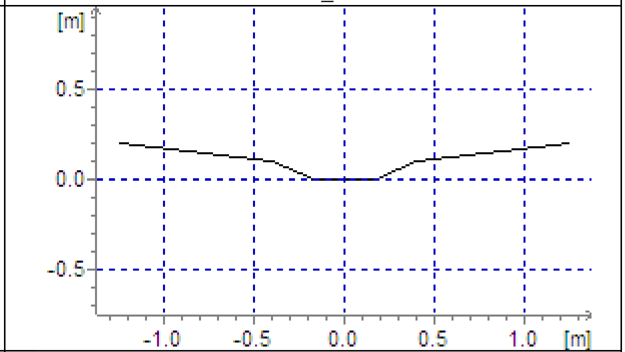
CRS_A



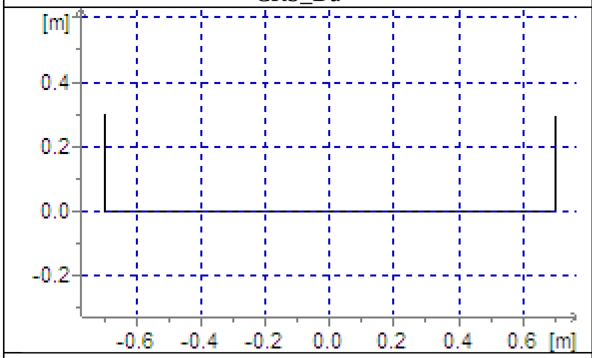
CRS_B



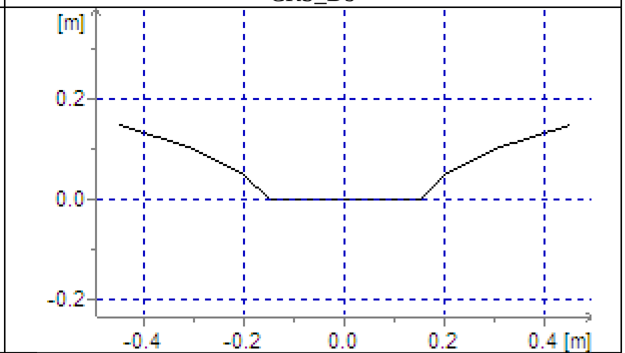
CRS_Da



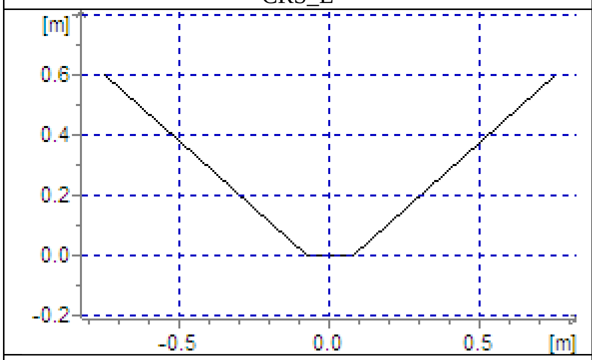
CRS_Db



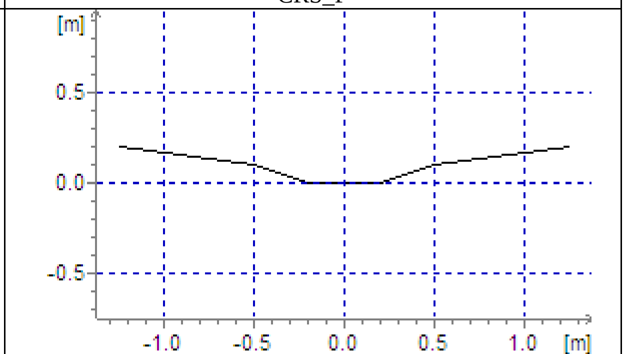
CRS_E



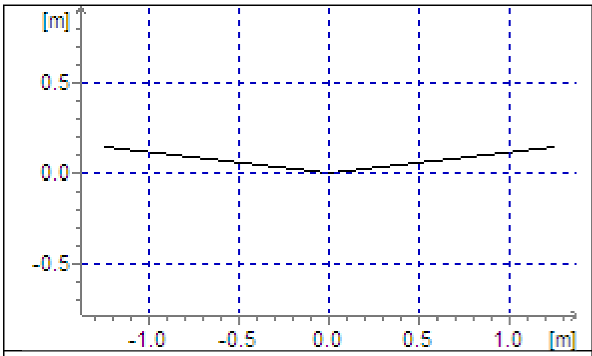
CRS_F



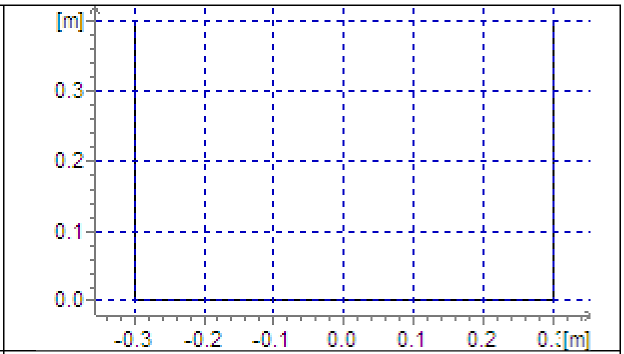
CRS_Ha



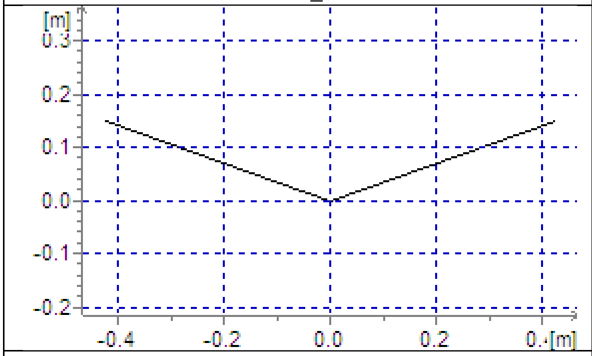
CRS_Hb



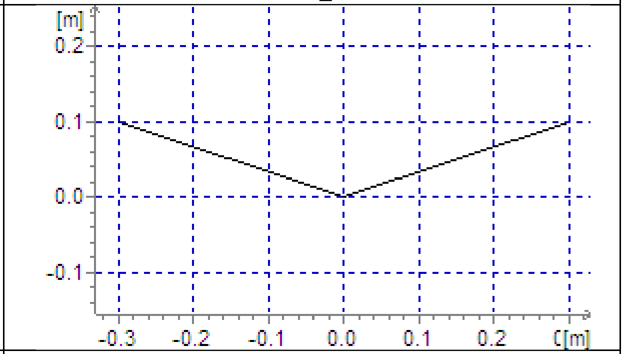
CRS_I



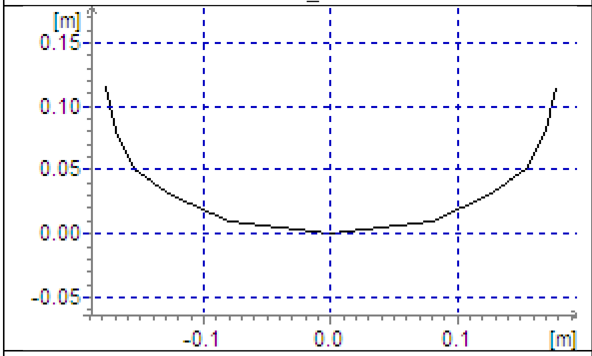
CRS_K



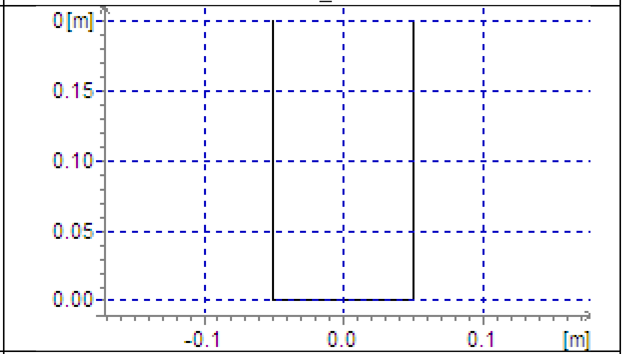
CRS_L



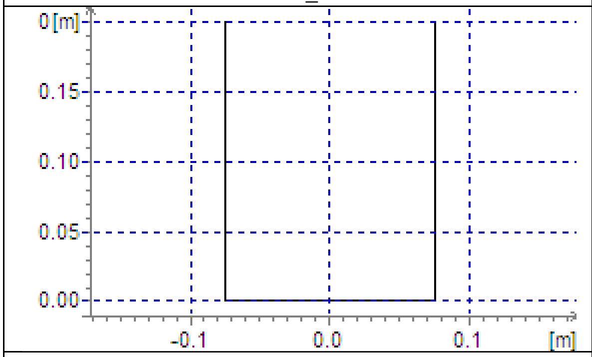
CRS_N



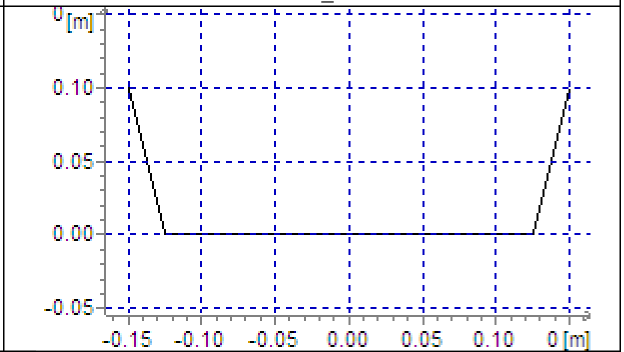
CRS_O



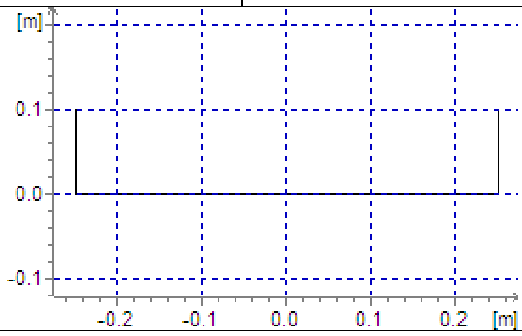
CRS_P



CRS_Q



CRS_R



CRS_S

Table B-3: The material ID's for the links in the hydraulic model.

