A case study of runoff coefficients for urban areas with different drainage systems

Christine Thomas

Water and Environmental Engineering
Department of Chemical Engineering
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Department of Chemical Engineering
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Supervisor: **Doctoral student Salar Haghighatafshar**
Examiner: **Senior lecturer Karin Jönsson**

Picture on front page: Pond in Augustenborg. Picture taken by Salar Haghighatafshar

Postal address	**Visiting address**	**Telephone**
P.O. Box 124	Getingevägen 60	+46 46-222 82 85
SE-221 00 Lund, Sweden		+46 46-222 00 00
Web address		**Telefax**
www.vateknik.lth.se		+46 46-222 45 26

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Summary

According to the United Nations, half of humanity now lives in cities. In recent years, many countries have seen a rise in the number of and intensity of heavy rainfall events due to the effects of climate change. The consequences of urban flooding in such a context can vary between just causing mild discomfort to tremendous devastation. Building cities that can resist flood is quickly becoming an important part development plans across the world.

One of the upcoming techniques to handle storm water is to install sustainable drainage systems (SuDS). The primary goal of SuDS is to reduce to strain on the storm sewer network by retaining storm water on the surface as long as possible. While the water is retained at the surface, it is utilized to improve the local biodiversity and for its aesthetic value. Typical SuDS include vegetated swales, green roofs, detention ponds and open channels.

The city of Malmö, Sweden is that has borne the brunt of several unusually high rainfall events in recent years. To build the resilience of the city against future heavy rainfall events and prevent basement flooding, a number of residential areas and parks across the city have incorporated SuDS along with traditional pipe systems. One such project in Malmö is Eco-city Augustenborg, which is the study site the current project. The project focuses on the northern part (NS) of the study area with implemented SuDS and one part that uses a traditional pipe system (the PS).

Using rainfall and discharge data measured on-site, the total runoff volume from the PS and NS for several rainfall events was calculated. Measured data from the PS and NS were used compute two values of runoff coefficient (φ) for the study area – one for all pervious surfaces and one for all impervious surfaces (roofs, asphalt and concrete), based on the rational method approach. Results from calculated runoff coefficients were then applied to a 1D MIKE URBAN model of the site to simulate runoff during non-uniform rainfall events. Four historical observed rainfall events were used in this project. The results show that, although the methodology proposed above is very simple, calibration curves were obtained with volume errors as low as 5.8% and peak error as low as 1.07%. The model appears to be more effective for heavier rainfall events.

Furthermore, through a literature review portion of the project it is revealed that a widespread confusion exists in scientific articles between the terms Effective Impervious Area (EIA) and Directly Connected Impervious Area (DCIA). These terms are very important in runoff studies but they are often used interchangeably in literature. A better understanding of their definitions would make results from different runoff studies more consistent and comparable. A distinction between the definitions of the two terms is sought out and articulated in this project.

Abbreviations and Symbols

1D	One-Dimensional
2D	Two-Dimensional
DCIA	Directly Connected Impervious Area
DEM	Digital Elevation Model
EEA	European Environment Agency
EIA	Effective Impervious Area
GIS	Geographic Information System
MKB	Malmö Kommunala Bostads
NS	Northern System
PS	Pipe System
REIA	Runoff Equivalent Impervious Area
SMHI	Swedish Meteorological and Hydrological Institute
SuDS	Sustainable Drainage Systems
TIA	Total Impervious Area
TPA	Total Pervious Area
UDFCD	Urban Drainage and Flood Control District, Colorado
UN	United Nations
φ	Runoff Coefficient
φ_{per}	Runoff coefficient for pervious surfaces
φ_{imper}	Runoff coefficient for impervious surfaces

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1 Introduction

With global temperature steadily on the rise, there is a great need to develop flood resilient and sustainable cities. The Swedish Meteorological and Hydrological Institute (SMHI) climate scenario predicts that by the turn of the century, the mean annual temperature in Southwestern, Götaland, Sweden could increase by up to 5.2°C (SMHI, 2017). Globally, increases in temperature are expected to result in higher annual precipitation and an increase in the number of extreme rainfall events in a year (SMHI, 2017). Such an increase in annual precipitation elevates the risk of flooding in places that are not equipped to handle them (Donat *et al.*, 2016).

According to recent studies by the United Nations, as of 2016 approximately 54.6% of the world's population lives in cities. The same report also highlights flooding as the most prevalent natural disaster that affects cities both in terms of loss of life and economic losses (United Nations, 2016).

Preparing cities for such scenarios requires extensive studies into soil properties, land use types, local topography, etc. of the local catchment. Once the correct information about these parameters is available, urban planners can use this data to design more water efficient societies with drainage systems that are adapted to handle future extreme precipitation events.

One of the important parameters required for estimation of runoff flow is the runoff coefficient of the catchment. This parameter varies greatly from location to location and is generally a source of error when designing the hydraulic network for a place. This project is an attempt to determine and analyse the runoff coefficients for urban areas with different drainage systems.

The study area for this project was Eco-city Augustenborg in Malmö, Sweden. Augustenborg uses a mixed drainage system combining Sustainable Drainage Systems (SuDS) and conventional pipe drainage systems to manage storm water. The project focuses specifically on two sub-catchments in the area-the Northern System (NS) which uses SuDS, and the Pipe System (PS).

1.1 Aim

The key objective of the project is to study the rainfall runoff response mechanism of SuDS and PS in order to better plan future urban drainage systems. The goal of this project is to evaluate how well rainfall runoff response of the NS and PS can be modelled by simplifying surface classifications to just two groups of pervious and impervious.

Moreover, it is tested whether the calculated runoff coefficients for pervious and impervious surfaces using the onsite measurements of rainfall and runoff could be used to calibrate a 1D model of the site in MIKE URBAN. This concept is tested on the Augustenborg study area. Through this, the project also aims to explore a simplified methodology to calibrate a 1D drainage model as well as the advantages and drawbacks of using a 1D model to simulate flow through SuDS.

Additionally, through a literature study, the definitions and the relevance of Total Impervious area (TIA), Effective Impervious Area (EIA) and Directly Connected Impervious Area (DCIA) in runoff calculations are evaluated.

2 Literature Review

2.1 The Rational Method

The origins of the rational method to compute peak runoff for a rainfall seem to date as far back to 1850 where an Irish engineer Mulvaney (1850) published the principles that the method is based on. However, modern American engineers credit Kuichling (1889) who was engaged in sewer design. The British credit the method to Lloyd-Davies (1906) who used it for sewer design calculations in England (Butler and Davies, 2004; Shaw, 1994).

The Rational Method developed by Kuichling (1889) is an equation frequently used by hydrologists and civil engineers to calculate peak flow from small catchments of less than 20 hectares (Svenskt Vatten AB, 2016). The main appeal of the rational method lies in its simplicity and its widespread acceptance, although this has often been criticized by academics. However, when properly understood and applied, the rational method provides satisfactory results when designing urban storm drainage networks (UDFCD, 2016). In its most basic form, the rational method equation is as follows:

$$Q = \varphi \times i \times A \tag{2.1}$$

Where:

Q = peak discharge rate (l/s)
i = rainfall intensity (l/s.ha)
φ = runoff coefficient (-)
A = area of the catchment (ha)

The intensity of the rainfall is obtained from intensity-duration-frequency (IDF) tables. The duration of the design storm is the time of concentration of the catchment. The time of concentration is defined as the time (from the start of the storm) after which the most remote parts of the catchment contribute to runoff from the outlet of the catchment.

As stated by Cleveland *et al.* (2011), the assumptions made in employing the rational method are as follows:

- The rainfall has a uniform time distribution for at least a duration equal to the time of concentration of the catchment
- The maximum runoff occurs when the rainfall intensity lasts at least as long as the time of concentration
- The value of φ is constant during a storm
- The contributing area is kept constant during the rainfall

Numerous modifications can be made to the rational method. For example, the Swedish practice for pipe system dimensioning includes a climate factor term in addition to those seen in equation 2.1 (Svenskt Vatten AB, 2016).

2.2 The Runoff Coefficient (φ)

Different portions of a watershed have different degrees of perviousness. The impervious parts of the catchment are those that do not allow incoming rainfall to infiltrate through them into the ground immediately and the pervious parts readily allow infiltration until they get saturated (UDFCD, 2016). The runoff coefficient, φ, is a coefficient with a value between 0 and 1 defining the proportion of rainfall that turns into runoff. This value is crucial to the correct implementation of the rational method equation. When φ=0, it means that all of the water falling on the surface infiltrates through it, and when φ=1, it means that all the water falling on the surface turns into runoff. Inaccurate results can occur upon using the rational method due to difficulties in selecting an appropriate value of φ, since φ effectively determines the amount of incident rainfall that is being routed as runoff (Beven, 2012). The value of runoff coefficient encompasses the effects of infiltration, interception, evapo-transpiration and retention by a certain type of surface and thus the selection of an appropriate value for calculations requires sound judgement from an experienced engineer (UDFCD, 2016).

Tables of runoff coefficient for various types of materials and surfaces are available in many hydrological textbooks and design manuals for use in engineering design and calculations. As an example, the Swedish design practice (Svenskt Vatten AB, 2016) recommends that the values of φ to be used are as described in Table 2.1 unless some other values can be proven to be more correct.

Table 2.1 Runoff coefficients for different types of surfaces for short duration design rainfall, translated to English from Publication P110 (Svenskt Vatten AB, 2016).

Type of surface	Runoff coefficient (φ)
Roof without storage	0.9
Concrete or asphalt, rock with large slopes	0.8
Cobbled stone with gravel joints	0.7
Gravel road	0.4
Rock with small slopes	0.3
Gravel paths	0.2
Park	0.1
Lawn, pasture, etc.	0-0.1
Forest, no slopes	0-0.1

However, the reasoning behind the selection of these values is not mentioned. Thus a large degree of ambiguity continues to exist regarding the validity of the available values of runoff coefficient (Young *et al.*, 2009). This ambiguity is amplified by that fact that runoff coefficient varies based on a number of local conditions apart from just the type of surface.

Sriwongsitanon and Taesombat (2011) studied the effects of land cover such as forest area, agriculture, urban areas, etc. on runoff coefficient. They found that the correlations between runoff coefficient and peak flow vary differently for different land uses and depending on the severity of the rainfall event. Pre-existing soil moisture conditions including previous rainfall events and snowmelt also affect the runoff coefficient (Merz *et al.*, 2006).

Merz *et al.* (2006) conducted studies on the spatio-temporal variation of runoff coefficients in six climatic regions of Austria. They found that the runoff coefficient varies more between different regions than it does between storms of different intensities or antecedent soil conditions within a region. The Austrian runoff coefficients appear to be driven mainly by the water balance of the catchment. This means that the spatial variation of runoff coefficients closely resembles the spatial variation of annual precipitation. The regions that yield the highest runoff coefficients are the regions with the highest mean annual precipitation, and not necessarily the steepest regions. This is in contrast to the findings of Gottschalk and Weingartner (1998) who interpret runoff coefficient of Swiss catchments to be driven primarily by the physiographic conditions like slope and altitude. Neither of these two studies are specific to urban catchments, but instead their studied catchments are classified only by other characteristics such as slope, altitude and climate.

When rainfall and runoff data are available, there are two concepts that can be employed to calculate the local runoff coefficients at a specific location. One method is to invert equation 2.1 and use it to calculate φ. The second way to determine the runoff coefficient is a volumetric approach where φ is equal to the ratio of runoff volume to precipitation volume. The latter here (the volumetric approach) is not strictly in accordance with the rational method which actually gives only the *peak* flow. However, these two concepts are frequently used interchangeably in literature as the rational method runoff coefficient (Cleveland *et al.*, 2011). This concept is also used later in this project.

An additional concept used to understand the runoff coefficient, which was originally conveyed by Kuichling (1889), is that it is a proportion of the total drainage area that is functionally impervious and contributes to runoff during a particular rainfall event (Cleveland *et al.*, 2011). This concept is used in some computer models that treat the percentage of impervious area in a catchment as the actual runoff coefficient. The importance of impervious area fractions is discussed further in other sections of this report.

2.3 The Drainage Area

The areas contributing to runoff are primarily the impervious areas. Impervious surfaces in urban settings include concrete rooftops, parking spaces, tarred roads, etc. The total sum off all of these surfaces is called the Total Impervious Area (TIA). A number of runoff calculations use TIA as the area term since TIA can be calculated in a straightforward manner from aerial photographs of the catchment. However, recent studies show that runoff from the catchment may be described better by Effective Impervious Area (EIA) than it is by TIA (Ebrahimian *et al.*, 2015). EIA is defined by Ebrahimian *et al.* (2016) as the 'portion of TIA that is hydraulically connected to the storm sewer network'. This definition seems to indicate that all of the precipitation falling on an area equal to the EIA will become runoff. In other words, if EIA is used for runoff computation, then φ=1.

Directly Connected Impervious Area (DCIA) is another term that is frequently used in runoff studies. Ebrahimian *et al.* (2016) define DCIA as the 'areas that are directly connected to the drainage system by impervious pathways.' DCIA can be defined by field inspection or accurate map data.

Ebrahimian *et al.* (2015) use the terms EIA and DCIA exclusively in their report on determination of effective impervious area in urban watersheds. They cite Boyd *et al.* (1993) who determined that typically the EIA is equal to or less than the DCIA. This indicates that EIA is a portion of DCIA and that impervious runoff actually only occurs from a portion of the DCIA. EIA and DCIA mark different degrees of hydraulic connectivity to the storm drainage system.

The difference between the terms EIA, DCIA and TIA is important since *all* storm water from the EIA turns into measurable runoff. All the water falling on TIA initially runs off, however, some of the storm water from the TIA will infiltrate through pervious surfaces or evaporate, depending on the ground conditions and will thus be lost before it leaves the catchment. Thus, the problem with using TIA instead of EIA or DCIA for runoff studies is that it may result in overestimation of the runoff volumes and rates (Alley and Veenhuis, 1983). EIA and DCIA also differ from one another. Blockages of pipes and their inlets may be one of the reasons for the difference between EIA and DCIA. The fundamentally different approaches to obtaining the values of EIA and DCIA could also be the reason for the difference between them – DCIA is obtained from maps while EIA is averaged based on measured data regarding rainfall-runoff response (Ebrahimian *et al.*, 2016).

In contrast, most literature employ the terms DCIA and EIA interchangeably (Han and Burian, 2009; Lee and Heany, 2003; Sutherland, 1995), with either one of the terms being used consistently in each study. This inconsistent terminology results in ambiguity between the terms EIA and DCIA since both terms are used but not clearly distinguished and there is no mathematical relationship between the two. In this report, a distinction is made between EIA and DCIA matching the definitions provided by Ebrahimian *et al.* (2016).

Boyd *et al.* (1994) determined EIA by plotting total rainfall depth against runoff depth for three drainage basins in Australia using up to 47 rainfall events for each. The idea was that if the EIA remains constant for all the rainfall events, then the regression would be a linear one with slope equal to EIA and intercept (on the rainfall axis) equal to initial loss. This method can be used to determine EIA of a region if large quantities of historical rainfall runoff records are available. The method can also be used to identify whether runoff occurs from just impervious or also pervious surfaces depending on the severity of the rainfall event – for small rainfall events, runoff is generated only from impervious surfaces while for large rainfall events, runoff is generated from both pervious and impervious surfaces. However, use of this method requires a person skilled to correctly separate between the small rainfall events and the large ones. The user must possess very good understanding of the local hydrology and precipitation patterns in order to identify and separate the rainfall events that would be considered 'small' and 'large' in study location. Boyd *et al.* (1994) separate small rainfall events as those of depth 40 mm depth and below, however this may not necessarily be true for other locations.

