Norwegian University of Life Sciences
Faculty of Environmental Science and Technology
Department of Mathematical Sciences and Technology

Master Thesis 2015
30 credits

Urban Runoff Drainage: Case of Kjelsrud in Oslo

Fida Amin Moheseen

FOREWORD AND ACKNOWLEDGMENTS

This master thesis was written as completion to the master Water and Environmental Technology, at the Norwegian University of Life Sciences under Department of Mathematical Sciences and Technology (IMT).

The topic was selected in co-operation with Water & Wastewater Agency (Vann og –avløpsetaten- VAV) at Oslo Municipality. The thesis focused on estimating the hydrologic response of Kjelsrud with considerations to the urbanisation plan and the impact of the climate change on the rainfall patterns, also suggesting two independent alternatives for the drainage system.

The completion of this thesis was demanding and challenging, but the knowledge in respective area and hydrology models I gained was worthwhile.

I would like to thank my supervisor from the university professor Oddvar Lindholm for his substantial guidance and support.

From VAV I would thank Tharan Fergus for the good co-operation. Thanks to Lars Buhler for his guidance in SWMM, to Andrea Zuur for revising and editing. From Vital Veks As, I would thank Jostien Sundby for his good communication and Roar Finsrud for having access to his data.

Thanks for my husband Ahmad Charafeddine for his support and patience all the way. Also Thanks for my four daughters Selma, Bisan, SaraLea and LillyMina for their support and curiosity about stormwater. Thanks for my family in Bethlehem/ Palestine for their support regardless the distance and for Nora, Are and Dejan for being there always.

ÅS, 17.august 2015

Fida Amin Moheseen

SAMMENDRAG

Mengder av urbane avrenning øker på grunn av både urbanisering og klimaendringer. Den urbane plan for nedslagsfelt av Kjelsrud innebærer å øke tette flater i form av veier og hustak, deretter øker avrenning. Den nedslagsfeltet på 38 hektar vil være i stand til å generere en betydelig mengder av avrenning på 5069 l/s etter gjeldende nedbør og 11406l/s med klimaendring i tillegg. Målet av denne oppgave er å gi VAV to uavhengige alternativer av urbane dreneringssystem for håndtere disse mengder. Den hydrologiske resultater av de to alternativer ble anvendt i forbindelse med systemene reaksjon på utformingen storm.

Det først alternative er et lukket ledning system, hvor det hydrologiske resultater basere seg på en SWMM modell. Den modellen ble brukt for å evaluere dreneringssystemet sin reposene på både nåværende nedbørsmønsteret og den forventet økning som skyldes på klimaendringer. Hvor modellen viser høy risiko for flom og oppstuving da avrenning overstiger den dimensjonerende kapasitet av systemet.

Det andre alternative var overflate drenering med åpne kanaler basert system, hvor LOD klimatilpassing tiltak (grønntak og fordrøyningsdammer) var introduserte til nedslagsfeltet. Dersom disse tiltakene har flere funksjonaliteter, også evner til å håndtere avrenning lokalt. De manuelle beregninger for disse strukturene gitt et bilde om hydrologiske ytelsen av disse strukturene for tilpasse også med økning i nedbør.

Den økonomiske kostnaden for begge alternativer var betydelig, men man kan undersøke nærmere på en kombinasjon av de to alternativer. Hvor det andre alternative kan integreres i det første alternative for å støtte rør systemet og redusere risiko for flom samtidig.

ABSTRACT

The volume of the urban runoff is subjected to increase due to urbanization and climate change. The urbanisation plan for the catchment of Kjelsrud implies increasing the impervious surfaces in forms of roads and rooftops, in return this increases the generated runoff. The catchment area of 38 ha will be able to generate a considerable runoff volume of about 5069 l/s under current rainfall and 11406 l/s with climate change consideration.

The goal of this thesis is t o provide VAV with two independent alternatives for the design of urban drainage system. The hydrologic performance of the two alternatives was used in relation to the systems response to the design storm.

The first alternative is a pipe-based alternative; a SWMM model was used to evaluate the drainage system responses to both the current rainfall patterns and the expected increase due to climate change. The model shows high risk for flooding and manholes surcharge as the runoff exceeded the pipes designed capacity.

The second alternative was the surface drainage open channel –based system, where stormwater mitigation structures (green-roof and detention ponds) were introduced to the catchment for their multi-functionality and ability to handle the runoff locally. The manual calculations for these structures provided with estimations about the hydrologic performance of these structures to adapt also with increase in the rainfall.