Material ID	Manning’s Number (M)
Boulders	60
Cement Mortar	77
Concrete (Smooth)	85
Concrete_Onion	85
Cube_Canal	80
Grass	10
Grass_Macadam	55
Mortar (Smooth)	100
Plastic	80
Stone	85
Stone_Concrete	65

B-2. Hydraulic Model Data: Nodes

The nodes in the system are shown in Figure B-2, their dimensions and properties are given in Table B-4. The geometrical properties of the basins are given in Table B-5.

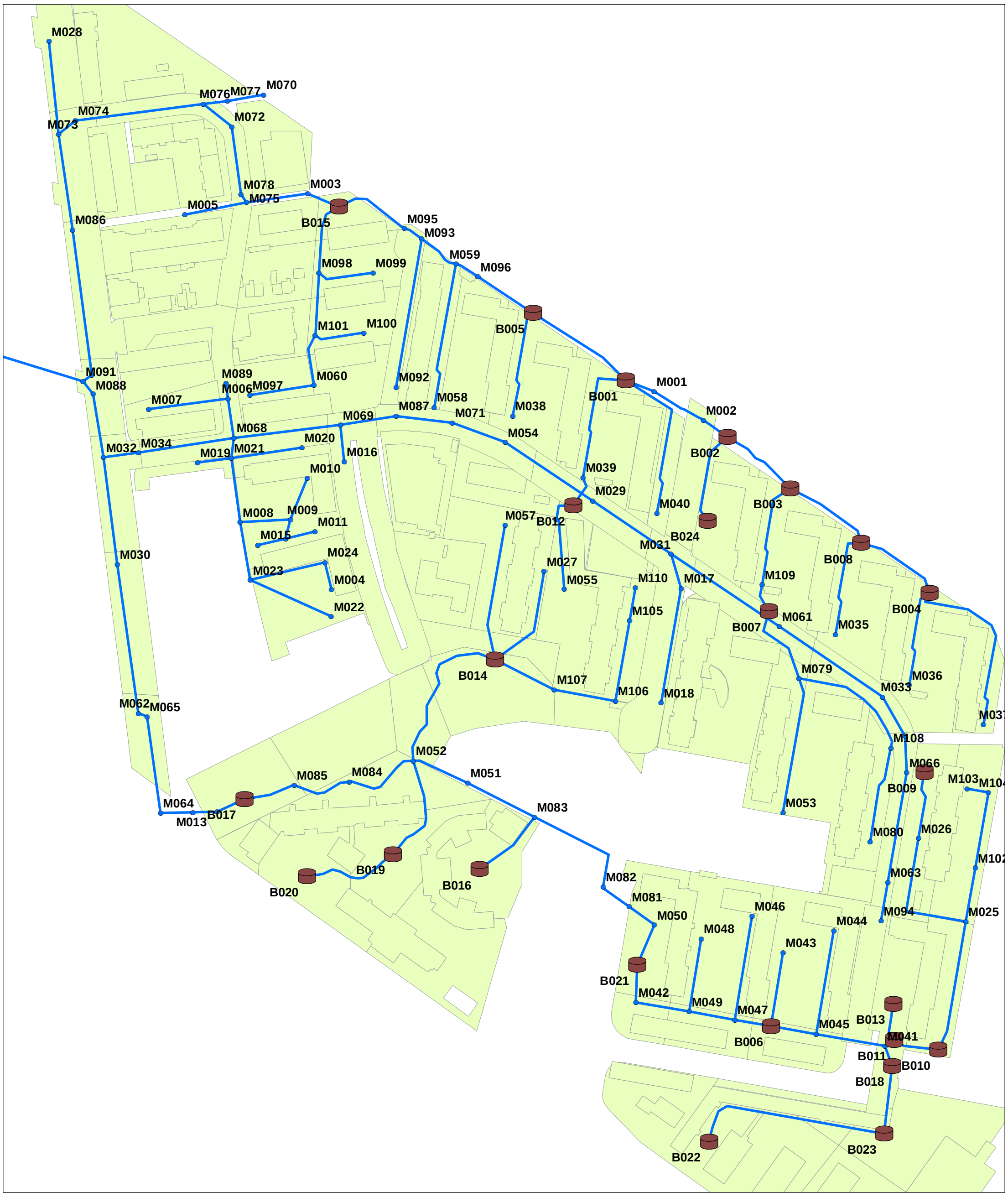


Figure B-2: The nodes in the open stormwater system. "B" indicates a basin and "M" indicates a manhole.

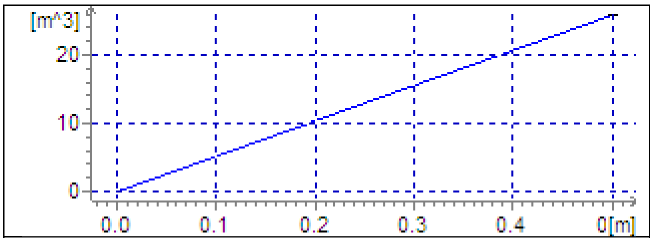
Table B-4: Shows the nodes in the open stormwater system.

Node ID	Geometry ID or Diameter (m)	Invert Level (m)	Ground Level (m)
B001	Pond XII a	12.30	12.90
B002	Pond X	13.60	14.10
B003	Pond IX c	14.00	14.40
B004	Pond IX a	14.50	14.80
B005	Pond XII b	12.10	12.40
B006	Pond XX	14.56	15.32
B007	CULVERT1	14.10	14.60
B008	Pond IX b	14.20	14.40
B009	Pond VIII	15.30	15.70
B010	Pond III	15.34	15.55
B011	Pond II	15.31	15.77
B012	CULVERT1	13.60	14.10
B013	Pond IV	15.40	15.60
B014	Pond XVIII	13.80	14.06
B015	Pond XIII	10.90	11.40
B016	Pond XIX	14.80	15.00
B017	Pond XV	12.70	13.10
B018	Pond I	15.16	15.66
B019	Pond XVI	14.00	14.30
B020	Pond XVII	14.30	14.50
B021	Pond VII b	14.57	15.10
B022	Pond V	15.80	16.20
B023	Pond VI	15.70	16.10
B024	Pond XI	14.00	14.40
M001	0.70	12.90	13.90
M002	0.70	13.10	14.10
M003	0.70	9.23	11.19
M004	0.40	13.45	13.60
M005	0.70	9.20	11.06
M006	0.80	9.09	12.01
M007	0.40	12.05	12.25
M008	0.50	12.40	12.65
M009	0.50	12.80	13.20
M010	0.40	13.22	13.40
M011	0.40	13.48	13.68
M012	2.45	12.13	13.41
M013	0.60	11.58	14.33
M014	0.40	13.40	13.71
M015	0.40	13.45	13.60
M016	0.80	9.20	12.80
M017	0.60	14.25	14.40
M018	0.60	14.75	14.90
M019	0.40	12.08	12.28
M020	0.40	12.35	12.50
M021	0.40	12.00	12.20
M022	0.40	13.70	13.81
M023	0.40	13.29	13.40
M024	0.40	13.30	13.55
M025	3.00	15.35	16.00

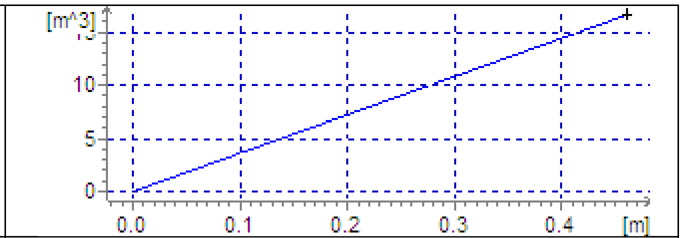
Node ID	Geometry ID or Diameter (m)	Invert Level (m)	Ground Level (m)
M026	0.40	15.40	16.10
M027	0.45	13.87	14.09
M028	0.70	8.84	10.74
M029	0.70	11.41	13.86
M030	0.80	8.71	12.65
M031	0.70	11.86	14.36
M032	1.00	8.52	11.87
M033	0.60	12.71	15.20
M034	0.80	8.98	11.96
M035	0.40	14.80	15.50
M036	0.40	15.08	15.78
M037	0.40	15.60	16.30
M038	0.40	13.09	13.70
M039	0.40	13.40	14.02
M040	0.40	13.90	14.64
M041	0.70	15.28	15.74
M042	1.50	14.76	15.37
M043	0.40	15.45	16.07
M044	0.40	15.60	16.22
M045	0.70	14.94	15.40
M046	0.40	15.45	16.07
M047	0.70	14.84	15.30
M048	0.40	15.45	16.07
M049	0.70	14.81	15.27
M050	1.50	14.70	15.37
M051	2.50	13.40	13.80
M052	2.50	13.30	13.60
M053	0.40	14.80	15.50
M054	0.70	10.43	13.38
M055	0.40	14.10	14.80
M056	0.40	13.90	14.60
M057	0.40	14.10	14.80
M058	0.40	12.87	13.48
M059	1.50	11.70	12.40
M060	0.40	12.30	12.95
M061	0.70	12.35	14.70
M062	0.80	8.86	13.75
M063	0.60	13.65	15.25
M064	0.60	10.01	14.00
M065	0.70	9.03	13.83
M066	0.60	12.93	15.30
M067	0.60	12.83	15.45
M068	0.70	9.03	12.33
M069	0.70	9.13	12.66
M070	0.60	8.94	11.40
M071	0.70	9.61	13.09
M072	0.70	9.04	10.85
M073	1.00	8.73	10.80
M074	0.80	8.87	10.78
M075	0.70	9.14	11.06
M076	0.80	8.83	10.86

Node ID	Geometry ID or Diameter (m)	Invert Level (m)	Ground Level (m)
M077	0.60	8.88	11.00
M078	0.70	9.12	10.92
M079	0.40	14.50	15.40
M080	0.40	15.20	15.90
M081	0.70	14.58	15.37
M082	0.70	14.50	15.00
M083	0.60	13.60	14.00
M084	2.50	13.20	13.40
M085	2.50	13.00	13.20
M086	1.00	8.58	10.92
M087	0.70	9.25	12.94
M088	1.00	8.41	11.38
M089	0.60	9.11	11.89
M090	0.80	8.38	11.31
M091	1.20	8.33	11.08
M092	0.80	12.35	12.50
M093	1.50	11.50	12.20
M094	0.60	13.75	15.31
M095	1.50	11.30	11.60
M096	1.50	11.90	12.10
M097	0.40	12.35	12.50
M098	0.40	11.60	12.30
M099	0.40	11.70	12.42
M100	0.40	12.30	12.93
M101	0.40	12.20	12.90
M102	2.50	15.00	15.60
M103	0.40	15.40	16.20
M104	2.50	15.10	15.65
M105	0.40	14.55	15.20
M106	0.40	14.50	15.18
M107	0.40	14.45	14.60
M108	0.40	15.10	15.30
M109	0.40	14.05	14.75
M110	0.40	14.58	15.23

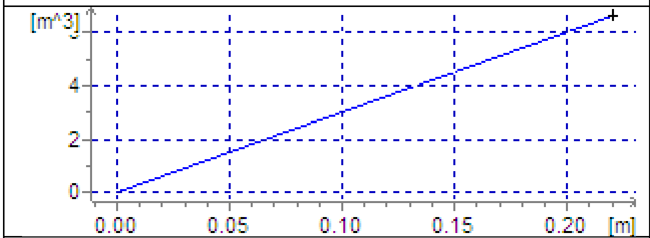
Table B-5: Shows the basins' geometry, in the form of depth-volume relationships.