With a combination of modern GIS technologies and field studies many of the difficulties in measuring TIA and DCIA can be overcome (Brabec *et al.*, 2002). However, a lot of manual work is required to classify the impervious areas and then separate those that could be further classified as DCIA. These methods of estimating impervious areas are referred to as the direct methods.

The indirect methods of estimating DCIA are based on land use, TIA, population density, etc. Indirect methods usually take the form of a statistical equation that can be used to calculate DCIA based on some other properties of the watershed (Sahoo and Sreeja, 2016). These equations are highly location and data specific, and could result in considerable errors if applied in different locations. Some of the equations found in literature to determine DCIA are reported below.

According to Alley and Veenhuis (1983) based on 14 urban watersheds in Denver, Colorado:

$$EIA = 0.15 \times TIA^{1.41} \qquad (2.2)$$

Where both EIA and TIA are percentages of the total watershed area. This relationship is specific to the region for which it was derived; however, it illustrates the method that can be used to formulate such a relationship for other regions. Alley and Veenhuis (1983) define EIA in the same way as Ebrahimian *et al.* (2016), but they derive the above relationship based on the lengths and widths of streets using aerial photographs.

Sutherland (1995) developed a series of equations known as the Sutherland equations in order to determine the value of EIA based on TIA for various types of sub-catchments with different levels of connectivity to the storm water network. The equations proposed by Sutherland take the general form of:

$$EIA = A \times TIA^{B} \qquad (2.3)$$

Where EIA and TIA are percentages of total watershed area and A and B are two unique values which satisfy the conditions that if TIA =1 then EIA =0% and if TIA =100 then EIA =100%. These conditions imply that if impervious surfaces comprise only 1% of the total watershed area, then EIA is zero, i.e. it does not contribute to runoff at all. Similarly, if a 100% of the total watershed area is impervious, then the entire area is considered as EIA. Although Sutherland uses the term EIA in the original article, upon reading his definition of the term carefully, it was determined that his equations actually describe DCIA. This equation will be used later in this study to determine DCIA.

The values of A and B for different types of sub-catchments are given in Table 2.2. For all the forms of the equation, the prerequisite condition is that the TIA of the basin is at least 1% of the total sub-basin area.

Table 2.2 Values of A and B of the Sutherland Equation (Sutherland, 1995).

Sub-catchment Type	A	B	Eqn.
Average Catchment: Mostly connected to storm sewer by curb and gutters, no dry wells or other infiltration, rooftops are not directly connected to storm sewer.	0.1	1.5	(2.3.1)
Highly Connected Catchments: Similar to condition 1 but rooftops are connected to storm water system.	0.4	1.2	(2.3.2)
Totally Connected Catchments: 100% of impervious area is directly connected to the storm water sewer system.	1	1	(2.3.3)
Somewhat Disconnected Catchments: At least half of the areas within the catchment are not directly connected to the storm water sewer but are handled by dry wells, swales, grassy areas, etc. Rooftops are not directly connected to storm sewer.	0.04	1.7	(2.3.4)
Extremely Disconnected Catchments: Only small portions of the urban areas within the catchment are connected to storm sewer. Alternatively, majority of the sub-catchment drains into infiltration areas.	0.01	2.0	(2.3.5)

Concluding Remarks
To sum up the findings from this section, DCIA is measured from maps, GIS and field visits whereas EIA is calculated based on rainfall runoff response. The entire DCIA need not necessarily contribute to runoff while hundred percent of the EIA contributes to runoff. DCIA consists of impervious surfaces only, whereas EIA may also include pervious surfaces that are contributing to runoff to the outlet. Finally, DCIA of an area is static, in that it remains constant for every rainfall event unless some construction/urbanisation requires more impervious surfaces to be laid out in the area. EIA, on the other hand, is a dynamic area that can change for different rainfalls and also changes in size during a particular rainfall event. The exact surfaces that constitute EIA for a particular rainfall may not be possible to predict.

Most literature fails to recognize these inherent differences between the terms and so use them interchangeably. Many of the articles in literature that mention EIA actually deal with DCIA. In order to overcome this confusion amongst the various terms, Haghighatafshar *et al.* (n.d.) calculated a term called the runoff equivalent impervious area or (REIA) of Augustenborg, Malmö based solely on the rainfall-runoff data collected on site. There are numerous articles published about determining 'EIA' using GIS for land surface classifications and from studying maps. However, since EIA depends on rainfall runoff response only, this should not logically be possible. For the same reason, it also seems improbable that mathematical relations can be made to compute EIA just from the TIA. Therefore, when using empirical equations found in literature to calculate the area contributing to runoff, it is crucial to understand the author's definition of the term and exercise some degree of caution and professional judgement in using them. This widespread ambiguity between the terms in literature makes it very difficult to study these terms separately and distinguish between previous studies of the two terms.

2.4 Sustainable Drainage Systems (SuDS)

The key intent of a conventional drainage system is to collect and transport urban runoff as quickly as possible. This transport is performed via sewer systems after which it is released into a nearby receiving water body. Sometimes, the water is even treated at a treatment plant before being released to the receiving body. The processes of urbanization and climate change places enormous amounts of stress on conventional drainage systems by increasing the operational load on them. Additionally, land cover modifications as a part of urbanization also affect the water quality of the generated runoff. (Zhou, 2014).

There are many disadvantages to employing a conventional or pipe system. The cost and time required to maintain, restore or scale-up pipe systems, is often very high. Pipe systems also fail to recognize the potential recreational and aesthetic value of sustainable storm water drainage solutions and choose instead to dispose storm water runoff as soon as possible. Additionally, if the pipe system is used as a combined sewer system, combined sewer overflows may occur which result in pollution of the receiving waters. (Zhou, 2014).

Sustainable Drainage Systems (SuDS) techniques are applied in many parts of the world alongside with conventional systems. According to Charlesworth and Booth (2016), the purpose of SuDS is to manage storm water in such a way that:

- Reduces flooding and the related damage
- Improves water quality
- Improves and protects the local environment
- Improves health and safety

▪ Ensuring stability and robustness of the drainage system

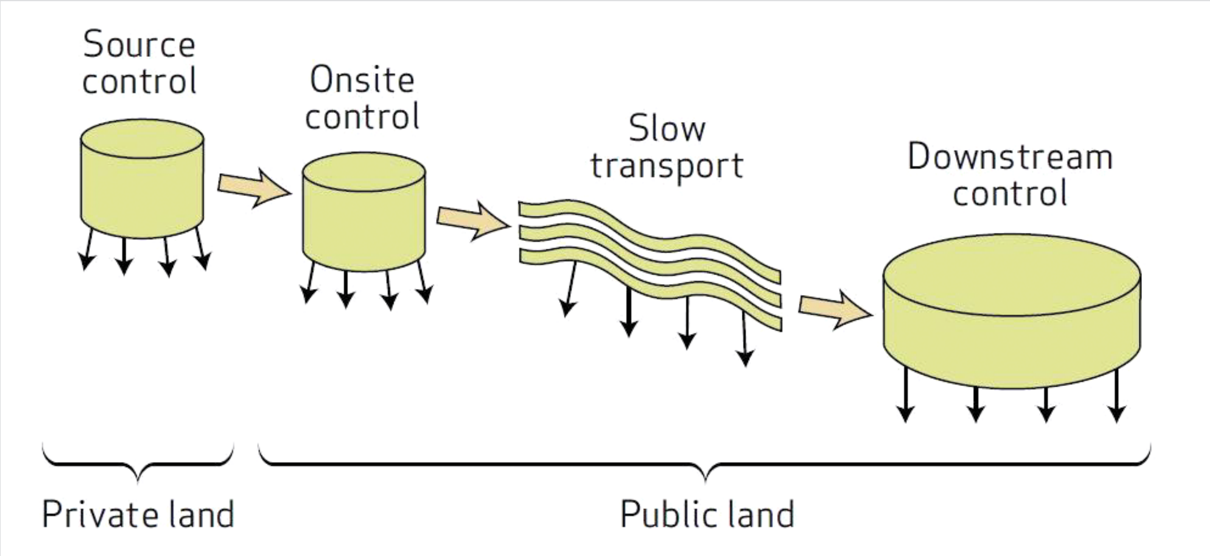


Figure 2.1 The four classifications of SuDS.
Source: Published with kind permission from VA SYD and Pekka Kärppä (Stahre, 2008).

There are three classifications of SuDS based on a hydrological perspective – Source control measures, on-site control measures and downstream measures (Zhou, 2014). Stahre (2008) also uses an additional category called slow transport. Source control is tackled by the installation of structures such as green roofs, pervious pavements, rain gardens and other infiltration surfaces on private land. On-site facilities are similar to source control measures, but they are situated on public property and therefore are managed by the local municipal board. Slow transport measures include open ditches, swales and canals. Downstream control features are various larger structures such as detention ponds or wetlands that are used to temporarily detain the water (Stahre, 2008). The four classifications are illustrated in Figure 2.1.

One of the biggest problems with implementing SuDS in the modern context is the allocation of responsibility for maintaining the system. In the present urban system, water is managed by either governmental bodies or private water companies. This has resulted in a setting where the individual consumers of water are no longer concerned with exactly how they obtain or dispose their water (Charlesworth *et al.*, 2016). Policies, strategies and laws are now often set up by governments as a way to ensure safe and efficient management of surfaces water resources. As stated by Warwick (2016), water related laws construed in different ways depending on their level (international, national or local) or the local scenario. It is often difficult to consolidate different aspects of surface water management such as water quality and flood risk management into a single coherent piece of legislation. However, having these in place could be a good starting place to implement SuDS in cities.

The city of Malmö in southern Sweden may be considered as a model city as far as the implementation of SuDS is concerned. Since the early 1990s, there has been a shift towards the use of open drainage systems in the city (Haghighatafshar *et al.*, 2014). Peter Stahre describes 18 different projects that incorporate various principles of SuDS around the city of Malmö in his book 'Blue-Green fingerprints in the city of Malmö, Sweden' (Stahre, 2008). The city of Copenhagen, Denmark is presently considering the employment of sustainable drainage solutions as part of their cloudburst management plan. However, at present Copenhagen has a predominantly combined sewer system. These two cities lie very close to each other and therefore have very similar climates. Their different choices of urban drainage have been compared by Haghighatafshar *et al.* (2014) who analyse the cities' approaches to employment of different drainage systems and flood protection.

Public perception, awareness and involvement is the next important step in successful SuDS implementation. Public participation can be encouraged by a rewards system such as reducing water utility bills for individual household that employ small SuDS in their own property (Everett, 2016). Public awareness is important so that people do not misuse SuDS in public spaces, and understand their purpose. Everett *et al.* (2016) interviewed local people on a site in Portland, Oregon and one in Bristol, UK where SuDS had been present for 8-10 years. The responses of the people reflect their lack of knowledge about the purpose, benefits and maintenance about their local swales or pervious surfaces, even after being surrounded by them for long. This included a story of an elderly resident disposing garbage into a bioswale. In contrast, Augustenborg in Malmö is regarded as a successful example where local residents contributed to implementing SuDS by suggesting designs and reporting issues when something did not work as expected (Stahre, 2008). The setting up of SuDS in Augustenborg included several community meetings and workshops with the residents to obtain their input (EEA, 2016).

The transition towards use of SuDS has generally been quite slow. Elliott and Trowsdale (2007) speculate that the reason for the slow growth could be, in part, due to a shortage of suitable SuDS design tools and models. Although implementation of SuDS has been beneficial to water quality and quantity management, Zhou (2014) reviewed many cases that pointed out some drawbacks. With respect to the response of SuDS to increasing loading, he says, "It was found that the SuDS techniques impact water flows; however, the reduction of water volume is very limited in extreme events and sensitive to local conditions, such as size and duration of rainfall event, soil material and texture." However, the additional lag time and evened out flow resulting from SuDS are worthwhile advantages.

2.5 Urban hydrological modelling

The use of numerical computer models has become a popular solution for runoff computation and designing of pipe networks. Of the various types of models available, one-dimensional (1D) models are amongst the most commonly used, due to their simplicity, high degree of accuracy and relatively short runtime. 1D models are limited in that they can only predict the surcharge flow volume from a pipe system – this means that they cannot compute overland flow. So while they can determine whether a pipe system will overflow or not, they will not be able to determine the extent of flooding on the surface. There is also no interaction of flow between sub-catchments (Chang *et al.*, 2015). These models provide a satisfactory approximation as long as water stays within the flow channels. If there is overtopping of water over the curbs and the direction of flow changes, then 1D models are no longer sufficient (Leandro *et al.*, 2009) and a more complex model is required. The 1D modelling software developed by DHI are MIKE URBAN, MIKE HYDRO River and MIKE 11. Other commonly used 1D models are Storm Water Management Model (SWMM) and MOUSE. MIKE URBAN can use either a SWMM% engine or MOUSE engine to model collection systems (DHI, 2016a).

If both pipe flow and surface flow are to be considered, a two-dimensional (2D) approach might be preferable (Leandro *et al.*, 2009). In 2D models, the topography of the land surface in the form of a digital elevation model (DEM) is used to calculate the flow paths of the water. 2D models are also more useful than 1D models when the slope of the surface is weak (Lhomme *et al.*, 2006). MIKE 21 developed by DHI is an example of a 2D modelling software.

2D models are very computationally intensive when compared to 1D models. The 2D models require far more data and are very time consuming to run. Lhomme *et al.* (2006) created a simplified 1D GIS based model to replace a complex 2D model of a small area in the city of Nîmes, France. They reported that the 1D model took 20 minutes to run whereas the runtime of the 2D model went into several hours.

Typically, 1D models do not allow water that surcharges from a manhole to flow over the surface. Instead, the water is just held above the manhole until the capacity of the pipe is available again. Some examples of models that do this are (SWMM) and MOUSE. In order to deal with this, a dual drainage concept was introduced American Society of Civil Engineers (ASCE, 1993) which recognized two drainage systems in every urban catchment – the minor system and the major system. The minor system consists of the conventional storm pipes that can accommodate moderate and frequent flows. The major system comprises of streets and other structures on the surface which operate when the runoff flow exceeds the capacity of the minor system (ASCE, 1993). The conventional design system ignores the major system (Kolsky *et al.*, 1999).

The dual drainage concept was translated into numerical models as the 1D/1D model. 1D/1D models were the first type of urban flood models adopting the dual drainage concept, whereby the major and minor systems were modelled in 1D simultaneously and allowed to interact with each other (Leandro *et al.*, 2009). Thus, these models allow the surcharged flow to move along the surface and return to the sewer system via downstream manholes. (Nordlöf, 2016). An example of a 1D/1D model is SIPSON (acronym for Simulation of Interaction between Pipe flow and Surface Overland flow in Networks), developed by Djordjević *et al.* (2005).

Recently, 1D/2D models have also developed which couple 1D sewer models with 2D overland flow models. 1D/2D models can be used when runoff is no longer confined by a drainage system (as assumed in the 1D and 1D/1D models). Such a situation could occur when, for example, the flood depth exceeds the crest of a pond. To model such a scenario, 2D overland flow model is required (Chang *et al.*, 2015). Models that couple 1D sewer flows with 2D overland flows have been developed by a number of researchers using various approaches (Hsu *et al.*, 2000; Chen *et al.*, 2007; Chang *et al.*, 2015). MIKE FLOOD is used to integrate 1D models by DHI with the 2D model MIKE 21.