The economic cost for suggested alternatives was considerable, but one can investigate further a combination of the two alternatives. Where the second alternative can be integrated into the first alternative to support the pipes and reduce the risk for flooding.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF ABBREVIATIONS

AR5	The Fifth Assessment Report
BGF	Blue-Green Factor
C_f	Climate Change Factor
CSO	Combined sewer overflows
DANVA	Danish Water and Wastewater Association
EEA	European Environmental Agency
IDF	Intensity Duration Frequency
IPCC	Intergovernmental Panel on Climate Change
LID	Low Impact Development
LOD	Local stormwater Disposal (in Norwegian. Lokal overvannsdisponering)
NPRA	Norwegian Public Road Administration (in Norwegian Staten vegvesen)
NWA	Norwegian Water Association
NOK	Norwegian Kroner
OECD	The organisation for Economic Co-operative and Development
PP	Poly Propylene
PVC	Poly Vinyl Chloride
SSB	Statistical Central Bureau (in Norwegian Statistisksentalbyrå)
SWMM	Storm Water Management Model
SWO	Stormwater Outfall
VAV	Municipality Water & Wastewater Agency (in Norwegian Vann og –avløpsetaten)
WTP(s)	Wastewater Treatment Plants(s)

1. INTRODUCTION

Runoff volume is normally larger in urban areas compared to natural and green areas during comparable rainfall events. The increased area of impervious surfaces (rooftops and roads) in urban areas decreases infiltration and increases surface runoff, consequently increasing the risk of flooding. Historically, urban runoff was collected and routed into a conventional system together with wastewater (i.e. combined system). During heavy rain events, the capacity of combined systems can be exceeded, causing manhole surcharge, flooding, and overloading the capacity of wastewater treatment plants that can result in the release of untreated wastewater into recipient. The most common alternative is to establish an independent drainage system for stormwater. Stormwater mitigation measures that reduce urban runoff and increase infiltration can decrease the total volume of water reaching a stormwater system as well as improving water quality.

The impacts from urbanization and climate change pose significant challenges for the Oslo Municipality Water & Wastewater Agency (VAV). Increasing population growth and subsequent construction of new urban areas is significantly increasing the area of impervious surfaces. In addition, climate change is influencing precipitation patterns (both snow and rainfall) in urban areas. VAV is therefore requiring urban development plans to include stormwater mitigation measures that are intended to protect water resources and reduce the risk of flooding.

1.1 RESEARCH GOAL:

The goal of this thesis is to provide VAV with two independent alternatives for the design of urban drainage systems in the new urban area in Kjelsrud. Both alternatives will address the catchment response to a design storm and the expected future challenges of urbanization and climate change. Alternative 1 includes a pipe-based drainage system and Alternative 2 includes surface drainage – based system with includes stormwater mitigations.

1.2 METHODOLOGY

The hydrological modelling program Storm Water Management Model (SWMM), developed by the U.S. Environmental Protection Agency, was used to simulate the runoff generated from current and future precipitation patterns to estimate the hydrological capacity of the two proposed alternatives and the efficiency and performance of the proposed stormwater mitigation measures. Excel was used to calculate the catchment response to the design storm using the Rational formula, and to generate graphs and tables.

1.3 RESEARCH QUESTIONS

The primary research questions in this thesis include the following:

- ❖ Which of the two proposed alternatives is more sensitive to both climate change and urbanisation?
- ❖ Which mitigation structure can be used for flood prevention and increasing infiltration?
- ❖ Which of the two alternatives is more cost effective?
- ❖ What would be a third alternative?

1.4 THESIS STRUCTURE

This thesis is divided into two parts. Part I discusses theory including an extensive literature review of the most relevant issues related to urban drainage. Part II includes simulation results from SWMM for both of the two proposed alternatives, results from the Rational formula and discussion.