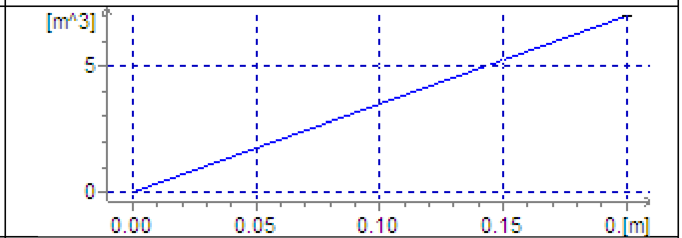
Pond I



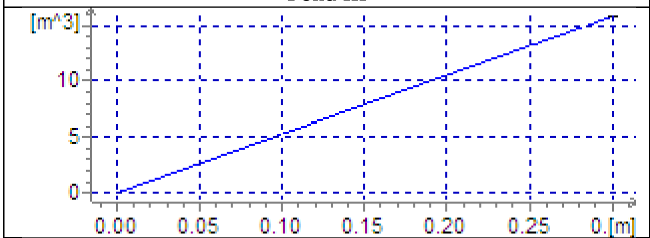
Pond II



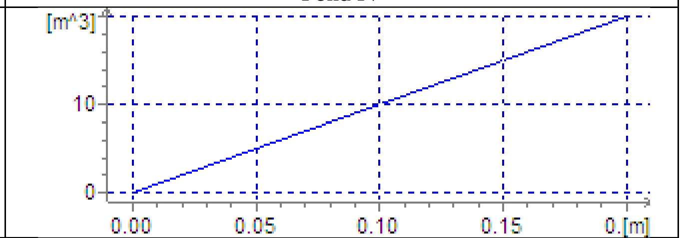
Pond III



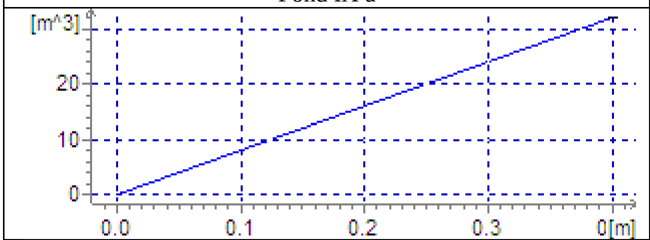
Pond IV



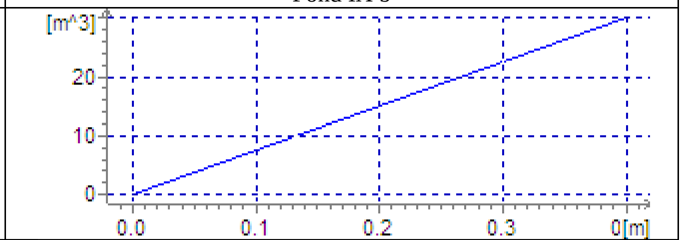
Pond IX a



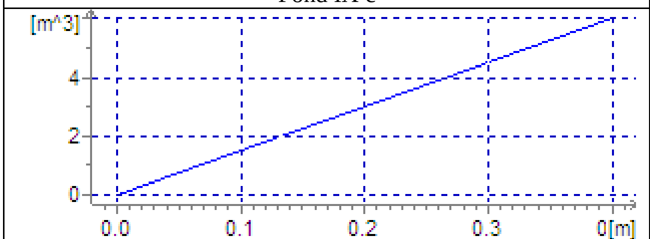
Pond IX b



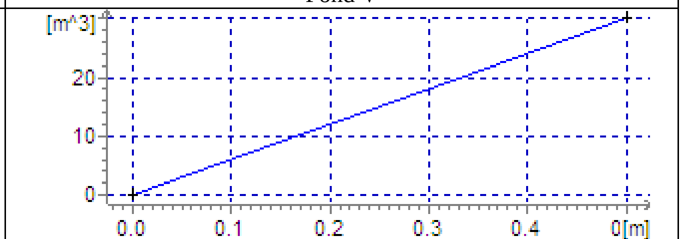
Pond IX c



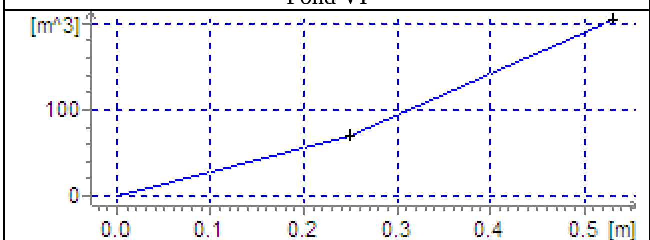
Pond V



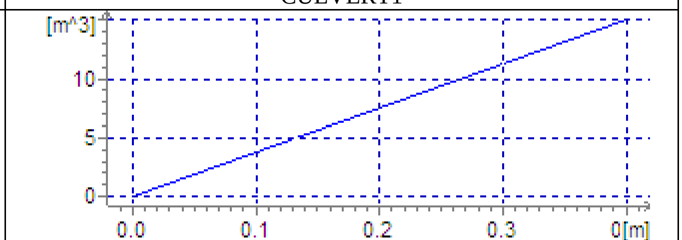
Pond VI



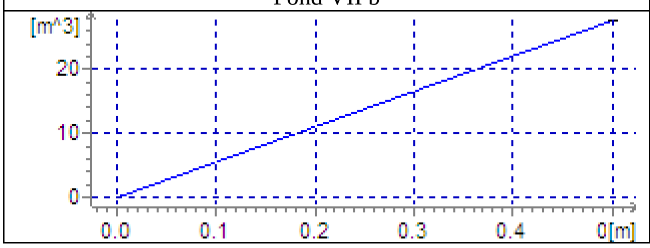
CULVERT1

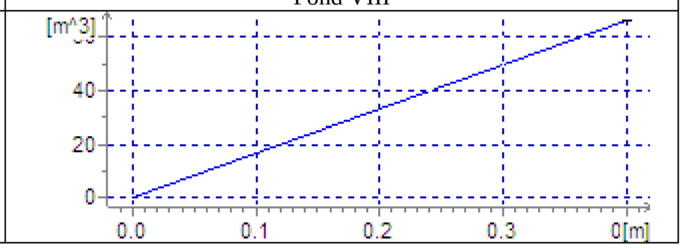


Pond VII b

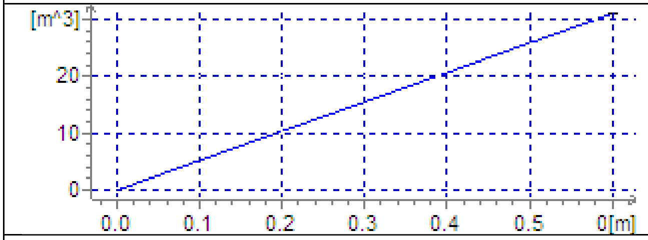


Pond VIII

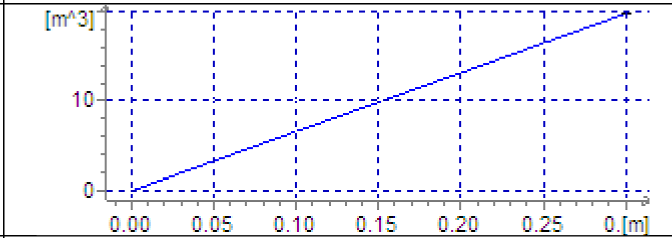




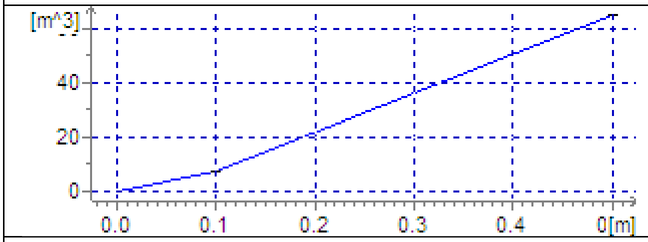
Pond X



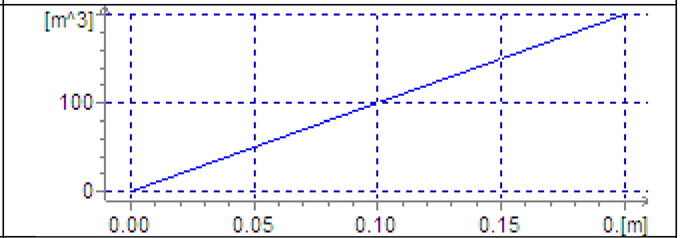
Pond XI



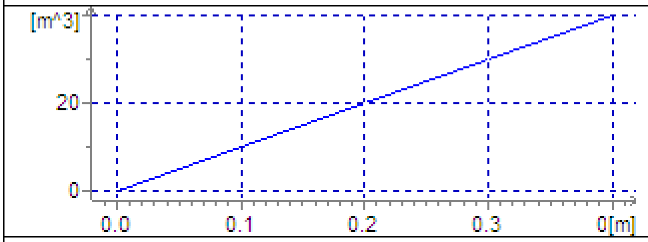
Pond XII a



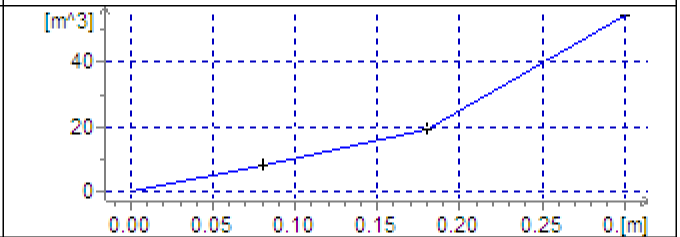
Pond XII b



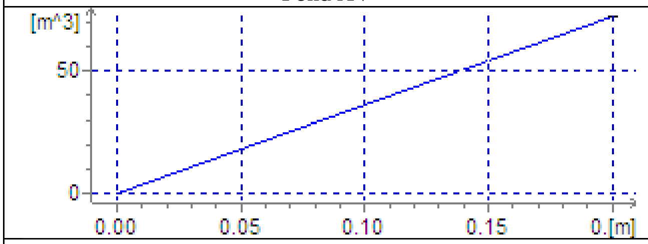
Pond XIII



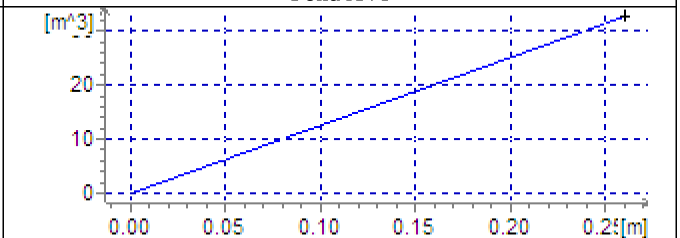
Pond XIX



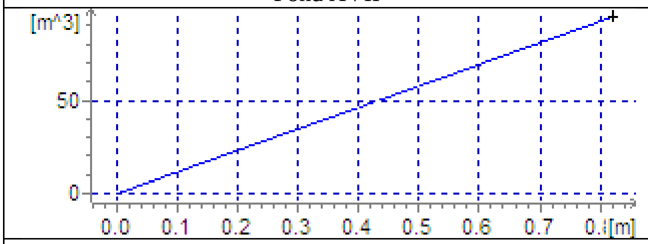
Pond XV



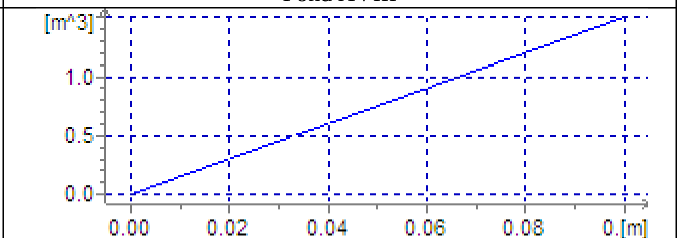
Pond XVI



Pond XVII



Pond XVIII



Pond XX

Pond XXI

Appendix C. Conventional Pipe Network Model Data

C-1. Hydraulic Model Data: Links

Figure C-1 shows the links in the pipe network model. The details are given in Table C-1.

Figure C-1: Shows the links in the pipe network model.

Table C-1: Details of the links of the system.

Link ID	Diameter (m)	Length (m)	Upstream Level (m)	Downstream Level (m)	Slope %
SW001	0.30	18.33	13.74	13.24	1.09
SW002	0.50	15.17	12.13	11.59	1.98
SW003	1.00	77.57	8.33	8.18	0.19
OP027	0.15	17.32	13.45	13.30	0.87
OP028	0.15	49.64	12.05	10.83	0.30
SW004	0.32	24.84	9.30	9.11	0.08
OP029	0.30	31.33	12.80	12.40	0.82
OP030	0.30	40.19	12.40	12.00	1.00
OP031	0.15	12.37	13.40	12.80	4.85
OP032	0.15	27.77	13.22	12.80	1.26
OP033	0.15	19.14	13.48	13.40	0.42
OP034	0.15	17.46	13.45	13.40	0.29
SW005	0.30	35.53	13.10	12.90	0.56
SW006	0.30	18.83	12.91	12.52	1.59
SW007	0.30	20.16	13.60	13.40	1.98
SW008	0.30	12.69	12.00	11.00	7.88
OP035	0.15	22.12	14.25	11.86	10.81
OP036	0.15	72.28	14.75	14.25	0.69
OP051	0.15	21.25	12.08	12.00	0.39
OP038	0.15	44.07	12.35	12.00	0.79
OP026	0.30	36.35	13.29	12.40	2.43
OP040	0.15	55.14	13.70	13.29	0.48
OP041	0.15	47.75	13.30	13.29	0.03
OP042	0.40	23.18	14.76	14.75	0.13
OP043	0.40	27.15	14.73	14.70	0.11
SW009	0.50	6.05	9.14	9.13	0.17
SW010	0.80	91.21	8.58	8.40	0.20
SW011	0.60	6.63	8.38	8.35	0.45
SW012	0.50	57.92	8.84	8.76	0.14
SW013	0.30	22.40	12.93	12.85	0.36
SW014	0.50	42.56	9.12	9.06	0.14
SW015	0.50	38.31	9.23	9.16	0.18
SW016	0.60	21.85	8.98	8.93	0.23
SW017	0.50	58.85	11.86	11.46	0.68
SW018	0.50	19.92	11.58	10.01	0.40
SW019	0.50	60.08	9.03	8.98	0.08
SW020	0.50	65.42	11.41	10.53	1.35
SW021	0.32	9.47	9.38	9.09	0.21
SW022	0.32	22.86	10.19	9.97	0.96
OP044	0.30	51.16	14.00	13.76	0.10
OP045	0.15	58.48	14.00	13.77	0.77

Link ID	Diameter (m)	Length (m)	Upstream Level (m)	Downstream Level (m)	Slope %
OP046	0.30	57.45	14.20	14.00	0.35
OP047	0.15	65.72	14.80	14.20	0.91
OP048	0.15	61.20	15.08	14.50	0.95
OP049	0.30	55.23	14.50	14.20	0.54
OP014	0.30	114.38	15.60	14.50	0.96
OP039	0.15	67.19	13.09	12.10	1.47
SW023	0.50	35.17	9.61	9.27	0.97