When all hydrological process such as surface flow, infiltration, evapotranspiration and groundwater flow all need to be integrated together, there are models that are more comprehensive such as MIKE SHE by DHI. Domingo *et al.* (2010) created a hydraulic-hydrological model in Greve, Denmark by coupling MIKE SHE with MOUSE. The MIKE SHE model was used to simulate the hydrology and MOUSE was used to simulate pipe flow. Their goal was to develop a modelling technique that considers the entire water cycle of an urban area for flood analysis. They found that their method was able to simulate changes in hydrological processes such as infiltration and soil moisture. They concluded that their modelling procedure is necessary to model mixed-urban settings because interactions between hydrology and hydraulics could be of great importance in some flood prone regions.

There are many examples of numerical modelling of SuDS. Feitosa and Wilkinson (2016) used a 1D model called HYDRUS 1D to simulate the runoff response of green roofs with different soil depths based on the climactic conditions in Auckland, New Zealand. Gunnarson (2015) used MIKE 21 to model an infiltration swale in Solbacken. Rosa *et al.* (2015) used SWMM to model two watersheds in Waterford, Connecticut – one with a SuDS and one traditional. Jato-Espino *et al.* (2016) developed a model by integrating ArcGIS software with SWMM to demonstrate the benefits of SuDS for flood mitigation in a highly urbanized area of Spain.

According to Elliott and Trowsdale (2007), the availability of more efficient models for SuDS could result in more widespread implementation of the SuDS principles. They compared ten different currently available commercial models in relation to seven attributes. The attributes that they compared were: intended use, temporal resolution and scale, drainage network representation, runoff generation and flow routing, types of contaminants included in the model, low impact development structure included in the model, and user interface and possibilities to couple with other available models. Their study points out the specific strengths and shortcomings of these ten models and is useful in selecting an appropriate model based on the need.

3 MIKE URBAN

The MOUSE Engine is a powerful computational engine created by DHI which can be used for complex modelling of open and closed channel hydraulics, runoff generated from catchments, pollution transport in drainage systems and water distribution and collection systems (DHI, 2016a). MIKE URBAN, a 1D model used in this study, is a commercial program, which employs the MOUSE engine for hydraulic computations.

Some of the basic concepts and equations behind the MIKE URBAN software and the MOUSE engine are explained in this chapter. MIKE URBAN can also be used to model contaminant transport and some biological processes but these topics will not be discussed in this report as they are not covered within the scope of this project.

The modelling of a collection system using MOUSE requires a hydrological model and a hydraulic model. Figure 3.1 depicts the flow of information in the MOUSE hydrological model.

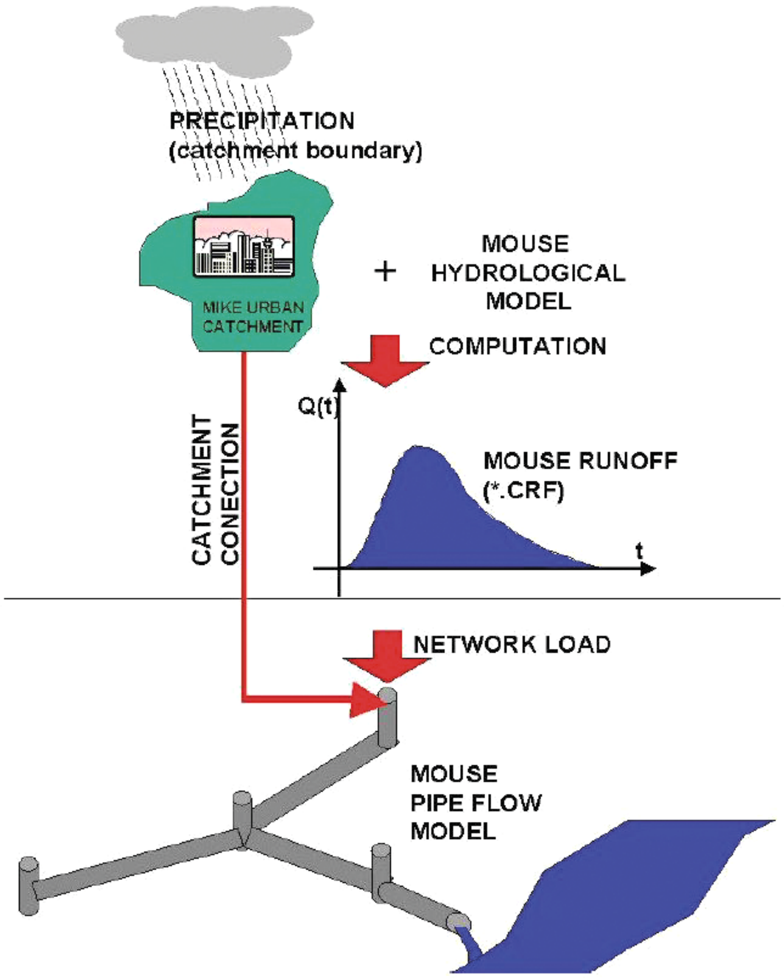


Figure 3.1 Figure showing flow of information in a hydrological model.
Source: Published with kind permission from DHI.

3.1 Rainfall Runoff Modelling

Mike URBAN offers four surface runoff models that can be chosen by the user- time-area method, non-linear reservoir method, and linear reservoir method and unit hydrograph model.

The Time-Area method is used in this project and is described in this section. The time-area method relates the amount of runoff generated to the size and shape of the catchment, and the concentration time.

The input data required for runoff computation by the time area method are:

General Catchment Data: The area being modelled must be drawn out as catchment polygons. For the surface runoff model to operate without error, each catchment polygon must have a unique catchment ID, a set of X and Y coordinates and a horizontal catchment area.

Imperviousness (%): The imperviousness fraction is the percentage of the catchment area that is considered to contribute to the runoff. This term is analogous to the runoff coefficient in the rational method. The value of imperviousness for each catchment polygon must be set by the user and it is a key parameter involved in model calibration.

Initial Loss: The initial loss is the depth of precipitation (m) that is required to fall on a surface to start runoff. The initial loss is a one-time loss that only has an effect at the beginning of the simulation. Its purpose is take surface depressions and other minor surface losses into account.

Hydrological Reduction: This parameter accounts for losses due to evapo-transpiration, imperfect imperviousness, etc. and it has a default value of 0.9.

Time-Area Curve: The time area curve is the parameter that the model uses in order to take the shape of the catchment into account. Mike URBAN contains three pre-defined time area curves which are

- TACurve1 – Rectangular Catchment
- TACurve2 – Divergent Catchment
- TACurve3 – Convergent Catchment

In addition, users can also define their own curves (DHI, 2016c).

Time of Concentration: The time of concentration is defined, as the time (min) required for water from the most distant part of the catchment to flow to the point of outflow. The catchment processing wizard is used to compute the time of concentration for each catchment automatically. It is calculated as the time required for the water to travel from the centroid of the catchment to the connected node (Shukri, 2010). For a higher accuracy, this parameter can also be entered manually.

Precipitation: The precipitation data is added to the model in the form of a time-series. Boundary conditions can be used to specify which catchments are affected by the precipitation.

Upon beginning the simulation, the model starts to generate a runoff only after the volume of precipitation has exceeded the initial loss specified. After this, the rainfall accumulated in each catchment for ever time step is calculated. The accumulated volume for a particular time step is the difference between the input in that step and the volume accumulated in the previous step.

The runoff finally stops when the accumulated rain depth over the catchment falls below the initial loss (DHI, 2016c).

3.2 Hydraulic Network Modelling

Once the runoff from each catchment is simulated, a hydraulic network model can be used to simulate the flows and depths in each pipe, channel, node and pond in the system. The transfer of runoff volume from the runoff model to the hydraulic network is done by connecting each catchment to a specific node in the network (DHI, 2016a).

The most basic constituents of the network model are links and nodes, which are discussed further in this section.

3.2.1 Links

Pipes, canals and open drainage channels are modelled in MIKE URBAN as links. A link is bounded on each end by a node. It can be either a straight line or a polyline if a curved channel is being modelled. The standard pipe cross sections available in MIKE URBAN for closed pipes are – circular, egg-shaped, O-shaped and rectangular. Any other open channel or closed pipe cross sections can be defined by the user using the Cross-Section (CRS) and Topography editors (DHI, 2016a).

A CRS is defined by specifying the width of the link at different heights from the bottom. Figure 3.2 is an example of how a CRS is defined. The length of the link is calculated by MIKE URBAN from the shape of the line. However, if the user specifies a length for a particular link, this value overwrites the one calculated by the software (DHI, 2016a).

H(m)	W(m)
0	0
0.01	0.1
0.03	0.15
0.04	0.15

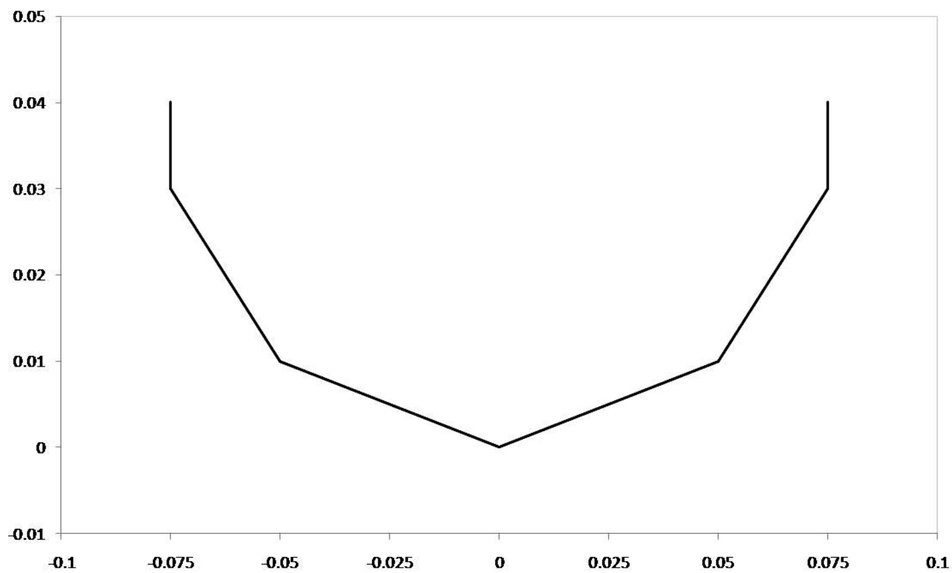


Figure 3.2 Example of CRS definition - height and width relationship entered in a tabular form as seen on the left results in the cross section seen on the right.

The material of the link is an important parameter since it determines the Manning number (or Hazen Williams coefficient) used in flow computations. The material properties can be edited using the MOUSE Materials editor. Either default values or local values can be used (DHI, 2016a).

Finally, the longitudinal profile and slope of a link are defined by specifying the bottom elevation at the upstream and downstream ends.

The equations that are used to compute flow in these links are discussed in Section 3.3. For open channels, computation of flow is only possible as long as the water level in the channel is equal to or less than the height of the channel. If the water level exceeds the channel height, simulation ends and an error message is generated unless extrapolation of the cross section is specified. For closed pipes, increase in flow will result in increase in pressure inside the pipe.

3.2.2 Nodes

Manholes, basins, outlets and storage nodes are considered as nodes. Every pipe network must contain at least one outlet, which is a node where water leaves the system. Based on the type of top cover, nodes are categorized as open, sealed or spilling nodes. Open nodes will release water to the surface when the water level in them exceeds ground level, and this overflow is reversible once the water level drops. Manholes and basins are modelled as open nodes unless otherwise specified. Closed nodes will not allow any water to overflow from them but will instead build up pressure inside the network. Spilling nodes will simulate water spilling irreversibly out of the system when the water level exceeds the ground level (DHI, 2016a). All the nodes are required to have a set of coordinates associated with them to specify the location. The model uses these coordinates to calculate the length of a link.

Manholes can only be cylindrical in shape and must have a defined top elevation, bottom elevation and diameter. When a structure can store a considerable volume of water and has an irregular cross section, the node can be modelled as a basin. Basin geometries are defined by a table relating height to corresponding surface area and cross sectional area (DHI, 2016b).

In addition to the links and nodes, control features such as weirs, orifices, pumps and valves can also be added to the network.

3.3 Description of unsteady flow in links

Flow through the open channels and pressurized pipelines, is computed by MIKE URBAN using the 1D Saint Venant's equations. The Saint Venant's equations, as used in MIKE URBAN, as follows (DHI, 2016b):

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (3.1)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial \left(\alpha \frac{Q^2}{A}\right)}{\partial x} + gA\frac{\partial y}{\partial x} = gA(I_0 - I_f) \qquad (3.2)$$

Equations 3.1 and 3.2 are the equations for conservation of mass (continuity equation) and momentum (momentum equation) respectively

Where,

Q = flowrate (m^3/s)
A = Flow area (m^2)

x = distance in the direction of flow (m)
y = depth of flow (m)
t = time (s)
g = acceleration due to gravity (m/s^2)
I_0 = slope of channel bottom
I_f = friction slope = slope of energy line
α = velocity distribution coefficient

Equation 3.1 conveys that the change in water entering a certain length of pipe is balanced by a change in flow area.

The momentum conservation equation takes inertia, pressure, gravity and frictional forces into account. The velocity distribution coefficient is used to relate the change in momentum due to unevenness in the velocity distribution across the channel section. It is given by equation 3.3, where *v* is the average velocity (m/s).

$$\alpha = \frac{A}{Q^2}\int_A v^2\, dA \tag{3.3}$$

The friction slope I_f is introduced into the formula in the form of the Manning equation, which is as follows:

$$I_f = \frac{Q|Q|}{M^2 A^2 R^{4/3}} \tag{3.4}$$

Where:
M = manning number
A = area
R = hydraulic radius

The use of Q|Q| instead of Q^2 for flow allows for computation of reverse flow (DHI, 2016b).

The 1D Saint Venant equations - 3.1 and 3.2 - are only strictly valid for open channel flow, due to the assumptions that they are based upon.

The continuity and momentum equations to be for pressurized pipe flows are equations 3.5 and 3.6 respectively (Leandro, 2008).

$$\frac{c^2}{g\,A}\frac{\partial Q}{\partial x} + \frac{\partial H}{\partial t} = 0 \tag{3.5}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\alpha \frac{Q^2}{A}\right)}{\partial x} + g\,A\,\frac{\partial H}{\partial x} = g\,A\,(I_0 - I_f) \tag{3.6}$$

Where:
H = piezometric head

C = celerity of the pressure waves

As seen, the equations for open channels and pipe flow are different and accommodating all these equations in one model for a mixed system would be complicated (Leandro, 2008). Therefore, in Mike URBAN, the closed conduits are modified to have a fictitious slot on the top so that they can be modelled using equations 3.1 and 3.2. MOUSE allows the pressure in a closed conduit to increase infinitely, thus the height of the slot can be infinite (DHI, 2016b).

4 Previous Runoff Studies in Augustenborg

Shukri (2010) modelled the open storm water system of Augustenborg using the MOUSE engine in MIKE URBAN by DHI. The model was calibrated and then three different scenarios were simulated. 1) The current conditions, 2) a comparison between the open storm water system and a hypothetical pipe system put in its place based on the discharge hydrograph at specific locations, and 3) a comparison between the open storm water system and the same hypothetical pipe system based on node flooding during extreme rainfall events. The results showed that the open storm water model simulated more lag and 40-50% lower discharge than the conventional system. The study also showed that the open storm water system was more adept at dealing with extreme flooding events.