Part1

THE LITERATURE REVIEW

2. BACKGROUND

2.1 URBAN DEVELOPMENT - POPULATION TRENDS IN OSLO

Population growth in Norway is expected to increase significantly, reaching approximately 6 million by 2029, 7 million by 2063 and 7.9 by 2100. By 2040, the population of Oslo municipality is expected to increase by 40 % to 832 000 (SSB, 2012). Population growth is a significant challenge for Oslo Municipality as well as communities in the Oslo-Akershus metropolitan region, forcing municipal governments to consider alternative urban planning strategies. The Plan and Building Department at Oslo Municipality (Plan - og bygningsetaten) has started implementing an urban development plan to meet expected growth, including building 70 000 new buildings between the year 2013-2035 (approximately 4000 houses/per year). Still, this might be not enough if current rates of population growth (about 1.97 persons/ house) continues, the planned number of houses might be increased to 114 000 houses by 2030. However, these numbers can be modified depends on the actual demand in the near future (OsloSpeilet, 2013).

2.1.1 THE EXPECTED ADVERSE IMPACT OF URBAN DEVELOPMENT IN OSLO

Urban planning is necessary for Oslo to address projected population growth. Planned development will affect land-use patterns in the municipality, increasing impervious surfaces, reducing the total green area and thus causing a decrease in infiltration and an increase in surface runoff. *Figure 2.1* shows the number of residential areas in Oslo Municipality, where most of dens residential areas are concentrated in the city centre.

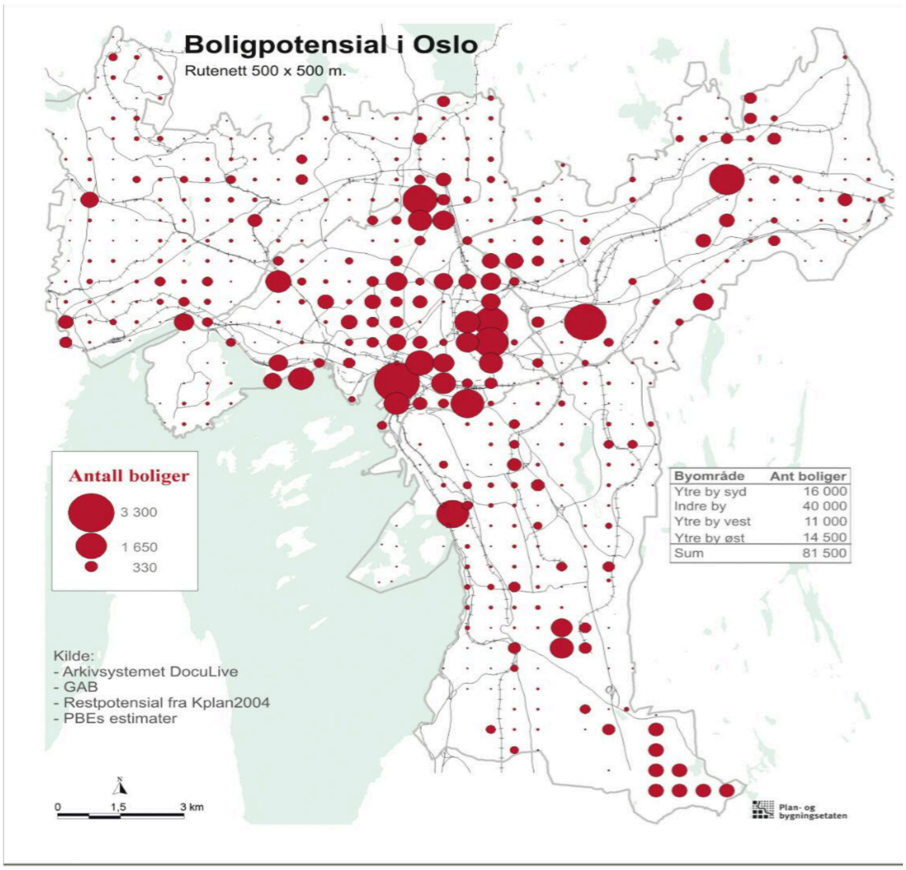


Figure 2.1: Map over number of residential areas in Oslo (OsloMunicipality, 2008)

2.2 CLIMATE CHANGE

With regard to climate change, this thesis will focus on changing in precipitation patterns, i.e. changing in the rainfall patterns. Other climate change related variables, such as temperature, sea level, rate of snow melting wont be discussed further.

2.2.1 GLOBAL PERSPECTIVE

The recent Fifth Assessment Report (AR5) released by the IPCC stated that human activities are the main cause of climate change with a probability of 95%, causing the average global temperature to increase and changing the distribution of moisture content in the atmosphere, consequently changing global precipitation patterns.