SW024	0.60	93.25	8.86	8.73	0.14
SW025	0.80	59.94	8.73	8.60	0.22
SW026	0.50	34.94	9.25	9.15	0.29
SW027	0.16	24.18	13.75	13.67	0.33
SW028	0.20	39.10	9.89	9.72	0.43
OP037	0.30	79.94	13.40	12.38	0.80
OP002	0.50	71.50	12.36	12.10	0.70
OP003	0.15	99.81	13.90	12.41	1.14
OP004	0.15	23.87	15.31	15.40	0.10
OP005	0.30	28.44	15.34	15.31	0.11
OP006	0.30	6.73	15.31	15.28	0.45
OP007	0.30	13.21	15.30	15.28	0.61
OP008	0.15	46.11	15.45	14.89	0.54
OP009	0.30	43.25	15.28	14.94	0.79
OP010	0.40	28.37	14.94	14.86	0.28
OP011	0.15	65.01	15.60	14.94	1.02
OP025	0.40	22.91	14.86	14.84	0.09
OP060	0.15	65.15	15.45	14.84	0.94
OP013	0.40	28.65	14.84	14.81	0.10
OP050	0.40	33.74	14.81	14.68	0.15
OP001	0.20	17.52	13.90	13.60	0.29
OP015	0.15	45.17	14.10	13.90	0.44
OP016	0.20	51.77	14.50	14.11	0.10
OP017	0.15	84.89	14.80	14.50	0.35
OP018	0.15	46.04	15.45	14.81	1.39
SW029	0.50	18.53	12.60	12.13	3.83
OP019	0.30	60.65	14.30	14.00	0.33
OP020	0.40	36.91	13.40	13.30	0.27
OP021	0.30	68.24	14.00	13.30	1.38
OP022	0.30	101.94	13.80	13.30	0.49
OP023	0.15	85.16	14.10	13.80	0.35
OP024	0.15	91.15	12.87	11.70	1.28
SW030	0.40	77.53	12.71	12.37	0.44
OP012	0.20	33.88	15.37	15.35	0.06
SW031	0.50	5.96	9.03	8.88	1.50
SW032	0.60	80.01	8.98	8.89	0.11

Link ID	Diameter (m)	Length (m)	Upstream Level (m)	Downstream Level (m)	Slope %
SW033	0.23	22.68	8.94	8.88	0.26
SW034	0.50	66.59	9.13	9.05	0.12
SW035	0.50	60.47	10.01	9.07	1.55
SW036	0.60	67.10	8.71	8.60	0.16
SW037	0.50	34.84	10.43	9.79	1.84
SW038	0.30	69.24	13.65	12.95	1.01
SW039	0.50	80.74	12.35	11.88	0.58
SW040	0.40	28.38	12.83	12.72	0.39
SW041	0.60	9.83	8.41	8.35	0.61
SW042	0.80	39.93	8.52	8.43	0.23
SW043	0.60	13.38	8.87	8.80	0.52
SW044	0.50	22.52	9.04	9.02	0.09
SW045	0.23	15.16	8.88	8.83	0.33
OP083	0.20	81.83	15.35	15.34	0.01
OP084	0.20	83.81	15.40	15.35	0.06
OP085	0.15	42.03	15.50	15.40	0.24
OP078	0.40	19.09	14.70	14.58	0.63
SW046	0.40	20.86	14.58	14.50	0.38
OP080	0.40	46.43	13.60	13.40	0.43
OP079	0.40	71.93	14.50	13.60	1.25
OP082	0.15	47.75	14.96	13.60	2.85
OP063	0.40	47.60	13.30	13.20	0.21
OP053	0.40	36.86	13.20	13.00	0.54
OP054	0.40	32.56	13.00	12.61	0.43
OP055	0.25	125.02	16.10	16.00	0.08
OP056	0.20	78.97	15.10	14.50	0.76
OP057	0.50	27.19	11.70	11.50	0.74
OP058	0.15	93.52	12.35	11.50	0.91
OP065	0.50	12.75	11.50	11.30	1.57
OP059	0.50	48.29	11.30	10.96	0.10
OP081	0.50	15.60	11.90	11.70	1.28
OP061	0.50	41.02	12.10	11.90	0.49
OP062	0.15	40.25	12.35	12.30	0.12
OP077	0.30	45.93	11.60	10.90	1.52
OP064	0.15	35.34	11.70	11.60	0.28
OP052	0.30	38.88	12.20	11.60	1.54
OP066	0.20	32.35	12.30	12.20	0.31
OP067	0.15	31.53	12.30	12.20	0.32
OP068	0.15	47.50	15.38	15.37	0.02
OP069	0.15	13.32	15.40	15.38	0.15
OP070	0.15	20.78	14.58	14.55	0.14
OP071	0.15	50.66	14.55	14.50	0.10
OP072	0.20	38.72	14.50	14.45	0.13

Link ID	Diameter (m)	Length (m)	Upstream Level (m)	Downstream Level (m)	Slope %
SW047	0.30	41.07	14.45	13.80	1.10
SW048	0.30	42.41	16.00	15.30	0.94
OP073	0.15	60.43	15.20	15.10	0.17
OP074	0.30	18.04	14.10	14.05	1.94
OP075	0.30	66.66	14.05	14.00	0.08
OP076	0.15	67.62	13.87	13.80	0.10
SW049	0.50	20.95	10.91	9.23	8.93

C-2. Hydraulic Model Data: Nodes

The layout of the hydraulic network and the locations of nodes are shown in Figure C-2, the details are given in Table C-2.