Kibirige and Tan (2013) developed a hydrological model of Augustenborg using MIKE SHE by DHI. The model was not calibrated or validated since there was no observed data available for this study. They simulate three scenarios – 1) 1 year prediction of hydrological balance, 2) 10 year extreme rainfall event and 3) 100 year extreme rainfall event and studied the results based on runoff and water balances. The purpose of this study was to evaluate the open storm water system and therefore there is no study of or comparison with the pipe system.

Nordlöf (2016) developed a coupled 1D/2D model of the storm water system at Augustenborg using MIKE FLOOD by DHI. In this study, pipe flow, open channel flow and runoff from effective impervious surfaces are modelled in a 1D model while overland flow and infiltration are modelled in a 2D model. The two models were then coupled to permit exchange of water between them. This type of model is very useful especially when SuDS is used since it can simulate runoff that is handled locally on the surface. The present work will build on this model.

4.1 Advantages and disadvantages of choosing a 1D model

The biggest problem with using a 1D model to simulate open drainage systems is that the model cannot handle surface flow. Thus, if the water level in an open channel exceeds the height of the channel, the model perceives this as an error and halts the simulation.

In order to counter this problem, the 1D model must be coupled with a 2D surface model as was done by Nordlöf (2016). 1D/2D coupled models permit the exchange of water between the pipe system modelled in MIKE URBAN and a DEM of the ground surface and can thus simulate the flow of water on the surface of the ground.

The disadvantage of using a coupled model is that 2D models often require a large simulation time, as it considers more parameters. The 2D model treats the entire surface as a grid comprised of cells in a 2-dimensional matrix, and for each simulation, calculations must be performed on every cell. Thus, the time required to complete a simulation may be in the order of a few hours, depending on the size of the catchment and complexity of the model.

The 1D model however, typically requires a simulation time that is of the order of a few seconds to a few minutes since there are fewer parameters involved. Using the current model, the network simulation of a 39 day rain series takes just about 1 minute to run. Thus, it is greatly beneficial to have a working 1D model of a catchment, as it provides a good estimate of the flow emerging from the system in a very short time. Based on the availability of data and the goals of the project, it was decided that a 1D model would suffice and MIKE URBAN by DHI was chosen as the software.

5 Methodology

5.1 Study Area

The area that this project focuses on is an Eco-city in Malmö, Sweden known as Augustenborg. It is a residential district constructed by MKB Fastighets AB (a Malmö based property management company) in the 1950s, about 20 ha in size inhabited by over 3000 people. Historically, this area was facing an increasing problem of unemployment, flooding and relocations. By the 1970s, the buildings and drainage systems of this area had become very outdated and it had lost its appeal (Stahre, 2008).

In the 1980s, the municipality of Malmö along with the housing agency MKB began renovating Augustenborg into Eco-city Augustenborg in order to improve the status of the area by turning it into a 'socially, ecologically and economically sustainable settlement' (Stahre, 2008). The renovations included shifting to more renewable energy sources wherever possible and employing Sustainable Drainage Systems (SuDS) in parts of the catchment. The SuDS was an important part of the renovation due to the flooding problem. The SuDS consists of green roofs and open surface water systems like ponds, swales, canals etc.

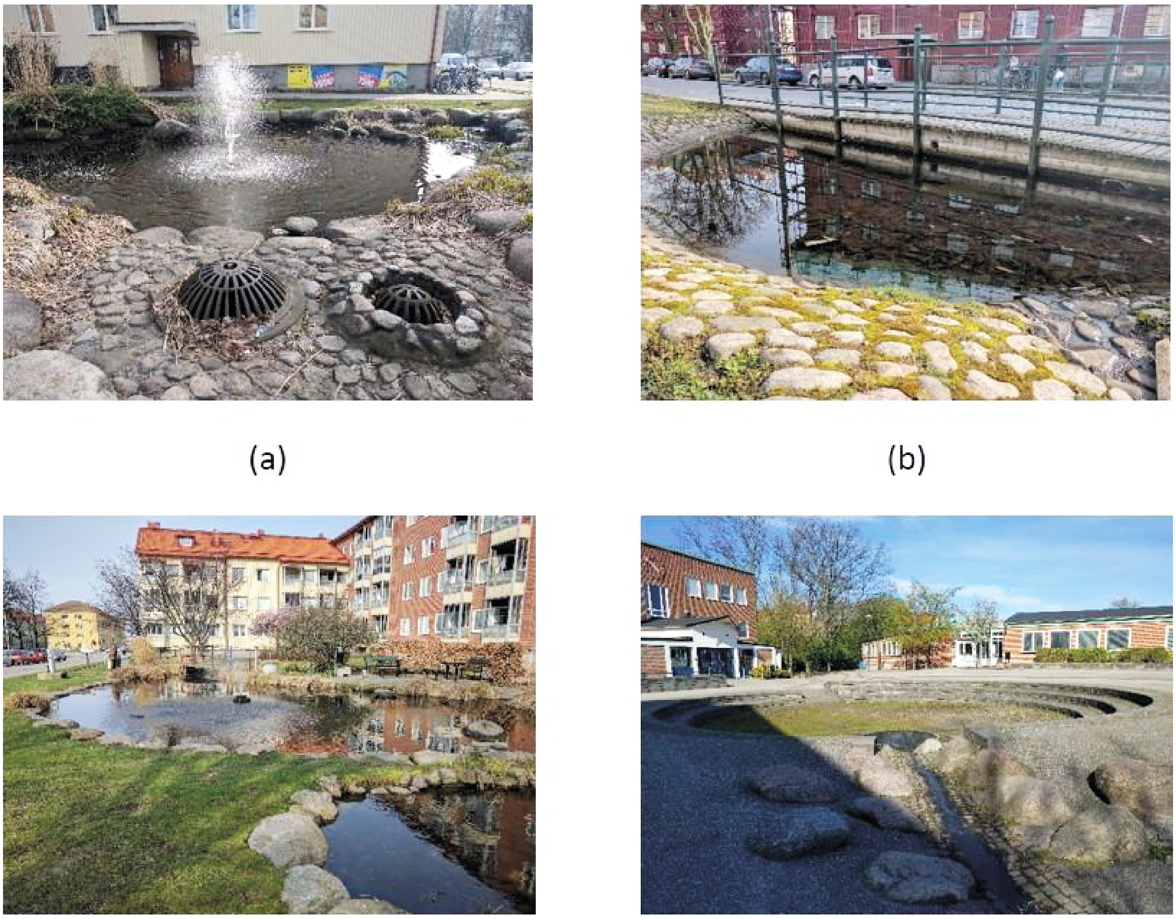
The Eco-city Augustenborg has won awards such as the United Nations World Habitat Award 2010 (BSHF, 2016). According to the European Environment Agency (EEA), the ability of this area to deal with flood events has significantly improved as a result of the development of this eco-city (EEA, 2016). Additionally, the unemployment in this region has reduced and Augustenborg has emerged as an international hub and example for those interested in green roofs (Stahre, 2008).

The Drainage System: Augustenborg originally had a combined sewer system that would get overloaded during heavy rainfall events and result in flooded basements. During the renovation, open storm water handling systems were installed in order to detain rainfall as much as possible near the source before releasing it into the sewer system. This gives the water more time to infiltrate and evaporate. The open storm water network employs the following

- **Canals**: Canals are open channels that slow down, retain and transport the water from the site. They can be concrete channels with rectangular cross sections of various depths (Figure 5.1 a – d) or swales with grassy or rocky bottoms (Figure 5.1 e and f). Some channels have obstacles at the base to delay the flow like the 'onion' gutters and the 'cube' canal. These are seen in Figure 5.1 (g) and (h) respectively.
- **Infiltration**: Infiltration of rainfall is ensured largely by the installation of a large number of green roofs including the botanical roof gardens. Figure 5.1 (i) shows a typical extensive green roof seen over a number of sheds in the region. Additionally there are also parks, grassy areas and permeable parking lots.
- **Detention and surface storage**: The runoff from the roofs and impervious areas flows through open channels into detention ponds. Figure 5.2 (a) is such a pond in the northern part of Augustenborg. Figure 5.2 (b) shows a culvert situated below a road so that the open drainage system can go across the street without causing any inconvenience. There are some areas in Augustenborg designed to detain water especially in case of a heavy flooding event. These include the grassy areas surrounding the double ponds, the amphitheatre (of the Augustenborg School), and the 'meandering creek' seen in Figure 5.2 (c) and (d) and Figure 5.1 (e) respectively (Nordlöf, 2016). These structures will fill

up with water during heavy rainfall events and prevent nearby structures and building from being affected.

Figure 5.1 Components of the open drainage system in Augustenborg that slow the flow; (a) – (h): Various open channels, (i): Extensive green roof.



(a) (b)

(c) (d)

Figure 5.2 Surface storage facilities in Augustenborg.

The current drainage network of Augustenborg can be divided into three different systems - the northern SuDS, southern SuDS and the pipe-bound system. The excess runoff from the open systems eventually drain into the municipal sewer network to be transported out of Augustenborg.

Table 5.1 Distribution of surfaces in the Augustenborg drainage systems

Surface Type	Pipe-bound System (ha)	Northern SuDS (ha)
Tiled Roof	0.5	1.7
Asphalt or Concrete	2	1.5
Grass	1	2.9
Green Roof	0	0
Sand	0	0.1
Gravel	0	0.1
Total Impervious Area	***2.5***	***3.2***
Total Pervious Area	***1***	***3.1***
Total	***3.5***	***6.3***

The distribution of different types of surfaces in the drainage systems are as seen in Table 5.1. In this project, impervious surfaces refers to tiled roofs, asphalt and concrete. All the other surfaces are considered as pervious.

Figure 5.3 is a map of Augustenborg showing the areas with different drainage systems. This case study focuses on the Northern SuDS (NS) and Pipe-bound system (PS) regions. The southern SuDS was excluded from this study since it includes large storage volumes and broad infiltration surfaces, which make it difficult to simulate in 1D.

Figure 5.3 Map of Augustenborg marking regions with different drainage systems. Source: Published with kind permission from Salar Haghighatafshar (Haghighatafshar et al., n.d.).

5.2 Data Collection

Doctoral student Salar Haghighatafshar at the Department of Chemical Engineering, Lund University has recorded rainfall, water level and flow (runoff) data from eleven measurement stations in Augustenborg. There is data available from these stations from June 2015 to December 2016. The flow readings have been recorded for every minute and the water level has been recorded every 10 minutes. The rainfall gauge records every 0.2 mm of precipitation. Calculations will be performed using this data to compute the runoff coefficients.

The details about the rainfall events used in this study are described below in Table 5.2. The hydrographs and hyetographs for these rainfalls are presented in Appendix I. Rainfall events R1, R2 and R3 are single rainfall events. R4 extends over a long period of about 39 days and contains multiple small rainfall events. It is used to study the performance of the model when applied to a discontinuous rainfall. The return period was calculated by Haghighatafshar *et al.* (n.d.) using the Dahlström equation to take Swedish conditions into account (Dahlström, 2010).

Table 5.2 Measured rainfall data used in this project.

Rainfall Event	Start	Stop	Depth (mm)	Volume Inflow (m^3/ha)	Return Period (months)
R1	2015-08-04 21:05:14	2015-08-05 01:52:54	15.6	156	4.7
R2	2016-06-23 21:47:57	2016-06-24 06:55:42	28.4	284	19.2
R3	2016-06-29 10:18:48	2016-06-29 14:28:04	22.8	228	21.7
R4	2016-07-30 07:44:32	2016-09-05 02:26:04	73	730	N/A

5.3 Determination of runoff coefficients (φ) using measured data

The runoff details regarding the rainfall events are as follows in Table 5.3. The runoff volumes from each system were calculated using the measured data from the site. The recording stations records flow (of runoff) every minute. The area under this curve was calculated using the trapezoidal method for each time step. This yields the total runoff volume.

Table 5.3 Measured rainfall-runoff data.

Rainfall Event	Total Runoff Volume from NS (m^3)	Total losses from NS (m^3)	Total Runoff Volume from PS (m^3)	Total losses from PS (m^3)
R1	96.45	886.35	174.39	371.61
R2	288.55	1500.65	352.19	641.81
R3	258.05	1178.35	269.16	528.84
R4	356.07	4242.93	688.11	1866.89

It was attempted to obtain a unique value of runoff coefficient that can be used for pervious surfaces and impervious surfaces respectively. This is done by solving the following system of equations for each rainfall event.

$$(A_{imper\_PS} \times \varphi_{imper} + A_{per\_PS} \times \varphi_{per}) \times V_{rain} = V_{runoff\_PS} \tag{5.1}$$

$$(A_{imper\_NS} \times \varphi_{imper} + A_{per\_NS} \times \varphi_{per}) \times V_{rain} = V_{runoff\_NS} \tag{5.2}$$

Where,

A_{imper_PS} and $A_{imper\ NS}$ are total *impervious* areas in the PS and NS respectively (ha).
A_{per_PS} and A_{per_NS} are total *pervious* areas in the PS and NS respectively (ha).
φ_{imper} and φ_{per} are the runoff coefficients for impervious and pervious surfaces respectively.
V_{runoff_PS} and V_{runoff_NS} are total runoff volumes from PS and NS respectively (m^3)
V_{rain} is the rainfall per unit area for the event under consideration (m^3/ha)

Equations 5.1 and 5.2 are re-arranged forms of the rational method with volumetric terms to remove the time dependency.

A second run of these calculations was done using DCIA of the NS and PS instead of TIA for the terms A_{imper_PS} and $A_{imper\ NS}$. DCIA was calculated using Sutherland equations 2.3.2 and 2.3.4 respectively (described in Section 2.3) for the PS and NS. The DCIA and its relationship with TIA for each system is seen in Table 5.4. The Sutherland equations was chosen instead of other methods available in literature since it differentiates between sub-basins with different levels of connectivity to the drainage system. It is assumed that this differentiation will reduce the error.

Table 5.4 Relationship between DCIA and TIA.

	TIA	DCIA	DCIA/TIA
PS	3.5	2.35	0.67
NS	3.2	2	0.625

5.4 MIKE URBAN Model

The rational method is typically employed to calculate just the peak flowrate or the runoff volume that is generated. When the dynamics of the flow are needed, computer models ca be used since they can simulate changes in flow with peaks and time lags as seen in reality. The MIKE Urban Model being used for this project was originally developed as part of a master's thesis at Lund University by Aza Shukri in 2010. A brief description of the model is presented below. For a more detailed explanation of the construction of the model, the reader is referred to the report by Shukri (2010).

5.4.1 The Runoff Model

The present runoff model uses the Time-Area method for a rectangular catchment. The parameters required by the time-area method are imperviousness, initial loss, hydrological reduction factor, time-area curve and time of concentration. The study area is modelled as a number of catchments that are classified based on their land-use type. This makes it possible to adjust the imperviousness of each catchment based on the type of surface/land-use. The various land use classifications used in this model are seen in Figure 5.4.

Precipitation data was available for the period mentioned in Section 5.2. The hyetographs of the rainfall events used in this project are seen in Appendix I. The locations of the flow meters and rain gauges are shown in Figure 5.3.

Figure 5.4 Classification of catchments based on land use.
(Created using ArcMap)

5.4.2 The Hydraulic Network Model

The hydraulic network model consists of links and nodes that represent the geometries of the pipes, canals, basins and manholes in the system. Each catchment must be connected to a node, so that the runoff accumulated on the catchment from the runoff model can be transferred into the network. The hydraulic network used in this project is seen in Figure 5.5.