Figure C-2: The pipe network nodes.

Table C-2: Details of the nodes in the system

Node ID	Diameter (m)	Invert Level (m)	Ground Level (m)
M001	0.70	12.30	13.60
M002	0.70	12.90	14.20
M003	0.70	13.10	14.40
M004	0.70	9.23	11.19
M005	0.40	13.45	14.10
M006	0.70	9.20	11.06
M007	0.80	9.29	12.40
M008	0.40	12.05	12.55
M009	0.50	12.40	13.10
M010	0.50	12.80	13.50
M011	0.40	13.22	13.70
M012	0.40	13.48	13.98
M013	0.50	13.60	14.60
M014	0.60	12.13	13.71
M015	0.60	11.58	14.33
M016	0.40	13.40	14.01
M017	0.40	13.45	13.90
M018	0.80	9.20	12.80
M019	0.60	14.25	14.90
M020	0.60	14.75	15.30
M021	0.40	12.08	12.58
M022	0.40	12.35	12.90
M023	0.40	12.00	12.80
M024	0.40	13.70	14.30
M025	0.40	13.29	14.10
M026	0.40	13.30	13.85
M027	0.50	15.35	16.30
M028	0.40	15.40	16.40
M029	0.45	13.87	14.39
M030	0.70	8.84	10.74
M031	0.70	11.41	13.86
M032	0.80	8.71	12.65
M033	0.70	11.86	14.36
M034	1.00	8.52	11.87
M035	0.60	12.71	15.20
M036	0.80	8.98	11.96
M037	0.50	14.00	14.70
M038	0.50	14.50	15.10
M039	0.70	12.10	13.00
M040	0.40	14.80	15.80
M041	0.40	15.08	16.08

Node ID	Diameter (m)	Invert Level (m)	Ground Level (m)
M042	0.40	15.60	16.60
M043	0.40	13.09	14.00
M044	0.50	13.40	14.32
M045	0.40	13.90	14.94
M046	0.60	15.28	16.04
M047	0.60	14.67	15.58
M048	0.40	15.45	16.37
M049	0.60	14.80	15.80
M050	0.40	15.60	16.52
M051	0.60	14.94	15.70
M052	0.40	15.45	16.37
M053	0.60	14.84	15.60
M054	0.40	15.45	16.37
M055	0.60	14.81	15.57
M056	0.60	14.70	15.67
M057	0.60	13.40	14.40
M058	0.60	13.30	14.20
M059	0.50	13.90	14.90
M060	0.40	14.80	15.80
M061	0.70	10.43	13.38
M062	0.50	14.20	15.00
M063	0.40	15.50	16.20
M064	0.50	15.34	16.10
M065	0.60	15.31	16.07
M066	0.50	13.60	14.60
M067	0.40	14.10	15.10
M068	0.40	13.90	14.90
M069	0.40	14.10	15.10
M070	0.40	12.87	13.78
M071	0.70	11.70	12.70
M072	0.40	12.30	13.25
M073	0.70	12.35	14.70
M074	0.80	8.86	13.75
M075	0.40	15.40	15.90
M076	0.50	13.80	14.70
M077	0.60	13.65	15.25
M078	0.60	10.01	14.00
M079	0.70	9.03	13.83
M080	0.60	12.93	15.30
M081	0.60	12.83	15.45
M082	0.70	9.03	12.33
M083	0.70	9.13	12.66
M084	0.60	8.94	11.40

Node ID	Diameter (m)	Invert Level (m)	Ground Level (m)
M085	0.70	9.61	13.09
M086	0.70	9.04	10.85
M087	1.00	8.73	10.80
M088	0.80	8.87	10.78
M089	0.70	9.14	11.06
M090	0.80	8.83	10.86
M091	0.60	8.88	11.00
M092	0.70	9.12	10.92
M093	0.70	10.90	12.30
M094	0.40	14.50	15.70
M095	0.40	15.20	16.20
M096	0.60	14.58	15.67
M097	0.60	14.50	15.30
M098	0.60	13.60	14.60
M099	0.50	14.80	15.50
M100	0.60	13.20	14.00
M101	0.60	13.00	13.80
M102	1.00	8.58	10.92
M103	0.70	9.25	12.94
M104	1.00	8.41	11.38
M105	0.60	9.11	12.19
M106	0.80	8.38	11.31
M107	1.20	8.33	11.08
M108	0.40	12.35	12.80
M109	0.70	11.50	12.50
M110	0.60	13.75	15.31
M111	0.60	12.60	13.70
M112	0.40	15.30	16.30
M113	0.70	11.30	12.30
M114	0.70	11.90	12.90
M115	0.40	12.35	13.00
M116	0.50	11.60	12.60
M117	0.40	11.70	12.72
M118	0.40	12.30	13.23
M119	0.50	12.20	13.20
M120	0.40	15.35	16.50
M121	0.40	15.40	16.50
M122	0.40	15.35	16.50
M123	0.40	14.55	15.50
M124	0.40	14.50	15.48
M125	0.40	14.45	15.10
M126	0.50	14.00	14.90
M127	0.50	14.17	14.97

Node ID	Diameter (m)	Invert Level (m)	Ground Level (m)
M128	0.60	14.65	15.70
M129	0.40	15.10	15.75
M130	0.50	14.05	15.05
M131	0.40	14.58	15.53
M132	0.40	16.10	16.90
M133	0.40	16.00	16.90
M134	0.40	14.00	14.90

Appendix D. Hydrological Model Data

The catchments are shown in Figure D-1. Details are given in Table D-1, and the impervious

Figure D-1: The hydrological model catchments.

Table D-1: The details of the catchments.

Catchment ID	Area (m^2)	Description	Impervious area %	Time of concentration (minutes)
1	1,478.2	RES_A	7.88	1
2	1,358.1	RES_A	7.88	1
3	1,332.5	RES_A	7.88	1
4	2,520.2	RES_A	7.88	2
5	800.4	RES_A	7.88	1
6	2,025.4	RES_A	7.88	2
7	3,242.6	HARD_A	7.20	2
8	3,425.3	RES_A	5.00	3
9	1,374.6	RES_A	5.00	2
10	1,976.2	HARD_A	50.00	3
11	1,442.2	RES_A	5.00	1
12	1,302.8	RES_A	5.00	1
13	2,569.5	RES_A	5.00	1
14	1,516.9	RES_A	5.00	1
15	661.3	RES_A	5.00	1
16	611.1	RES_A	5.00	1
17	1,506.4	RES_A	5.00	1
18	472.9	RES_B	3.00	3
19	995.7	HARD_A	50.00	1
20	2,848.6	HARD_A	22.50	3
21	2,558.9	RES_A	5.00	1
22	5,278.0	HARD_A	7.20	2
23	1,409.0	TILERF_A	53.55	1
24	1,532.7	RES_A	7.88	1
25	2,682.0	RES_A	7.88	2
26	2,843.7	RES_A	7.88	2
27	2,857.6	RES_A	4.50	2
28	3,501.7	RES_A	5.00	3
29	8,669.9	HARD_A	0.00	6
30	2,284.1	RES_B	2.70	1
31	2,292.5	PARK_A	2.70	2
32	3,042.8	PARK_A	2.70	2
33	1,589.2	PARK_A	2.70	1
34	2,529.7	PARK_A	2.70	1
35	3,229.3	RES_B	2.70	1
36	1,645.0	RES_A	5.00	3
37	1,279.3	RES_B	2.70	1
38	1,515.0	HARD_A	78.75	2
39	216.0	GRNRF_E	10.50	1
40	343.6	PARK_A	2.70	1
41	2,463.2	HARD_A	22.50	1
42	771.5	RES_A	7.88	1
43	1,337.4	RES_A	7.88	1
44	857.7	RES_A	7.88	1
45	255.3	HARD_A	45.00	1
46	1,315.9	RES_A	4.50	2
47	2,457.3	HARD_A	22.50	1
48	1,453.0	RES_A	7.88	2

Catchment ID	Area (m^2)	Description	Impervious area %	Time of concentration (minutes)
49	118.6	RES_A	5.00	0
50	2,028.1	RES_A	4.50	2
51	3,102.3	RES_A	4.50	1
52	947.9	RES_A	4.50	1
53	2,179.1	RES_A	4.50	1
54	2,690.2	RES_A	4.50	2
55	651.3	RES_A	4.50	0
56	1,065.9	RES_A	4.50	1
57	3,575.4	RES_A	4.50	1
58	184.8	RES_A	7.88	1
59	1,372.9	RES_A	5.00	2
60	2,713.1	RES_A	7.88	2
61	2,711.6	RES_A	7.88	2
62	1,857.9	RES_A	7.88	1
63	1,558.8	RES_A	4.50	2
64	876.8	TILERF_A	93.71	1
65	814.3	RES_A	5.00	1
66	2,455.3	RES_A	5.00	1
67	1,550.7	HARD_A	50.00	2
68	1,338.1	HARD_A	50.00	1
69	820.3	RES_A	7.88	1
70	3,086.2	RES_A	5.00	2
71	1,341.5	RES_A	5.00	2
72	1,119.0	TILERF_A	93.71	2
73	956.6	RES_A	7.88	1
74	44.5	GRNRF_E	16.54	0
75	686.2	TILERF_A	53.55	1
76	1,134.6	TILERF_A	93.71	2
77	24.6	GRNRF_E	10.50	0
78	39.8	TILERF_A	93.71	0
79	438.2	TILERF_A	53.55	1
80	655.5	GRNRF_I	3.15	1
81	7,379.6	GRNRF_I	3.15	8
82	44.6	GRNRF_E	10.50	0
83	22.5	TILERF_A	93.71	1
84	273.5	TILERF_A	59.50	1
85	475.8	TILERF_A	53.55	1
86	44.5	GRNRF_E	16.54	0
87	898.8	TILERF_A	53.55	3
88	49.4	TILERF_A	93.71	1
89	1,596.8	HARD_A	50.00	1
90	4,399.7	HARD_A	50.00	3
91	2,833.8	HARD_A	50.00	3
92	1,548.3	HARD_A	78.75	2
93	3,957.0	HARD_A	50.00	2
94	1,886.8	HARD_A	50.00	2
95	1,549.6	HARD_A	50.00	1
96	2,621.6	HARD_A	50.00	3
97	1,252.4	TILERF_A	53.55	2
98	42.9	TILERF_A	93.71	1