Links: The conventional pipes and open channels are represented as links. In the pipe system (PS), the links are given spherical cross sections and are defined by their UpLevel, DwLevel, and length. The irregular cross sections of the open channels are defined using the CRS editor. The method to define a CRS is explained in Section 3.2.1.

Nodes: The junctions and the points where storm water enters the system are modelled as circular nodes. Ponds are modelled as basins. 'Wet ponds' which have a permanent water level are modelled so that the downstream pipe leaves the pond at the permanent water depth required. This was set manually.

Some of the open drains in the site have sections along the channel that are broader than the rest of the channel, and serve to further store and detain water during very heavy rainfalls. These small expansions are modelled as dry ponds along the length of the channel. For dry ponds, the level at which the downstream pipe leaves the pond must be the bottom of the pond.

As seen in Figure 5.5, there is one output node in the network, which is located further downstream of all the measurement stations.

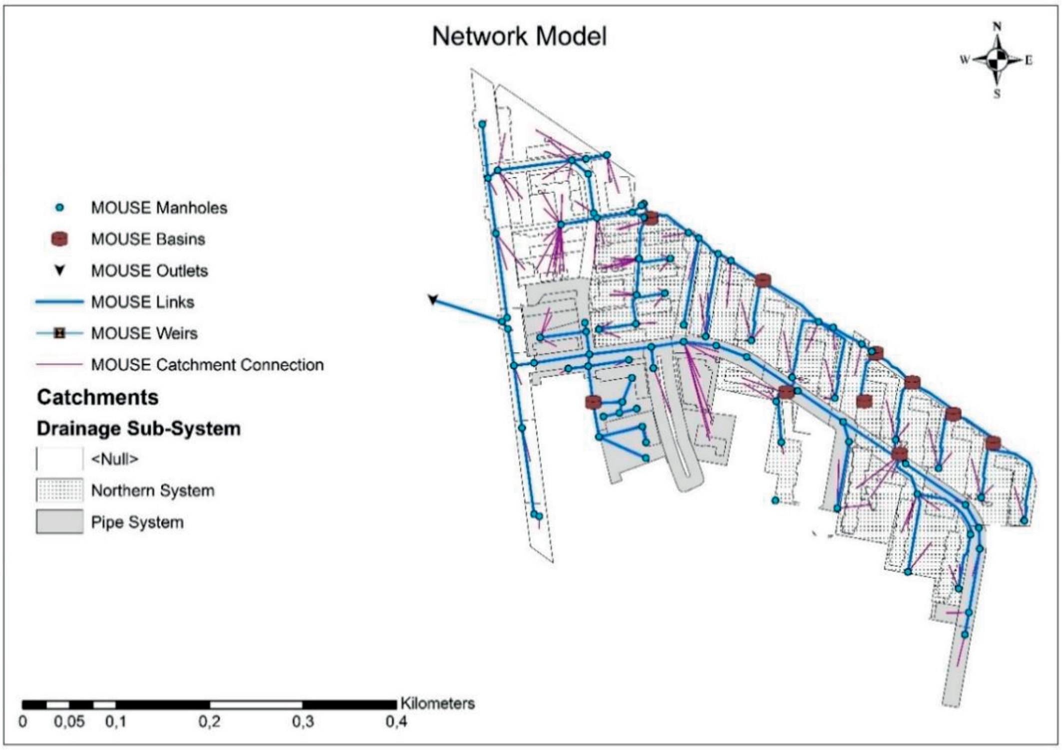


Figure 5.5 Components of the network model.
(Created using ArcMap)

5.4.3 Changes made to the previous models

The present work uses a purely 1D model of Augustenborg. Therefore it uses the model by Nordlöf (2016) as a base and retains some of the modifications made in that study such as the updated elevation data, updated basin geometry and updated CRS cross sections. The 2D makes use of a Digital Elevation Model (DEM) to identify depressions on the ground surface and model them as sinks or storages. However, since this is not a 2D surface model, additional basins were added along the open ditches in order to simulate their storage capabilities. This is similar to the network model created by Shukri (2010).

The ground elevations of many of the open channels were found to be below the surface and were corrected in order to raise them to the surface. Additionally some of the open drains were given exaggerated false cross sections in order to facilitate the running of the model without errors. This exaggeration is very marginal and does not affect the results much.

5.5 Calibration and Verification of the model

The imperviousness of catchments for the MIKE URBAN model was selected based on values of φ_{imper} and φ_{per} obtained from the using equations 5.1 and 5.2. The selection of these parameters is explained in the results section. The initial loss and mean surface velocity were adjusted to match the time lag between observed and simulated values. The model calculates time of concentration automatically. All the other parameters - Manning's number for each type of open channel and infiltration rate - were left unchanged from the original model by Shukri (2010).

The rainfall time series R1 was used for calibration and the model validation was completed with R2, R3 and R4.

6 Results and Discussion

6.1 Runoff coefficient (φ) values from measured data

When manually computing peak runoff rate using the rational method, the runoff coefficient is usually chosen from values available in design-manuals (such as those presented in Table 2.1) based only on the type of surface. There are many interpretations for what this coefficient implies- the runoff coefficient could either be a volumetric ratio relating runoff to rainfall depth or a factor used for area reduction. It is assumed that this value incorporates all the phenomena involved in runoff generation (such as infiltration and evapotranspiration) from the surface. No consideration is given to how connected the surface is to the storm drainage system. This is important because even if a surface is highly impervious, it may not contribute to the runoff at all unless it is connected to the drainage system. Additionally, the true runoff coefficient of a particular surface can vary based on a number of parameters such as the material, slope, presence of surface depressions, soil moisture content, etc. Thus, even different surfaces of the same material, such as two different tiled roofs, can have different runoff coefficients in reality, which is indicated by the proportion of runoff generated from the roofs that ultimately reaches the monitoring point/outlet. The runoff coefficients of each surface will also change depending on a particular rainfall event. (Sriwongsitanon and Taesombat, 2011; Merz *et al.*, 2006; Gottschalk and Weingartner, 1998).

One of the key principles of SuDS is to reduce/delay the volume of storm water entering the traditional piped drainage system. This is achieved by disconnecting areas from the existing pipe network and retaining the water at the surface as much as possible. Thus, manual calculation of runoff rate from SuDS using runoff coefficients like those in Table 2.1 does not really make sense.

To overcome these drawbacks, in this study measured rainfall and runoff volumes were used to calculate the runoff coefficients specific to the site. This was done by solving equations 5.1 and 5.2 and the results are presented in Table 6.1.

Table 6.1 φ values calculated from measured data.

Rainfall Event	Using TIA		Using Sutherland DCIA for impervious areas	
	φ_{imper}	φ_{per}	φ_{imper}	φ_{per}
R1	0.63	-0.45	0.54	-0.15
R2	0.62	-0.31	0.54	-0.02
R3	0.56	-0.21	0.48	0.06
R4	0.54	-0.39	0.46	-0.14

The solutions to equations 5.1 and 5.2 using the measured data (using total pervious and impervious areas) sometimes resulted in negative φ_{per} values, as seen in Table 6.1. By definition, runoff coefficients have a value ranging from 0 to 1. Thus, negative runoff coefficient values are not applicable in a realistic sense but could convey some information regarding the runoff patterns, which will be discussed in this section.

The results obtained upon using Sutherland DCIA instead of total impervious area (A_{imper_PS} and $A_{imper\ NS}$ in equations 5.1 and 5.2 respectively) can also be seen in Table 6.1. The rainfall events R1 and R2 still result in negative φ_{per} values. However, R3 has a positive value solution. It is of interest to note that φ_{per} values approach reality (≥ 0) upon using the DCIA, and are essentially equal to 0 for R2 and R3 which are the larger rainfall events used in this study.

The calculated values of runoff coefficients from the observed data is much smaller than the literature values presented in Table 2.1. This is in accordance with evidence amassed by scientists from analysing a great variety of storms over numerous watersheds varying considerably in size (Cleveland *et al.*, 2011). Kuichling (1889) states that if a solid surface has any degree of perviousness, the volume of incoming precipitation must be great enough at least to saturate the soil in order to generate runoff.

The inability to obtain reasonable solutions to these equations could be owed to the fact that these are very small rainfall events. In such cases, rainfall contribution to catchment runoff is very low as considerable surface storage, infiltration and evapotranspiration will occur both before and during rainfall events. It would also take a considerable duration and/or quantity of rainfall before the catchment is sufficiently hydraulically connected to the outlet in order to be explicable by flow equations. Additionally, flow measurements are more prone to error in the case of very low flows (Arregui *et al.*, 2007).

This logic follows with the fact that a reasonable solution could be obtained for R3 since this event occurs just a few days after R2. Thus, due to R2 (and possible rain in the days in between) providing the required antecedent properties to the soil and hydraulic connectivity to the catchment, a solution is possible for R3. To test this, longer periods of flow measurements (starting from the onset of a rainfall event) must be analysed.

Moreover, it is very interesting to note that the optimal solution could be achieved only when DCIA was used instead of TIA. This indicates that indeed only a portion of the impervious areas contribute to runoff, thus emphasizing the importance of estimating DCIA in addition to TIA.

A new fractionation model for the catchments was drawn up based on the findings from literature, and from observations made during this project. The fractionation model is illustrated in Figure 6.1. It depicts surfaces that constitute EIA. An 'active' area is one that is *hydraulically* connected to the drainage channel. As rainfall events become heavier, the pervious areas (PA) contribute more to the runoff and the 'active PA' increases which in turn activates more impervious areas. Thus, for heavy rainfalls, inactive impervious areas that are not directly connected to the drainage channel (TIA–DCIA) may become activated and become a part of the EIA. This relationship is depicted in Figure 6.1 as the dashed line connecting 'TIA–DCIA' with 'Active PA'. The active and inactive areas are thus dynamic and change with the intensity and volume of the rainfall event.

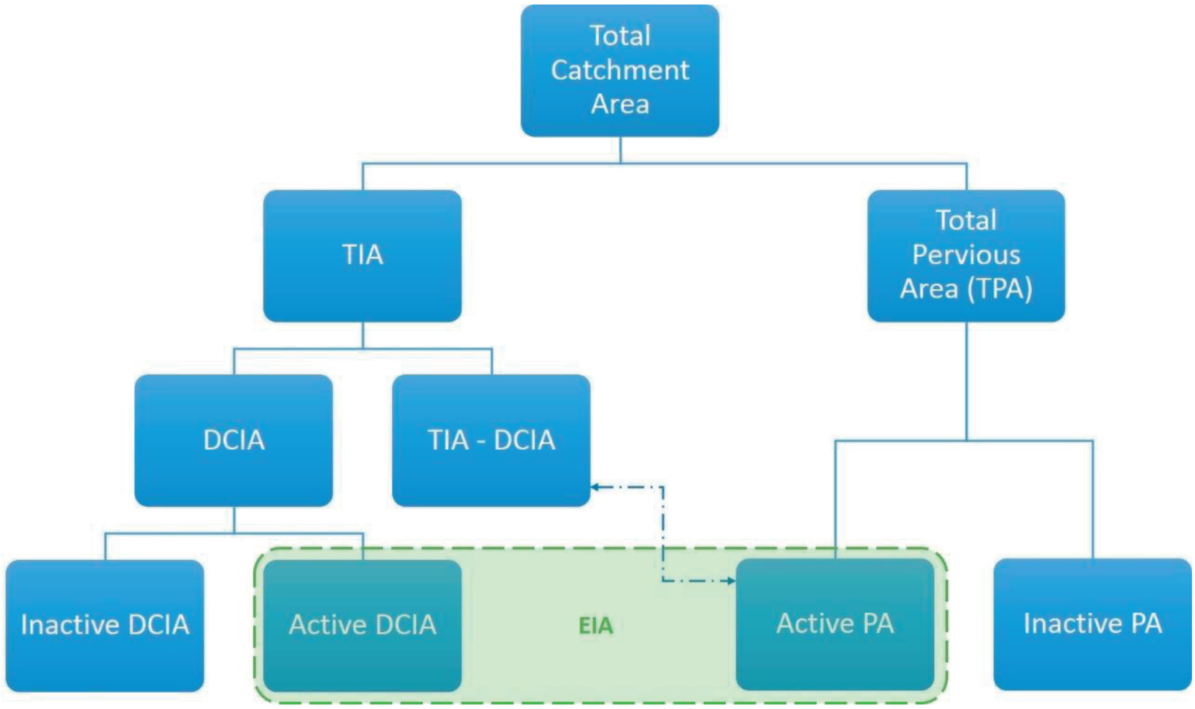


Figure 6.1 Fractionation model for areas contributing to runoff.

Errors are possible in the calculation of runoff coefficients above since the Sutherland equations were not designed for the Augustenborg catchment specifically. If equations relating DCIA and TIA could be obtained for these catchments specifically, it could be possible to get results that are more accurate. This is a very intensive GIS work especially for the northern system since it is difficult to estimate what fraction of the impervious surfaces effectively contribute to the runoff.

Since pervious surfaces constitute about 49% of the NS but only 28% of the PS (refer Table 5.1), the value of φ_{per} affects the calibration of the NS more than that that of the PS. Furthermore, not all the impervious surfaces in the NS are directly connected to the drainage system, and many of the impervious surfaces are routed into the drainage channel through pervious surfaces.

Conversely, the response of the pipe system is dependent more on $\varphi_{imper.}$ Impervious surfaces - mainly the asphalt street surfaces that are connected directly to the storm water drain -constitute 78% of the PS (Table 5.1). Thus, the values of φ_{imper} are always positive as seen from Table 6.1.

6.2 Runoff response mechanism of PS and NS

By studying measured rainfall and runoff data and observing the patterns that emerge, it is possible to understand a great deal, about how the PS and the SuDS in the NS respond to rainfall events and generate runoff.

It is observed that the response of the SuDS, in terms of initiating outflow, depends on the volume inflow of rain and not the intensity of the incoming rainfall. Overflow from the SuDS occurs every time a certain threshold volume is crossed and the final pond overflows. This threshold volume includes the storage capacity of the final pond and entire SuDS upstream from it. The time and volume required to cross this threshold are also controlled by the rate of infiltration and evapotranspiration relative to the incoming rainfall intensity. Thus, using just the intensity of a rainfall could be insufficient to predict the response of the SuDS. This is the reason why predicting runoff from SuDS using the conventional methods like the rational method are inadequate unless measured rainfall-runoff data is available.

Upon examining the measured rainfall-runoff data, it was observed that the depth of rainfall required to initiate runoff in the northern system was approximately 9.6 mm, 9.8 mm and 10.4 mm respectively for R1, R2 and R3. This is almost constant at a value of 10 mm. R4 consists of a series of very light rainfalls. It was the only period available within the measured data where a number of consecutive rainfalls occurred within a span of one month. The rainfall depth required to activate the SuDS was not consistent for these separate events within R4. This could indicate either measurement errors, or the effect of other factors such as antecedent soil moisture.

The outflow from the NS is controlled by just the outlet from the final pond in the system, that is, the capacity of the final pond. Thus, the minor variations in flow and blockages that occur before the final pond and the rainfall intensity variations do not reflect in the final hydrograph. This is the reason for the relatively smooth outflow hydrographs from the NS that are seen in Appendix I.

Haghighatafshar *et al.* (n.d.) proposed a conceptual model to explain the runoff response mechanism of SuDS. They suggest that the response of a SuDS can be discretised into several disconnected 'mini-catchments'. As the volume of a rainfall event increases, the capacities of the mini-catchments are exceeded and they become hydraulically connected to one another, provided that rainfall intensity is higher than the rate of evapotranspiration and infiltration. The final outflow from the system occurs when the capacity of the ultimate mini-catchment (such as the final pond in the present study site) is exceeded. The larger the storage capacity of the final mini-catchment (in comparison with the capacity of the rest of the SuDS), the smoother the outflow hydrograph will be.

A small initial threshold volume will be required to activate the pipe system as well, just like in the case of the SuDS. This volume depends on the elevation of the storm drain with respect to the surrounding surface that it drains. However, this threshold volume is very small for pipe systems since storm drains are generally at or slightly below ground level. With the threshold volume being so low, the response pattern of the PS is then dominated by the rainfall intensity. The final outflow hydrograph of the pipe system represents a cumulative effect of the water entering the PS from each storm drain of the system. Since the surfaces connected to the PS mostly consist of DCIA, the response is almost immediate. Each time the rainfall intensity increases, the water entering the pipe system increases and the outflow swiftly increases, and vice versa. Thus, every minor upstream fluctuation in flow somehow affects the outflow hydrograph of the PS. This results in a jagged outflow hydrograph whose shape mimics the rainfall hyetograph. This effect is seen clearly in the PS graphs in Appendix I.

6.3 Model Calibration and Verification

The current model is an attempt to simplify the calibration procedure by computing just two runoff coefficients - one for all pervious surfaces and another one for all impervious surfaces – using measured data, and then using these as the starting point for the model calibration. To combine the results of section 6.2 with the model, the highest of φ_{imper} with Sutherland DCIA from Table 6.1 (0.54) was used as the initial runoff coefficient for all impervious surfaces in the model and the highest value of φ_{per} using Sutherland DCIA (0.06) was set as the initial runoff coefficient for all pervious surfaces in the model. The runoff coefficient of a catchment is represented in Mike UBRAN by the 'imperviousness' parameter.

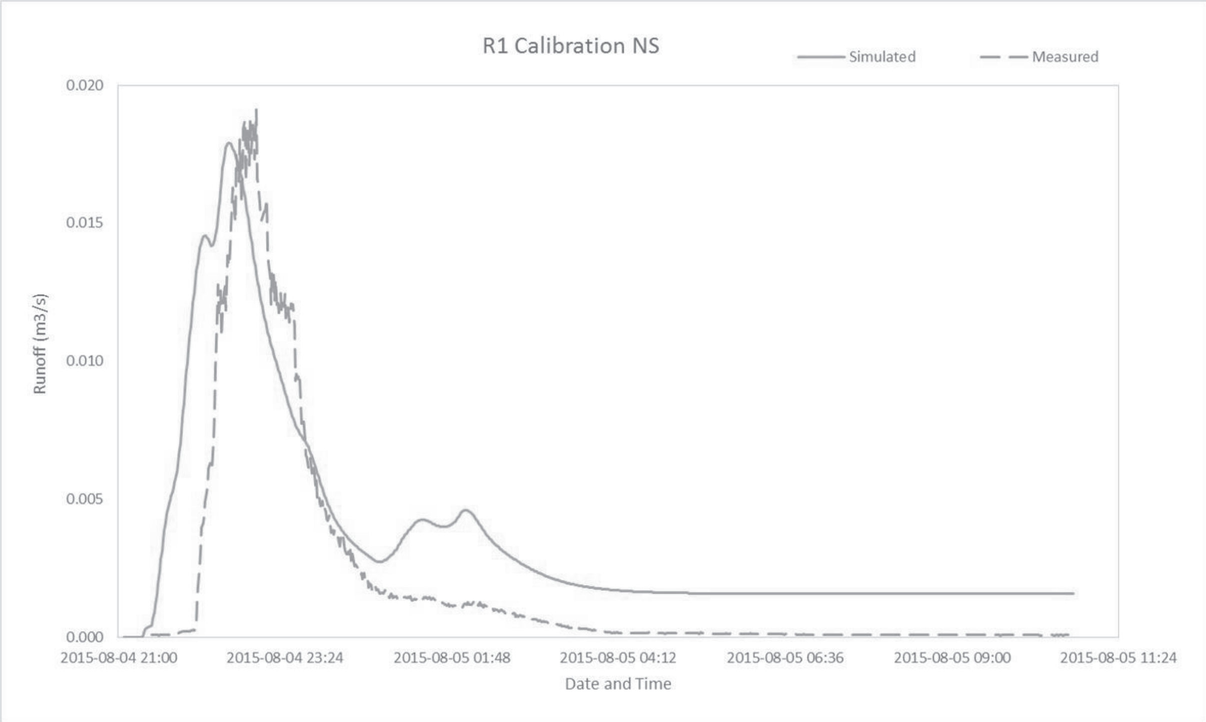
The model was calibrated using rainfall R1. On running the model with the initial runoff coefficients, it was found that the peak values and volumes of observed data did not match the simulated data in the NS. As seen in Table 6.1, the calculated values of φ_{per} are inconsistent – there is a difference of 0.21 between the maximum and minimum calculated value (as opposed to just 0.08 for φ_{imper}), and some of the values are negative, which cannot be modelled. Thus, the value of φ_{per} was deemed unreliable and pervious runoff coefficient was calibrated further by trial and error until the peak error was within 10% for the NS. This calibration of φ_{per} does not affect the calibration of the pipe system significantly, as the PS is majorly comprised of impervious surfaces. The final calibrated value of φ_{per} was not allowed to exceed 0.1 so as to stay within the limits for parks and lawns presented in Table 2.1.

The hydrological reduction factor was maintained at the default value. The surface velocity and initial loss were adjusted to reduce time lag. The final parameter values after calibration are seen in Table 6.2.

Table 6.2 Final calibrated parameter values.

Parameter	Value
Runoff coefficient for impervious surfaces	0.54
Runoff coefficient for pervious surfaces	0.10
Initial loss (mm)	3
Mean surface velocity (m/s)	0.6
Hydrological reduction factor	0.9

The result of the calibration using R1 is seen in Figure 6.2. The model was validated using R2, R3 and R4. The results are presented in Figure 6.3, Figure 6.4 and Figure 6.5 respectively. The R^2 value volume error and peak error of each simulation is presented in Table 6.3. The graphs are all plotted with runoff rate (m^3/s) on the y-axis, and date and time on the x-axis (YYYY-MM-DD hh:mm).



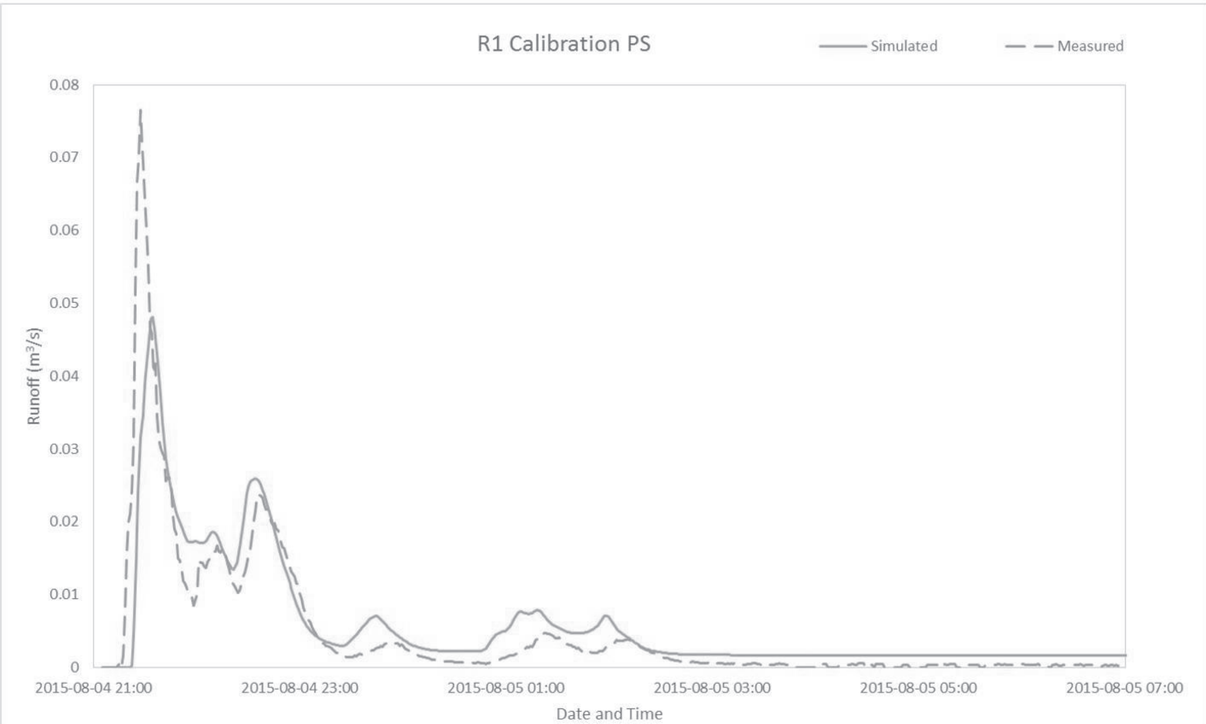
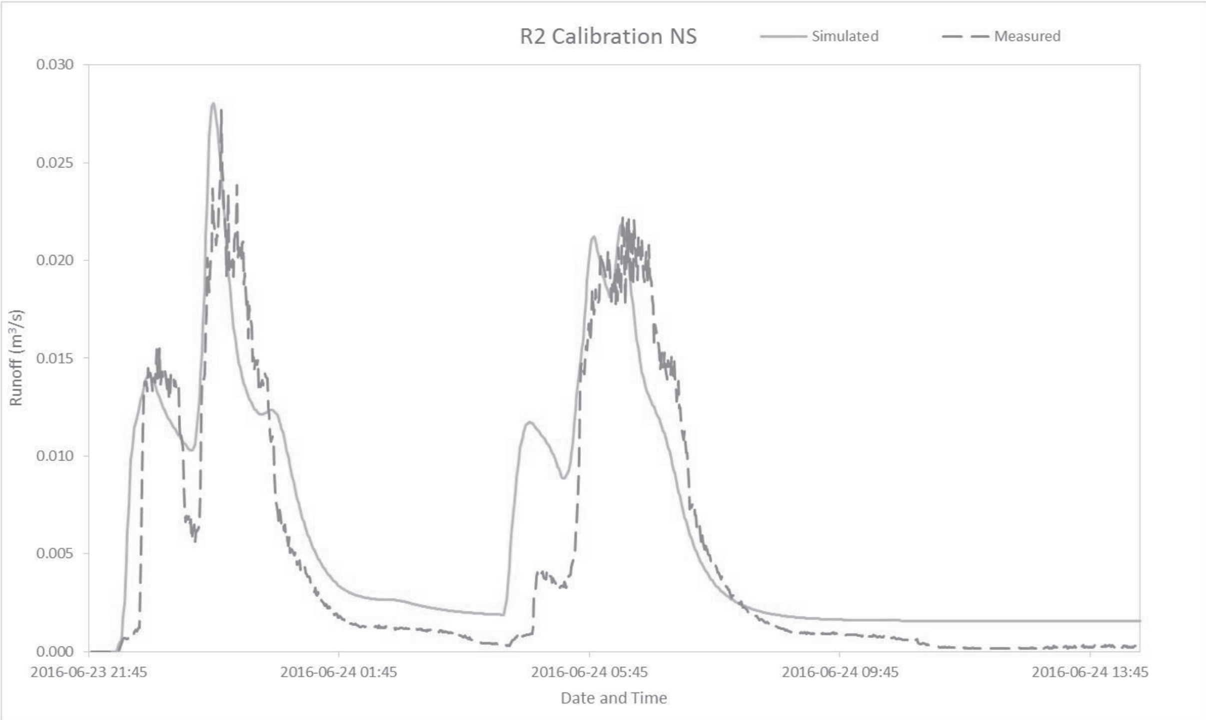


Figure 6.2 Calibration using R1 for NS (top) and PS (bottom)

The calibration shows a good correlation between the observed values and simulated values, especially for the NS.



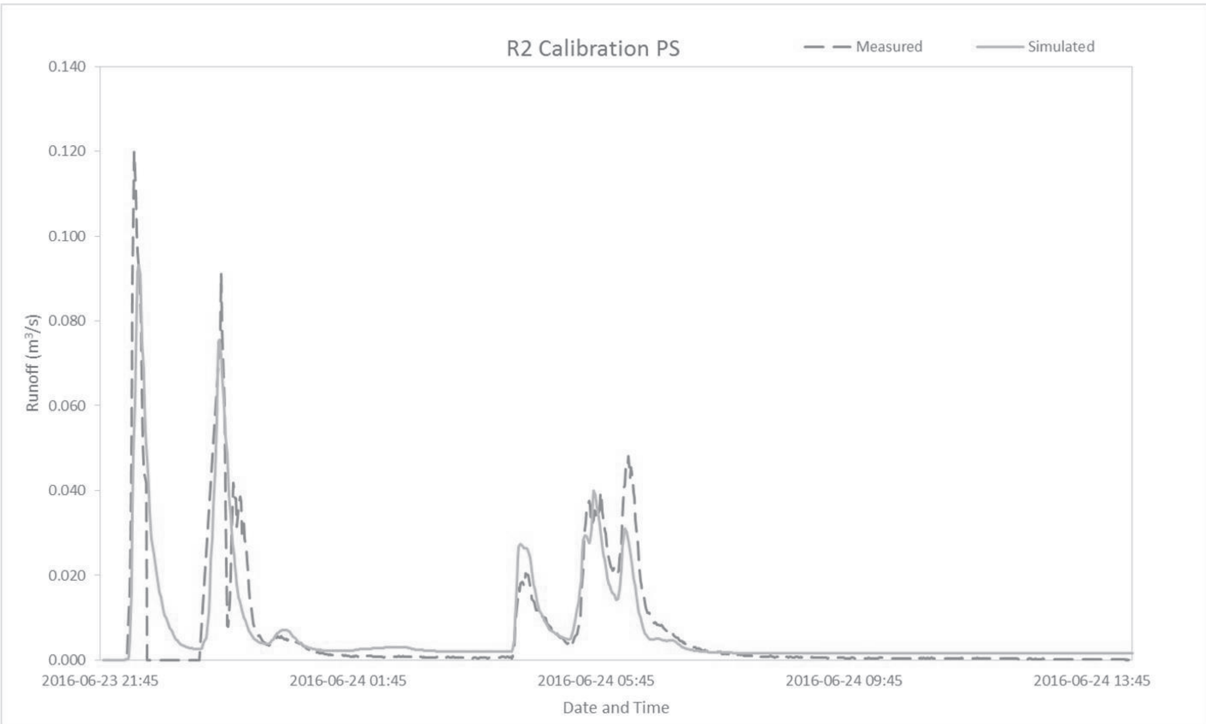
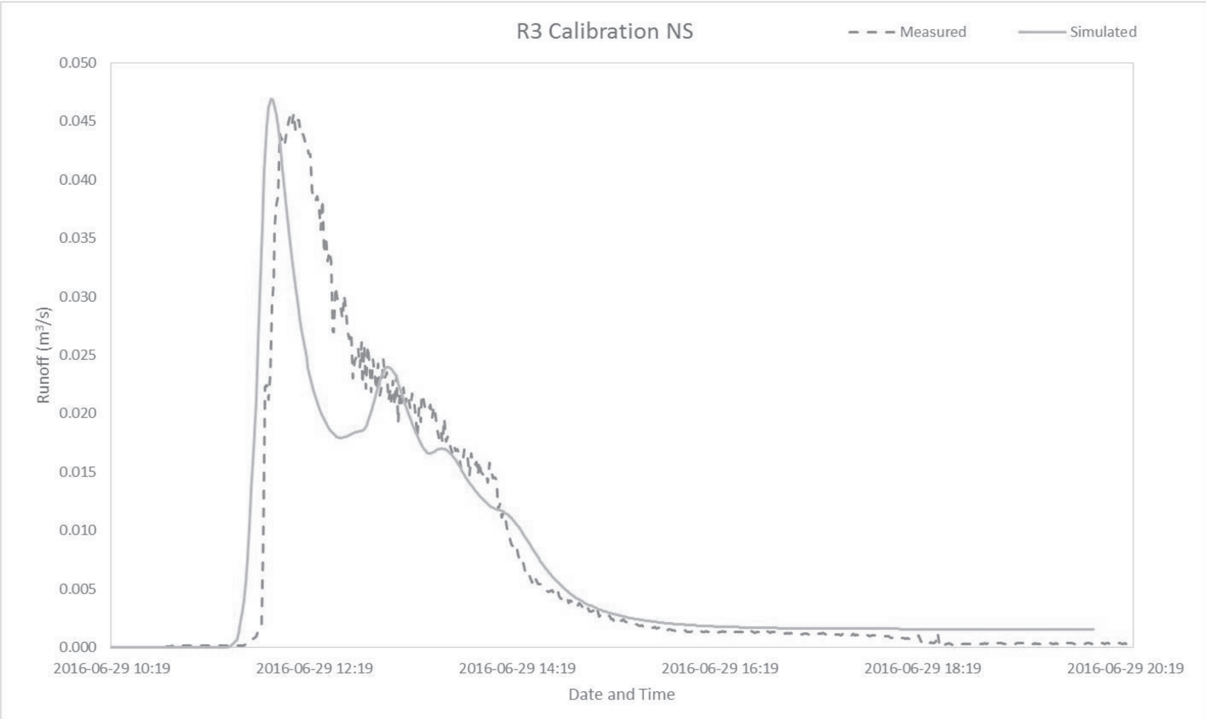


Figure 6.3 Verification of calibration using R2 for NS (top) and PS (bottom).