Catchment ID	Area (m^2)	Description	Impervious area %	Time of concentration (minutes)
99	1,300.4	TILERF_A	53.55	2
100	311.4	TILERF_A	53.55	1
101	304.0	TILERF_A	53.55	1
102	345.9	TILERF_A	53.55	1
103	476.1	TILERF_A	93.71	1
104	42.9	TILERF_A	93.71	1
105	44.4	GRNRF_E	16.54	0
106	44.7	GRNRF_E	16.54	0
107	490.2	TILERF_A	93.71	1
108	713.0	TILERF_A	93.71	1
109	394.3	TILERF_A	59.50	1
110	1,121.0	TILERF_A	93.71	1
111	1,037.7	TILERF_A	59.50	2
112	1,139.3	TILERF_A	93.71	2
113	1,124.3	TILERF_A	53.55	1
114	56.8	TILERF_A	93.71	0
115	1,120.5	RES_A	5.00	1
116	1,211.6	RES_A	5.00	1
117	962.7	RES_A	4.50	2
118	523.4	HARD_A	50.00	1
119	1,884.8	RES_A	4.50	3
120	16.4	GRNRF_E	10.50	2
121	69.3	TILERF_A	59.50	1
122	965.4	TILERF_A	59.50	1
123	29.5	TILERF_A	59.50	0
124	16.5	TILERF_A	59.50	1
125	695.7	TILERF_A	59.50	3
126	298.4	TILERF_A	59.50	0
127	38.6	GRNRF_E	9.45	1
128	539.8	GRNRF_I	3.50	1
129	528.9	TILERF_A	59.50	2
130	1,036.5	TILERF_A	59.50	1
131	1,581.5	TILERF_A	59.50	3
132	1,144.6	TILERF_A	93.71	2
133	1,432.7	TILERF_A	59.50	2
134	1,025.2	TILERF_A	53.55	1
135	44.1	GRNRF_E	16.54	0
136	836.6	TILERF_A	53.55	5
137	1,143.8	TILERF_A	93.71	2
138	322.7	TILERF_A	93.71	1
139	394.8	TILERF_A	59.50	1
140	1,115.6	TILERF_A	53.55	1
141	29.7	TILERF_A	93.71	0
142	1,123.6	TILERF_A	93.71	2
143	1,039.1	TILERF_A	53.55	1
144	1,120.7	TILERF_A	53.55	1
145	1,386.8	GRNRF_I	3.15	3
146	37.8	TILERF_A	93.71	0
147	22.9	TILERF_A	59.50	1
148	687.0	TILERF_A	59.50	1

Catchment ID	Area (m^2)	Description	Impervious area %	Time of concentration (minutes)
149	124.0	TILERF_A	59.50	1
150	517.6	TILERF_A	53.55	3
151	599.5	TILERF_A	59.50	1
152	443.6	TILERF_A	59.50	1
153	855.8	TILERF_A	59.50	1
154	1,107.3	TILERF_A	59.50	1
155	1,131.2	TILERF_A	93.71	2
156	41.8	GRNRF_E	10.50	0
157	479.0	TILERF_A	93.71	1
158	49.1	GRNRF_E	9.45	1
159	2,406.1	RES_A	4.50	1
160	29.2	TILERF_A	59.50	1
161	281.8	TILERF_A	59.50	2
162	22.8	TILERF_A	59.50	1
163	959.9	TILERF_A	59.50	1
164	34.6	TILERF_A	59.50	1
165	1,206.8	TILERF_A	59.50	1
166	299.1	TILERF_A	59.50	1
167	610.9	TILERF_A	59.50	1
168	51.9	TILERF_A	59.50	1
169	1,555.4	TILERF_A	53.55	2
170	378.8	TILERF_A	59.50	0
171	28.4	GRNRF_E	10.50	1
172	849.6	RES_A	5.00	2
173	1,214.4	HARD_A	50.00	1
174	1,970.2	HARD_A	50.00	2
175	1,735.9	HARD_A	50.00	1
176	1,112.9	HARD_A	50.00	0
177	2,075.3	HARD_A	50.00	2
178	1,654.2	HARD_A	50.00	2
179	2,143.2	HARD_A	50.00	1

Table D-2: The surface type corresponding to the codes in Table D-1.

Surface type	Code
Tile roofs	TILERF_A
Concrete and asphalt	HARD_A
Residential areas with veg. and concrete	RES_A
Sand covered park areas	RES_B
Extensive green roofs	GRNRF_E
Lawns	PARK_A
Intensive green roofs	GRNRF_I

Appendix E. Scientific Article

Comparison between an open stormwater system and a conventional pipe system using MIKE URBAN

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Abstract

The open stormwater systems are becoming increasingly acceptable solutions to handle the stormwater in urban areas in a sustainable way over the traditional stormwater sewer system. An existing open stormwater system in an urban area in southern Sweden is compared to an equivalent hypothetical conventional pipe network. Both systems are modeled in MIKE URBAN using the MOUSE engine, consisting of a rainfall-runoff model and a hydraulic network model. The comparison is based on discharge hydrographs and node flooding according to two rainfall scenarios simulating current conditions and synthetic extreme storm events. The open stormwater system demonstrated a longer lag time and a lower discharge rate, about 50% of the one simulated in the pipe network. The flooding was less severe in the open stormwater system under more extreme conditions.

Keywords: Hydraulics; hydrology; open stormwater systems; urban; modeling; MIKE URBAN; MOUSE

INTRODUCTION

The open stormwater systems are becoming more acceptable as a solution to handle the stormwater in urban areas in a sustainable way over the traditional stormwater sewer system (Stahre, 2008).

An open stormwater system can consist of any combination of facilities that contribute in reducing the flow rate and volume. This is achieved by infiltration, storage, detention and slow transport of the stormwater. Examples of these facilities are ponds of various types, vegetated buildings roofs, open channels, swales, constructed wetlands and existing natural landscape.

To justify the implementation of open stormwater systems in the existing urban areas or future developments, it is of interest to be able to model such systems to evaluate their performance.

This article discusses a comparison between an open stormwater system and an equivalent conventional pipe network using a computer model. The modeling is done using the water modeling software MIKE URBAN, which is typically used for modeling distribution networks and collection systems in urban areas. The comparison is based on the discharge rate, the lag time and flooding.

The model of the open stormwater system is based on an existing in Augustenborg Eco-City, an urban residential area in the city of Malmö southern Sweden, while the pipe network model is based on a hypothetical comparable system.

The objective is to compare the hydraulic performance of the open stormwater system in Augustenborg to a conventional stormwater sewer network based on the discharge hydrograph and the capacity in extreme storm events.

METHODS

Study Site

The study site is an urban residential area in the city of Malmö in southern Sweden called Augustenborg Eco-City. It consists mainly of apartment blocks covering an area about 32 hectares.

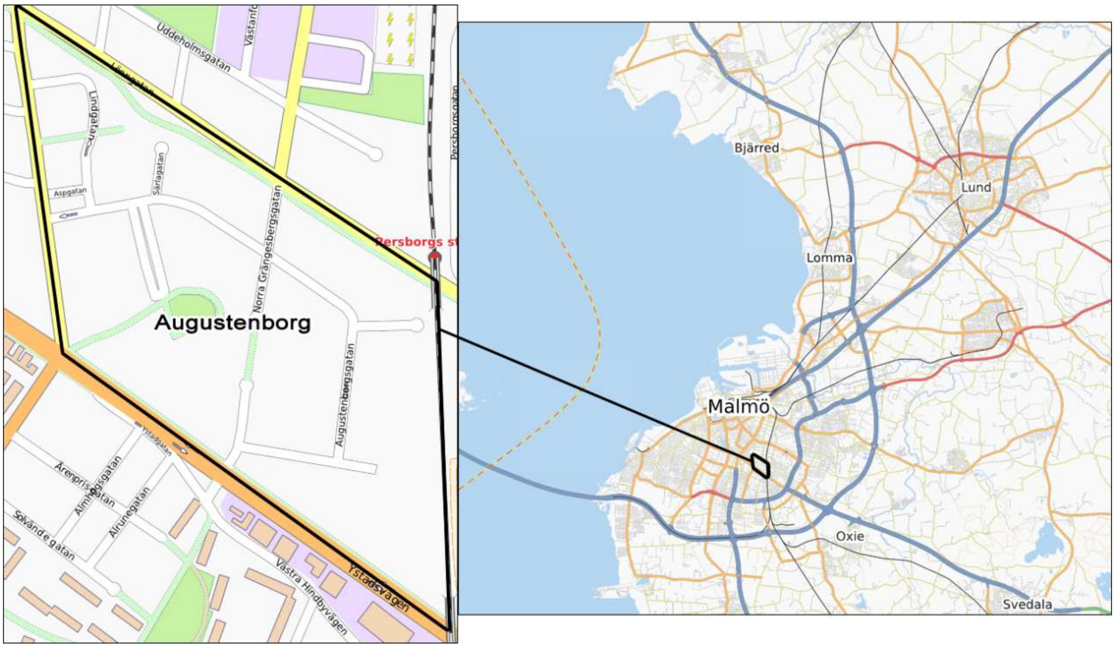
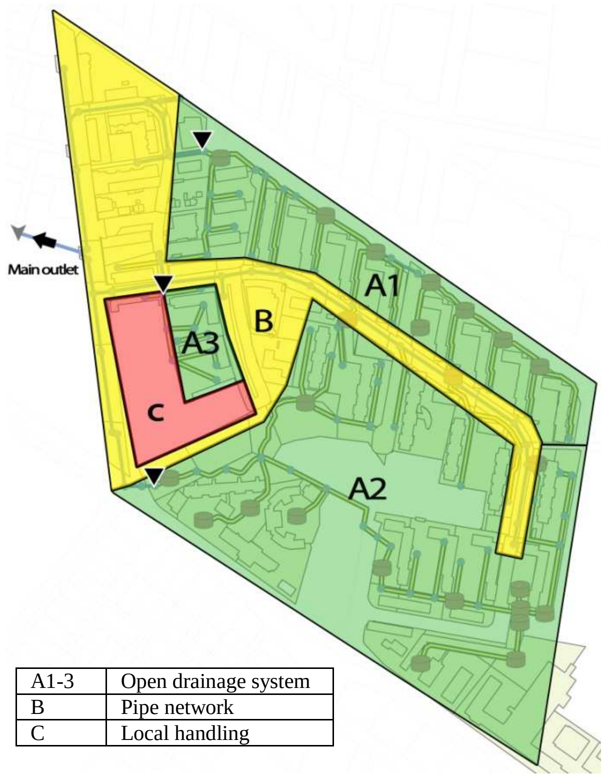


Figure 2: A map over Malmö showing Augustenborg Eco-City (OpenStreetMap, 2010).