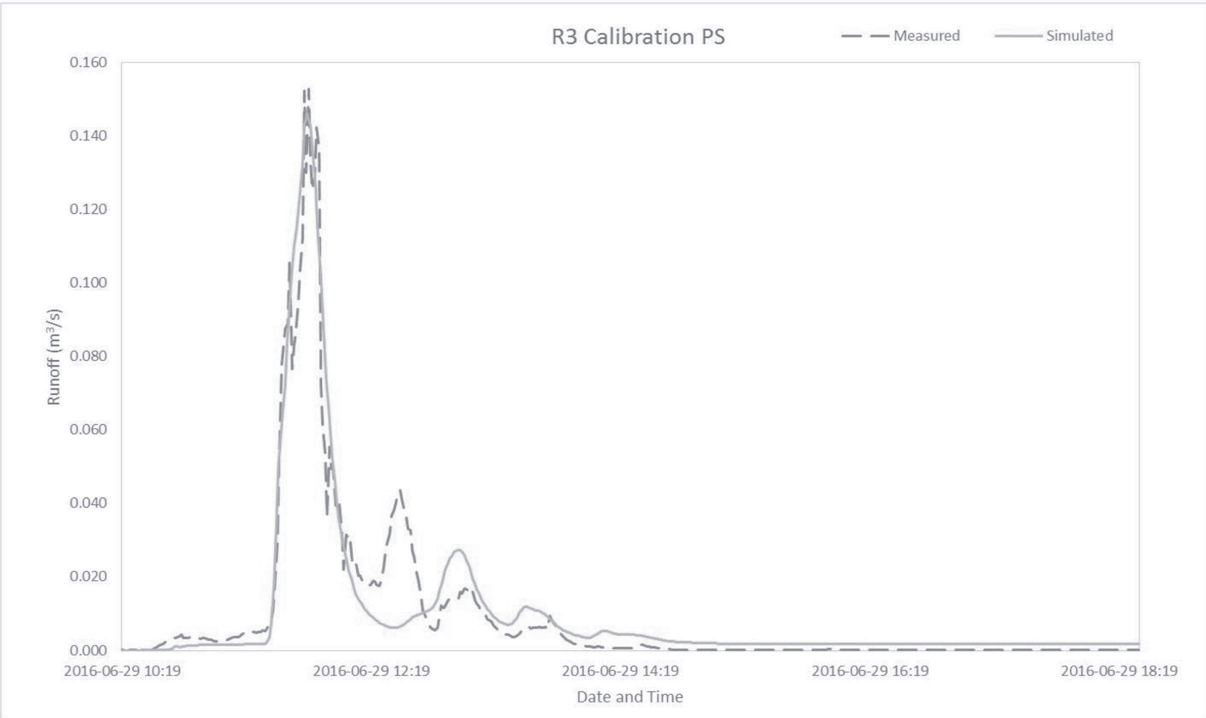


Figure 6.4 Verification of calibration using R3 for NS (top) and PS (bottom).

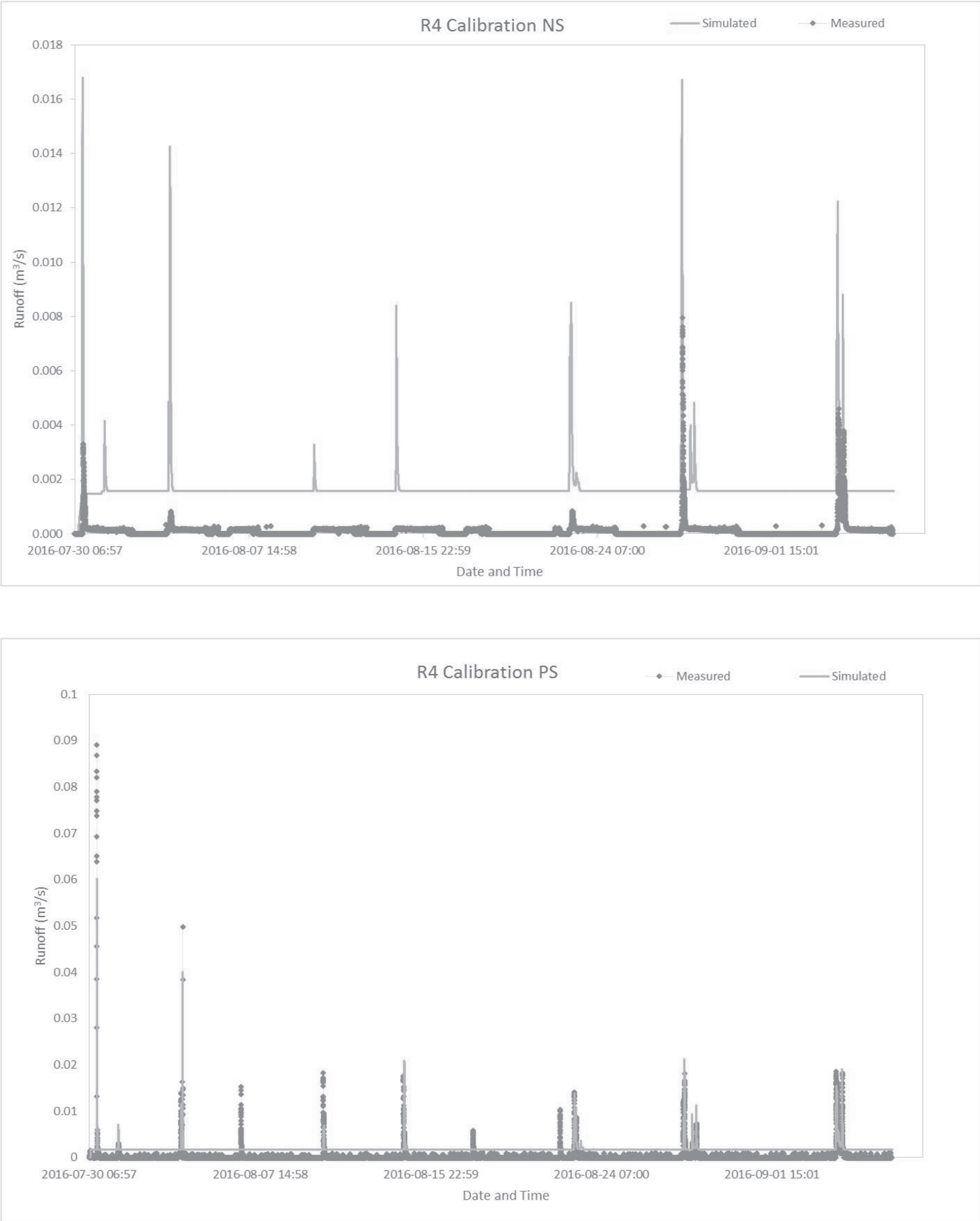


Figure 6.5 Verification of calibration using R4 for NS (top) and PS (bottom).

Table 6.3 Accuracy of calibration and validation.

		NS	PS
R1 Figure 6.2	$\mathbf{R^2}$	0.727	0.751
	Volume error (%)	84.9	33.29
	Peak error (%)	-6.31	-37.16
R2 Figure 6.3	$\mathbf{R^2}$	0.804	0.845
	Volume error (%)	25.41	17.96
	Peak error (%)	1.07	-22.47
R3 Figure 6.4	$\mathbf{R^2}$	0.783	0.9
	Volume error (%)	5.8	19.16
	Peak error (%)	2.75	-5.01
R4 Figure 6.5	$\mathbf{R^2}$	0.175	0.556
	Volume error (%)	1500.21	731.33
	Peak error (%)	110.72	-32.55

As seen from Table 6.3, Figure 6.3 and Figure 6.4, the model performs well, meaning that the peaks match well, even for R2 and R3, which are the heavier rainfall events.

The performance of the model is observed to be quite poor for the discontinuous rainfall event R4. The peak errors are rather high for R4, especially in the NS. This could be because the individual rainfall events within R4 are quite small and therefore more prone to errors- in both the simulation and the measurements.

A numerical error in the model became evident from the simulation of R4. As seen clearly in Figure 6.5, the outflow does not return to zero after the start of the simulation. Instead, the minimum discharge rate becomes constant at a value of about 0.0015 m^3/s. This gives rise to a serious volume error for the R4 simulation, since the time series is so long.

The same minimum flowrate occurs for all the other simulations as well, but is less evident in the other cases since they have larger peak flows in comparison. The source of the error was unable to be determined during this project. Initially it was suspected that the chosen values of Manning number could be the cause. This was disproven by running simulations with different manning numbers. The error was found to be connected with the CRS links. A small error occurred in a number of the CRS links and compounded along the chain until the output to give the observed result. Based on these attempts, it is speculated that the error is of a numerical origin. Further investigation is required to identify and solve the problem.

Since the runoff coefficients are so oversimplified, it is to be expected that the simulated flows will not match the observed flows exactly. It is also seen from Figure 6.2 to Figure 6.4, that the model does not reproduce the smoothness of the observed flowrates from the NS perfectly. Instead, the modelled flowrates from the NS show peaks that mimic the shape of their respective rainfall hyetographs seen in Appendix I.

Despite the oversimplification and the drawbacks mentioned above, a very satisfactory calibration is obtained just from using the computed φ values from Table 6.1. It is interesting that the calibration for the SuDS using this method yields such a good result immediately.

Depending on the final use of the model, it may be possible to use this version of the model directly without any more fine-tuning. However, if a higher degree of accuracy is required, further adjustments can be made to the runoff coefficients based on these results. The time required to achieve a working model using this method therefore is much shorter than if more land-use classification is employed.

Nordlöf (2016) modelled the same area with rainfalls R1 and R3 using a coupled 1D/2D model. The corresponding peak errors were -74% and -71% for the NS and -5% and +19% for the PS. In both cases, the present model appears to perform better. The volume errors reported by Nordlöf cannot be compared with the present results due to differences in the length of the time-series'. Especially in the case of the NS, the present work outperforms the model by (Nordlöf, 2016) and this is of great value as it demonstrates how simple 1D models can be used to model SuDS fairly effectively.

6.4 Limitations and potential sources of error

Some potential sources of error that could have an effect on the accuracy of the model are described below.

Rainfall was assumed to be uniform over the entire area in both the calculations as well as the simulations. It reality, however, rainfall varies spatially.

In the measured data from the site, a number of negative values of flow were recorded from the PS and NS. This could be due to errors of the measurement devices or due to backflow into the system. However, in the calculations and simulations in this project, the negative flow values were disregarded. It is possible that other measurement errors could also be present in the data.

The runoff volumes from each system were calculated from the measured flowrates using the trapezoidal method. The data has a high a resolution of one reading per minute, which justifies its use. However, it could give rise to errors at points where there are sudden and large changes in the flowrate within that resolution.

One possible explanation for the negative values of runoff coefficient in the pervious areas could be due to losses occurring in the form of infiltration, evapotranspiration and surface storage. The negative values could be conveying the information that some of the runoff from the impervious surfaces enters the surrounding pervious areas and is lost or detained before it can enter the drainage channels. Haghighatafshar *et al.* (n.d.) refer to phenomenon as 'Indirect Infiltration'. This phenomenon could be what causes the pervious runoff coefficient to become negative and the impervious surface runoff coefficient to become smaller than the expected values seen in Table 2.1. The total volume lost during each rainfall is seen in Table 5.3.

Since the 1D model cannot simulate overflow from the open channels, in some instances false cross-sections with slightly exaggerated depths or widths are used. This ensures that the model will continue to run without generating errors for minor overflows.

The same constant minimum discharge rate that is reported for R4 occurs in all the other simulations as well. However, it is less distinct for rainfalls R1-R3 since the peak discharge rates are much higher in comparison. In the case of R4, since the series is so long, the effect of the error adds up over time and becomes very severe. This error highlights how minor drawbacks or errors in the model can escalate with a longer time series. Thus, to model longer simulation periods, care must be taken to remove all errors.

A key drawback of the method proposed in this study is that it requires large quantities of measured rainfall and runoff data. To minimize errors, the resolution of the data must be very high. To be able to understand the response of the area to rainfall events of varying strengths properly, corresponding measured data should be available. This could take years to amass. However, in urban areas with reported flooding problems, the investment of time and money into amassing this quantity of data could prove to be worthwhile.

7 Conclusion

Measured rainfall and runoff volumes from the Northern SuDS and Pipe System in Augustenborg, Malmö were used to calculate the runoff coefficients of the pervious and impervious surfaces using the rational method through solving a system of linear equations. When the TIA was used, this calculation resulted in some negative values of runoff coefficient for pervious surfaces, which are not theoretically applicable to runoff calculations. The negative value of runoff coefficient might be an indication that runoff from some impervious surfaces is intercepted by the pervious surfaces and is excluded from the final discharge.

However, when Sutherland DCIA was used for the calculations, the runoff coefficient for the pervious surfaces approached zero. The highest values of φ_{imper} and φ_{per} obtained from the calculations with Sutherland DCIA were 0.54 and 0.06, respectively. It was also observed that aside from DCIA, pervious surfaces also play an important role in runoff generation especially in the Northern SuDS even for milder rainfalls.

Based on the observations made during this project and from the literature review, a new fractionation model was introduced to depict the different types of surfaces in an urban catchment that contribute to runoff. The introduced fractionation model provides deeper insights to different types of surfaces with respect to their hydraulic connectivity to the drainage system. Furthermore, it clear the ambiguity around the different terminologies within the field of urban runoff studies.