In addition to the residential buildings there is also a number of public buildings and parks. The settlement is inhabited by around 3000 people (eco-guide.net, 2006). Figure 2 shows the location of Augustenborg in Malmö city.

Originally, the stormwater was handled by a combined sewer system, but was later replaced by an open stormwater system in most parts of the area, joined together with a separate stormwater sewer which ends up in the main city sewer network. The stormwater is handled locally in a small area in Augustenborg. The different handling methods are shown in Figure 3.



A1-3	Open drainage system
B	Pipe network
C	Local handling

Figure 3: Stormwater handling methods in Augustenborg. The triangle symbols show the connection points of the open drainage system to the stormwater sewers.

Modeling

Two models were built using the MOUSE engine MIKE URBAN, the open stormwater system model and the conventional pipe network. Each model consists of a rainfall-runoff model and a hydraulic network model. The rainfall-runoff model is common to the open stormwater system and the conventional system.

The Rainfall-Runoff Model

The rainfall-runoff model consists of catchments, which represent the different surfaces that contribute to the runoff. These are defined by the area and geographical location.

The catchments were categorized into different categories which have different imperviousness values, these include tile roofs, concrete and asphalt, green roofs and lawns.

The time-area method is used to simulate the runoff from the rainfall, which requires knowledge about the impervious area percentage, hydrological reduction factor, initial loss, the time of concentration and the time-area curve for the catchments.

The Hydraulic Network Model

The hydraulic network model is built by defining the geometric and hydraulic properties of the nodes and links in the system.

The network layout for the open stormwater system was based on the site plan of Augustenborg and the sewer network maps. The dimensions of the links and nodes were either based on the detail maps or measured directly on the site. The elevations of the network grid points were taken from available survey data and the missing elevations were interpolated.

The hydraulic loss in the links was simulated using Manning roughness coefficient. The Manning coefficients for different types of channels were based on Table 3. For channels of composite cross-sections, consisting of more than one material, an average coefficient was estimated.

Table 3: Example values of roughness coefficient (M) based on values given by (French, 2007).

Material	Manning (*M*)
Concrete (rough)	68
Concrete (smooth)	85
Plastic	80
Grass (lawn)	20
Cement mortar (neat)	90
Masonry	40
Rubble	30

The infiltration was simulated as a constant negative load assigned to swales and grass channels in parks where infiltration is expected. The infiltration rate was set to 8.0 *mm/hr* (VAV, 1983).

The inlet points and junctions in the open drainage system were modeled as circular nodes of diameters equal to the width of the largest connected link. The geometry of the ponds (basins) was based on the site plan and field measurements. Engelund formula (MOUSE classic) was chosen to compute the head loss in the nodes.

The layout of the hydraulic network is shown in Figure 5-4.

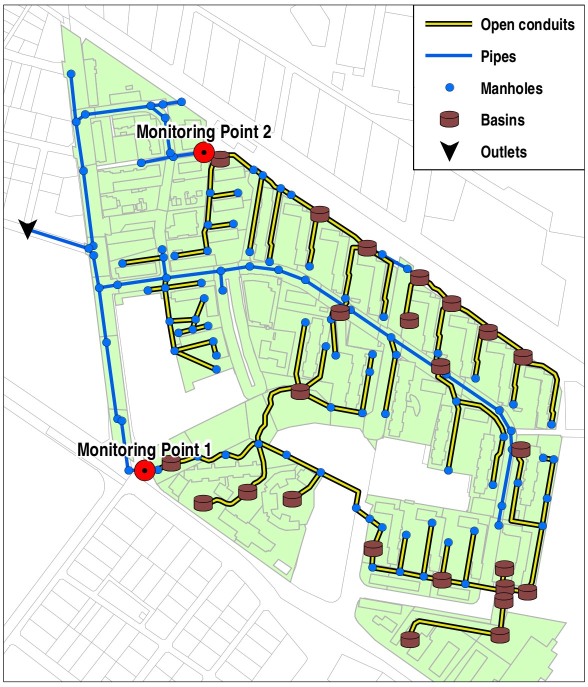


Figure 4: The layout of the hydraulic network of Augustenborg in MIKE URBAN and the location of monitoring points.

The catchment connections to the nodes were made to reasonably agree with the conditions in reality, and the water is assumed to travel from the centroid of the catchment area to the center of the node during a time equal to the time of concentration.

The same layout of the open stormwater system was used to model the hydraulic network of the conventional system by replacing the open drains with pipes, ponds with circular manholes and no infiltration was considered.

Model Calibration

The model was calibrated against measured flows at two monitoring points; those are shown in Figure 5-4 labeled monitoring point 1 and 2. There was one significant rainfall event during which the flow was measured in the monitoring points.

Simulation Scenarios

Two scenarios are simulated in this study; the first one is intended to compare the discharge rate and the lag time in the two systems. The second scenario compares the behavior of both systems in extreme weather when flooding is expected.

In the first scenario the current weather conditions are simulated in both models. The rainfall event used in this scenario was the maximum daily precipitation during the years 2007-2008. The rainfall depth during this event is shown in Figure 5.

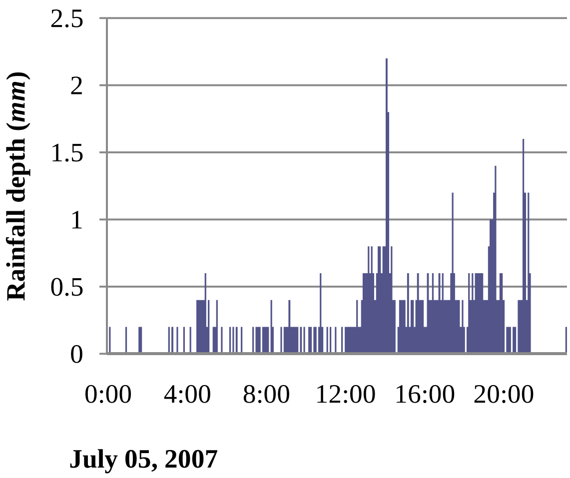


Figure 5: The rainfall event used in scenario 1.

For the second scenario, synthetic extreme storm events are computed using Dahlström formula (Svenskt Vatten, 2004). The rainfall duration-intensity curves were drawn for Malmö city for different return periods; these are shown in Figure 6-7.

Storms with different duration (10, 20 and 30 minutes) were used in the simulation.

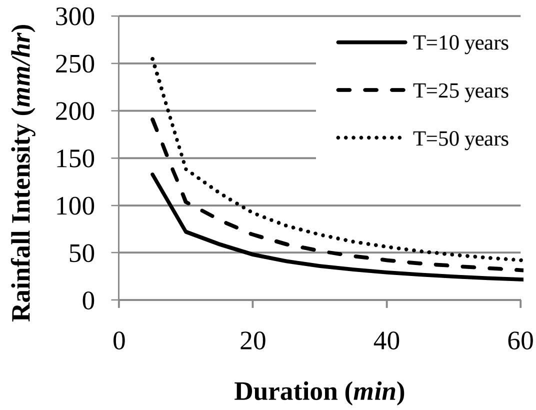


Figure 6: Rainfall intensity curves calculated using Dahlström formula for different return periods (T) (Svenskt Vatten, 2004).

RESULTS

Scenario 1

At monitoring point 2 it was noticed that the discharge rate in the pipe network was approximately double the discharge rate in the open drainage system as shown in Figure 6-5. This shows the reduction in discharge and the longer lag time resulted from the open solutions. The peak timing difference between the two systems was about 5 to 10 minutes, the pipe network being the faster one.

The pipe network response to smaller precipitation is also evident, which is clear at the beginning of the simulation. This is not seen in the open drainage system mainly because of the infiltration in the links and the volume stored in the ponds.

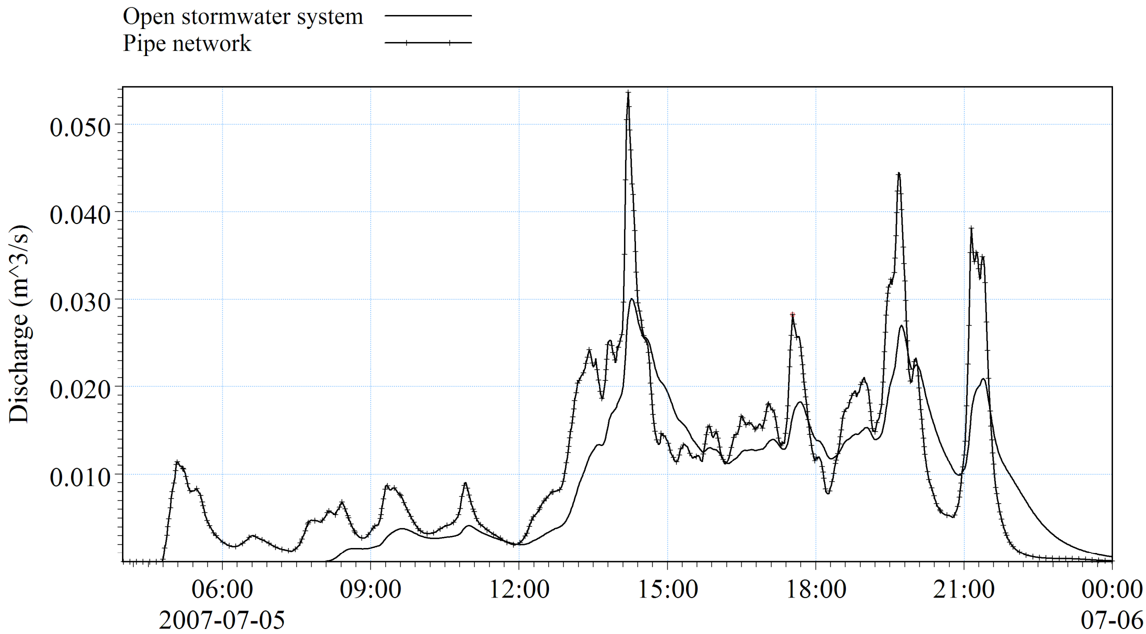


Figure 7: The simulated discharge at monitoring point 2 simulated for the open stormwater system compared to the discharge from the pipe network system at the same location..