The measured data also demonstrated that the outflow from SuDS shows greater dependence on the volume of rainfall compared to the rain intensity. For all three individual rainfall events studied in this project, the Northern SuDS overflows after approximately 10 mm of rainfall. Knowledge of such a response mechanism of SuDS is useful while planning and designing future SuDS for urban storm water management.

A MIKE URBAN model of the study area was calibrated based on the runoff coefficients calculated using Sutherland DCIA. This methodology provided good initial values of runoff coefficient for calibrating the MIKE URBAN model. The most interesting finding from this project was that despite using the total volume of runoff, and a linear equation (rational method) to calculate the runoff coefficients, these values were able to simulate non-uniform rainfall events with dynamic runoff patterns very well. The simplicity of the model is its key strength.

The principles behind this project should be applicable to any region and thus, the project provides a proof of concept and methodology to study the runoff coefficients for various urban settings.

8 Future Scope

The method tested out in this project has only been tried out in the Northern SuDS catchment of Augustenborg. The usefulness of this approach will be better understood if it is attempted on the Southern SuDS as well and on other sites employing mixed drainage systems.

It is necessary as a part of future studies to find the source of and rectify the constant minimum flowrate error that was reported previously. It would also be interesting to study the response of the model with a discontinuous rainfall series in which each individual event is of a higher volume.

Design rainfalls of different intensities and durations can be applied into the model to check if the calibration is still valid.

The theoretical calculations in this project use DCIA to calculate the runoff coefficients. No initial loss and rainfall reduction factors are considered here. However, the model takes an initial loss and a hydrological reduction factor into account. In addition, the model uses the imperviousness fraction as a percentage of the TIA of a catchment. The reduction factors in the manual calculations and in the model appear to be different and yet they correlate well. Analysis of the water balance in the model as well as in the calculations must be carried out to provide a deeper insight into the calculated runoff coefficients.

There is a function available in MIKE URBAN to model low impact development (LID) structures, which can be used to model the hydrologic performance of vegetative swales and green roofs. These have not been used in this current project and it could be beneficial to employ them in a future model.

As mentioned in the discussion, determination of DCIA of a catchment area is a very GIS intensive work. There is ample literature available explaining different techniques to determine DCIA both statistically and using GIS software. This could be an interesting project for the future.

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10 Appendix I: Hydrographs and Hyetographs

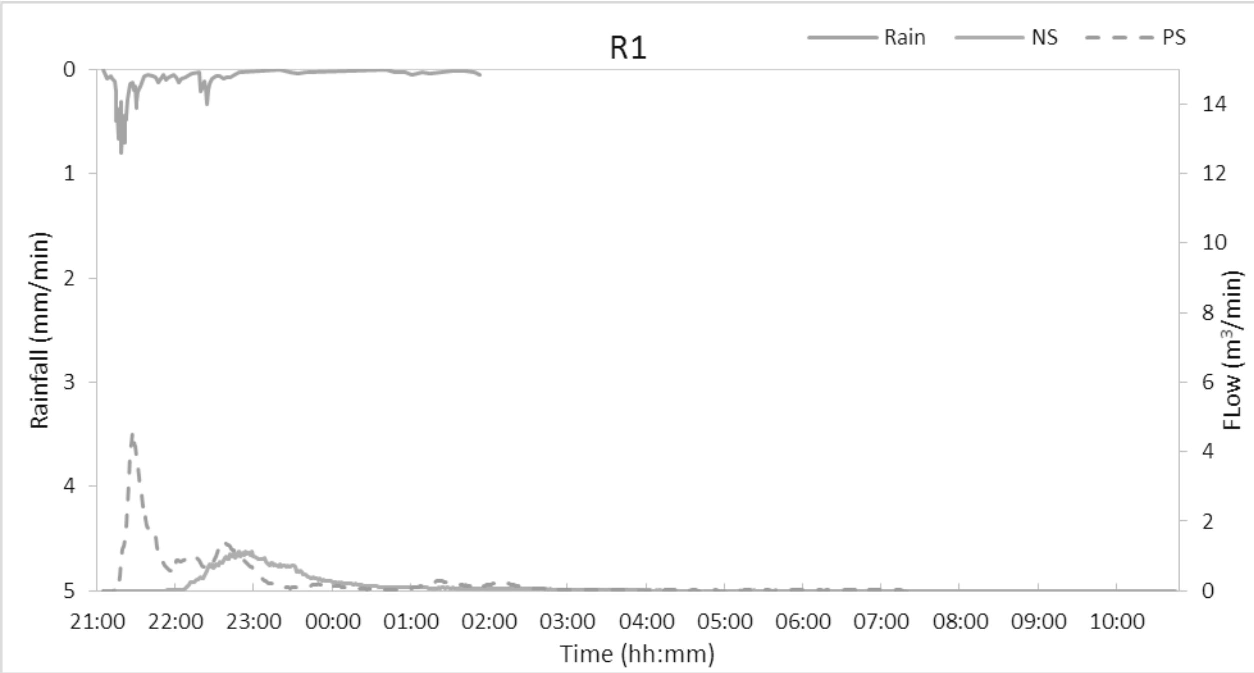

Figure 10.1 R1 Hydrograph and Hyetograph

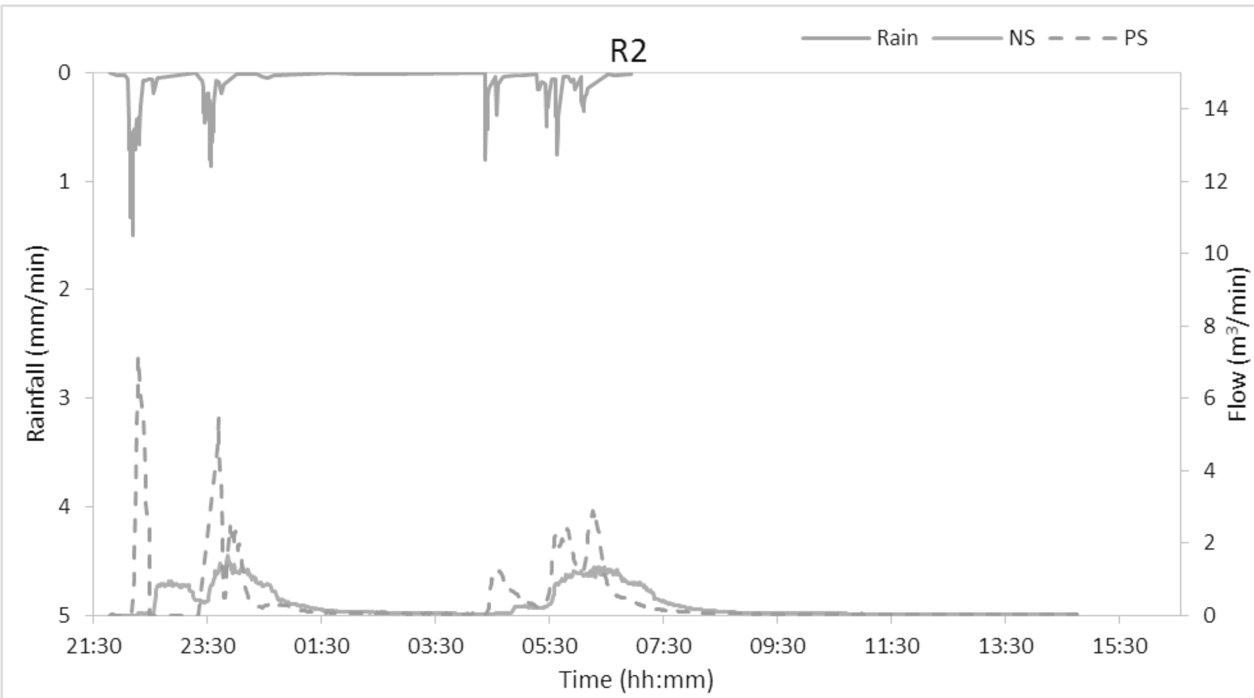

Figure 10.2 R2 Hydrograph and Hyetograph

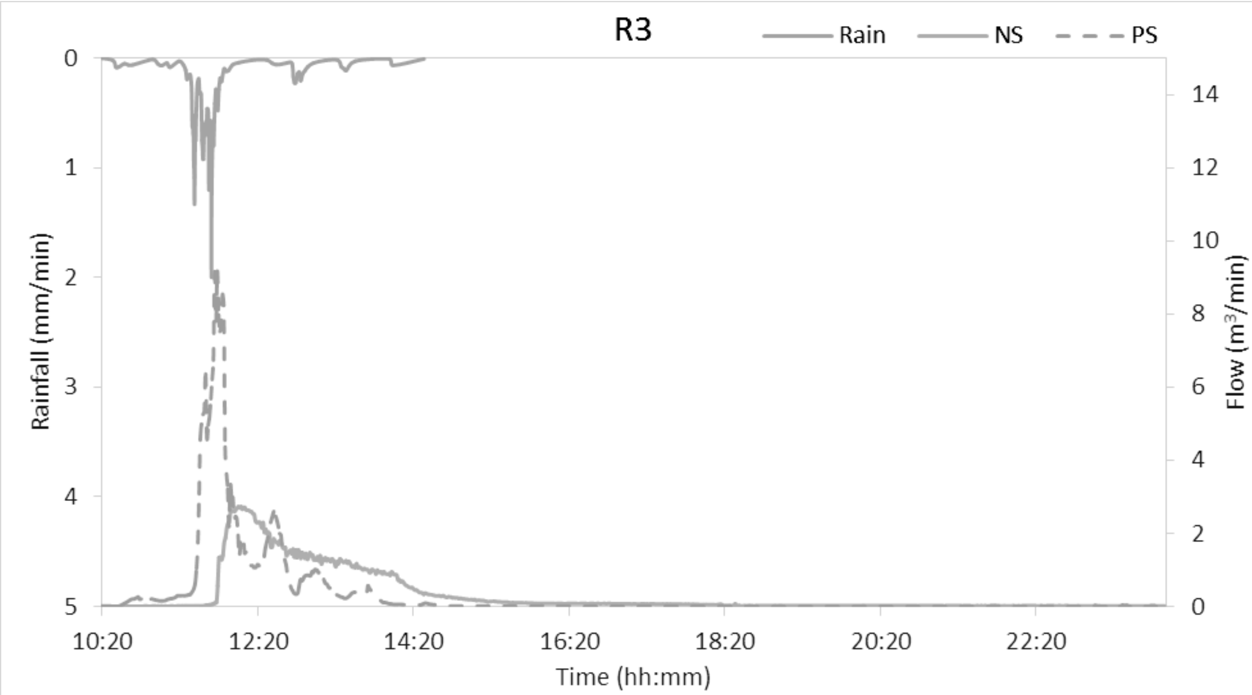

Figure 10.3 R3 Hydrograph and Hyetograph

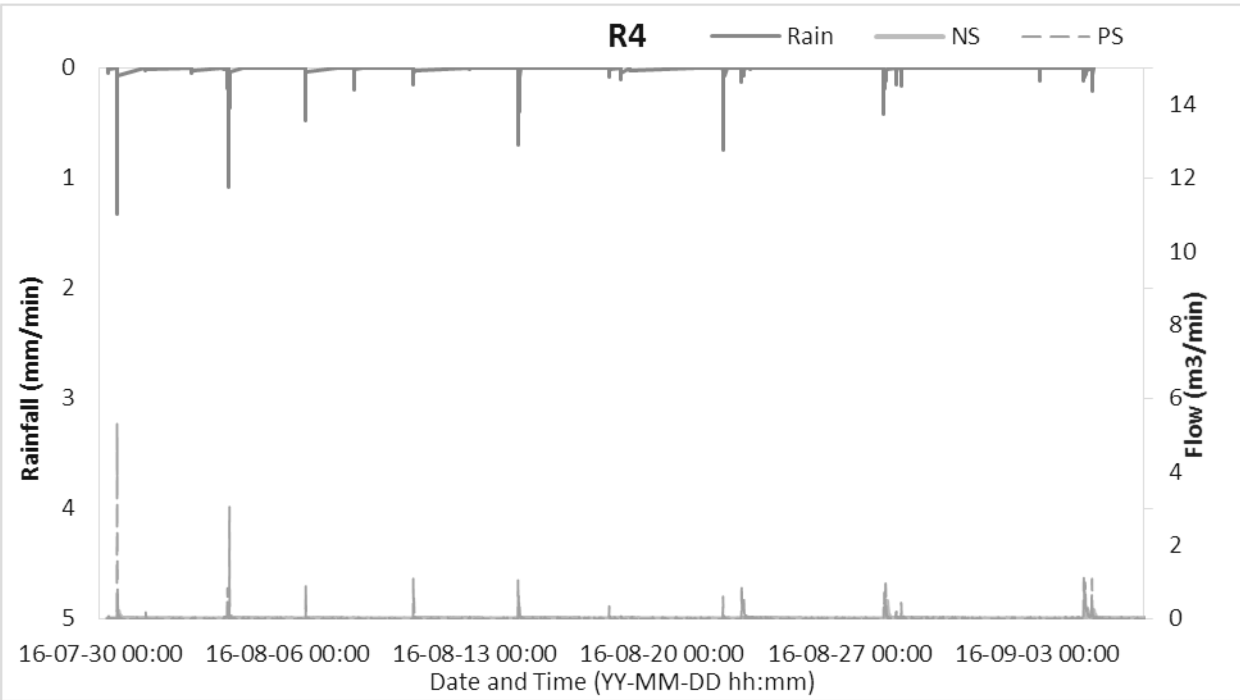

Figure10.4 R4 Hydrograph and Hyetograph

Building flood resistant cities

Popular Science Summary of the report 'A case study of runoff coefficients for urban areas with different drainage systems.'

Christine Thomas

In this report, we compare two methods that cities use to manage rainwater and prevent flooding. Modern city planners are using structures such as ponds and parks to make areas more resistant to floods. Read on to find out how these systems work.

When rain falls on a city, the water from the rainwater is usually transported away from buildings, streets and other surfaces as soon as possible using underground pipes. This is meant to prevent flooding. However, these pipes have a fixed capacity and if the rainfall is very heavy, this capacity will be exceeded and the pipes system will flood.

The rainwater that enters the pipe is usually the water that falls on hard surfaces like roads and roofs, which cannot absorb much water. Grassy surfaces and natural soil, on the other hand, can absorb some water. Thus, these kinds of surfaces are being employed by urban planners nowadays to reduce the amount of water entering the pipes, so that they do not overflow. In addition to using natural surfaces, they also use structures like ponds that can store water. These kinds of rainwater management techniques are present above the surface of the ground and are called sustainable drainage systems (SuDS).

The runoff coefficient is an important parameter, which describes what fraction of the incoming rainwater does not get lost in the soil or atmosphere and instead flows out of the area.

The residential area called Eco-city Augustenborg in Malmö uses SuDS as well as a typical pipe system, making it possible to compare the two systems on one site. Measurement stations at different locations on the site, measured the rainfall and the outflow from the area over a period of one and a half years. In this study, we use this measured data to study the differences between how the pipe system and the SuDS operate, which gives city planners some understanding on how to design better systems in the future. We also use the measured data to calculate the runoff coefficient of the hard surfaces (like roofs and tarred roads) and the permeable surfaces (like parks, yards, sand, etc.) in Augustenborg.

Finally, we used a computer model called MIKE URBAN in this project and calibrated based on the runoff coefficients obtained from the measured data. MIKE URBAN is the software used by many city planners to design drainage systems. In this project, we suggest a very simplified way in which measured data can be used to calibrate the model to a satisfactory level.

In this project, we also explain the relationship between different types of surfaces in an urban area that are important for rainwater runoff studies such as this.