The discharge in the open drainage system at monitoring point 1 shows a more pronounced effect of the open stormwater solutions. The smoother and more evened out discharge hydrograph shown in Figure 6-6 indicates longer lag time because of the larger number of ponds in this area. The peaks appear about 40 minutes later in the open stormwater system compared to the pipe network.

At this monitoring point less water volume is leaving the open system compared to the conventional system due to the infiltration along the water path compared to the pipe system.

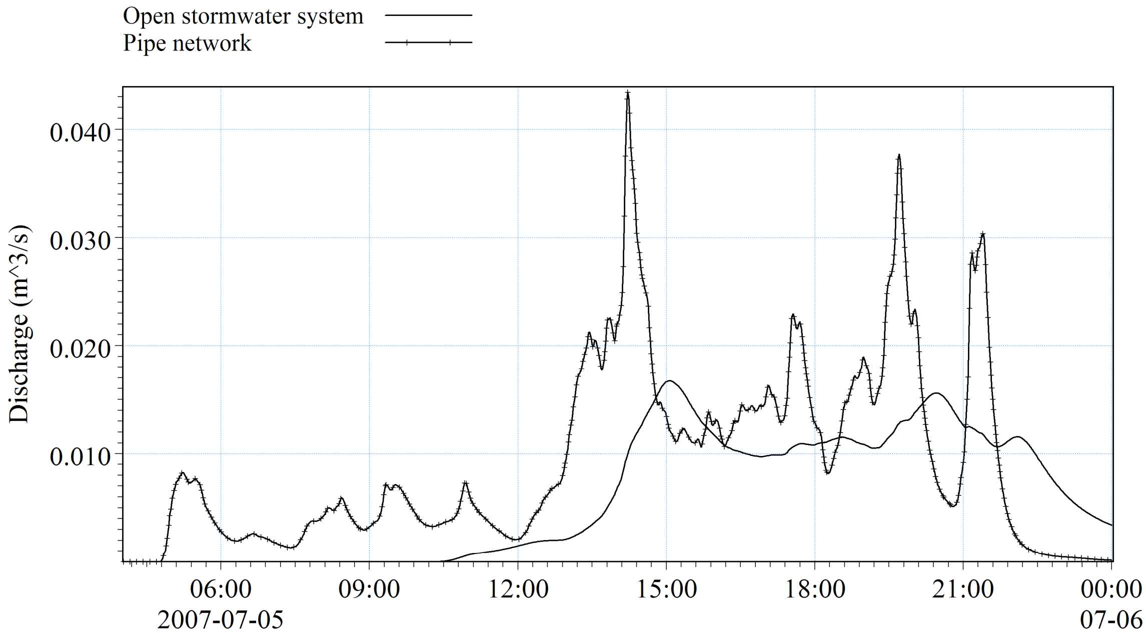


Figure 8: The simulated discharge at monitoring point 1 is plotted for both systems. The open drainage system exhibits a more evened out discharge hydrograph.

Scenario 2

During the 50 years storm event of 10 minutes duration (140 *mm/hr* intensity), it was noticed that there is flooding at some inlet points and shallow nodes in both systems.

The duration of the same 50 years storm event was extended to 20 minutes to investigate the behavior of both systems under more severe conditions. The 140 *mm/hr* intensity rain caused more flooding in both systems, but considerably higher in the pipe network.

The flooding in the open system occurred at inlet nodes and extended to the shallow nodes further in the system. The flooded nodes are shown in Figure 6-10. The flooding is calculated as the (water level minus ground level) in the nodes and the values are the maximum for the whole simulation period. The maximum flooding height was about 0.34m above ground level.

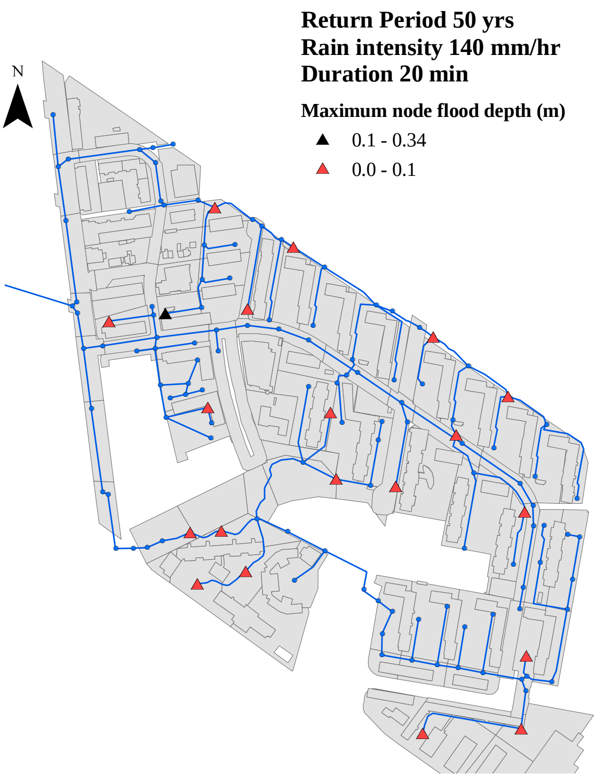


Figure 9: Maximum node flooding in the open stormwater system caused by the 140 mm/hr intensity rainfall for the duration of 20 minutes.

The situation was different in the pipe network, as the flooding was more extensive and of higher depth. The maximum flooding height was about 2 *m* as shown in Figure 6-11.

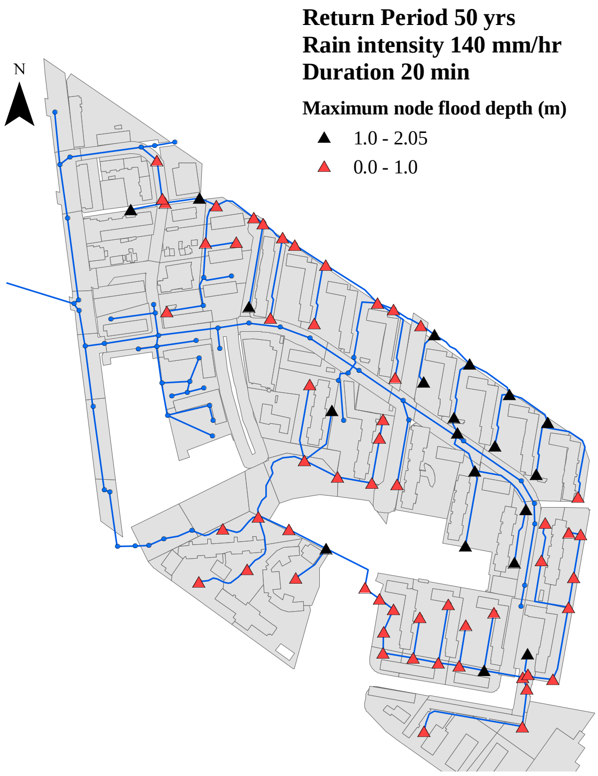


Figure 10: The maximum node flooding in the pipe network system caused by the 140 mm/hr intensity rainfall for the duration of 20 minutes.

DISCUSSION

Scenario 1

The discharge hydrographs observed at monitoring point 2 shown in Figure 6-5 show that the discharge rate in the pipe system is about the double of that in the open system. This difference is mainly attributed to the larger volume of the open system given by the larger links cross-sections area and the detention caused by the ponds.

The other factor is the lower roughness coefficients of the canals in the open system, which helps retard the flow and delay the peak timing.

These effects are more clearly seen in monitoring point 1 with even larger difference in the discharge and peaks timing, because of the larger volumes of the network elements causing more retention. In addition there is the local backwater effect in some locations due to mild slopes. The infiltration in ponds and canals contributes to the reduction of the total water volume.

One can argue about the fairness of this comparison regarding the way both systems are built. Since in pipe networks the designer would want to take the shortest paths, the fewest bents and turns, and more suitable slopes. While in open drainage systems for stormwater purposes the opposite is preferred, and here both systems are built with similar configurations.

However, a more careful design of the pipe network will most probably result in higher efficiency in terms of draining the stormwater more quickly. Therefore this comparison is still valid, and the model gives an idea on how both types of systems respond to high rainfall.

Scenario 2

Considering the rainfall scenarios in Figure 6-7 applied to the model, both systems handled the rain events with minor flooding, where the water level exceeded the ground level with a small water depth at most of the flooded nodes. However, with a more extreme case considered it was possible to distinguish which system handles such conditions better.

The open system handled the rainfall event better favored by its higher capacity. In open systems, even in flooding cases there is usually more room at the surface which can retain a certain volume of water during high peaks.

With regard to the modeling software, it is not possible to simulate surface flooding in MIKE URBAN, which can be important in modeling the open stormwater systems to assess any risks due to flooding. However the node flooding is of value in the design process to locate potential problem spots.

CONCLUSIONS

From the model results and the preceding discussion it can be stated that the open stormwater systems with green solutions do contribute in reducing the peak flows. In this case at least 50% was reduced compared to the traditional pipe system.

The open solutions also delay the peak flow timing, which helps preventing high flow problems downstream.

In more extreme rainfall events, the open solutions handle the flooding better than the conventional systems and flooding is less critical specially when the water path is situated in parks and areas at a safe distance from buildings. Compared to combined systems that can cause considerable damage when flooding occurs, or separate stormwater systems that can flood streets for example.

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