According to the same report, there are several indications that wet regions will become wetter, and dry regions will become drier. Increasing temperatures will result in an increase in the transport of moisture from dry regions to wet regions[1], subsequently causing changes in current precipitation patterns and influencing the precipitation distribution within and between regions. Extreme precipitation events are predicted to increase with 5-10% C^o, this depends on time scale, season and location.

This can be explained also based on the fact that the capacity of the atmosphere to hold water vapour increases per degree C^o of temperatures, the higher temperature the more vapour is produced, also the more precipitation will be produced. *Figure 2.2* shows the projected changes on temperature and precipitation between 2080-2099 and 2081-2100.

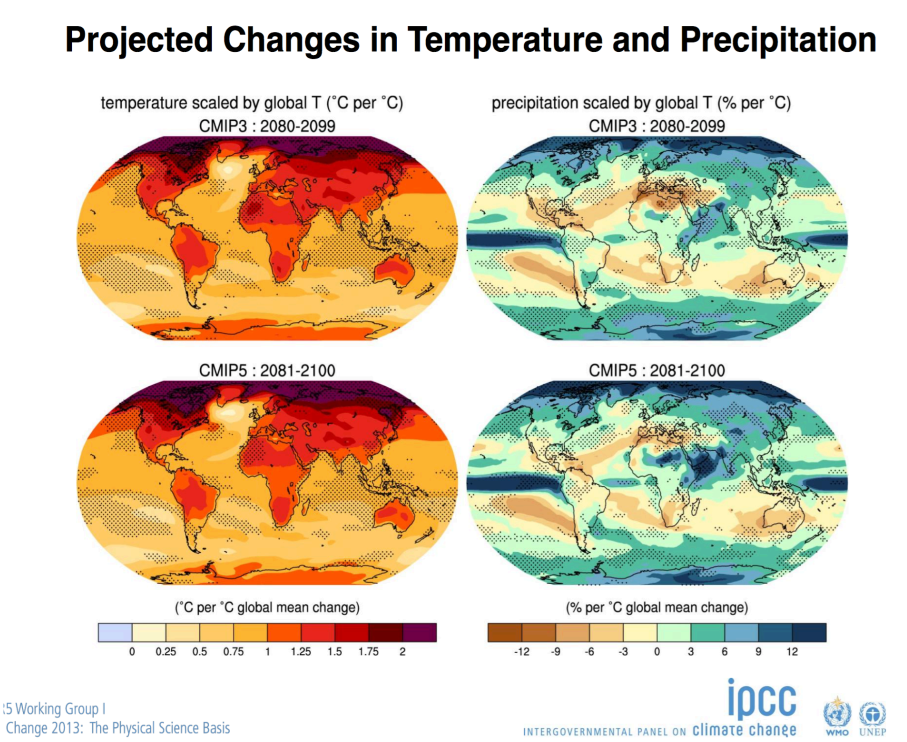


Figure 2.2: Projected changes on temperature and precipitation (IPCC, 2015)

---

[1] *AR5, chapter 7, section 7.6.2 the effect of Global Warming on Large Scale Precipitation Trends IPCC. 2013b. Fifith Assesment Report (AR5) [Online]. Working group I- Climate Change 2013: The*

2.2.2 LOCAL PERSPECTIVE IN NORWAY

In general, the climate in Norway varies considerably over both time and seasonality due to changes in air and ocean currents, topography and latitude. These factors in turn influence the current local variations as follows (NOU, 2009)

<u>Temperature variation</u>: The average annual temperature in Norway is about +1 ^{o}C, with a maximum of +6 ^{o}C at the coastline in the Southern and Western part of the country, and a minimum of – 4 ^{o}C at higher elevations. The warmest days (days with temperature more than 20 ^{o}C) are concentrated mostly in the eastern part of the country and along the Oslo fjord.

<u>Rainfall variation:</u> The average annual rainfall in Norway is 1486 mm. Troms, in the northern part of the country, and Gudbrandsdalen, located in the central part of Norway are considered the driest regions in Norway, receiving less than 300 mm/year). The west coast of Norway between Hardanger fjord and Møre receives greater than 5000 mm/year, making it one of the wettest regions in Europe.

Figure 2.3 is summarises the geographical climate variations in Norway, where average annual summer temperatures (^{o}C), and precipitation (mm) vary with altitude (m), based on data collected between 1960-1990 from 1683 weather stations in 441 municipalities (*source. http://www.biomedcentral.com*).

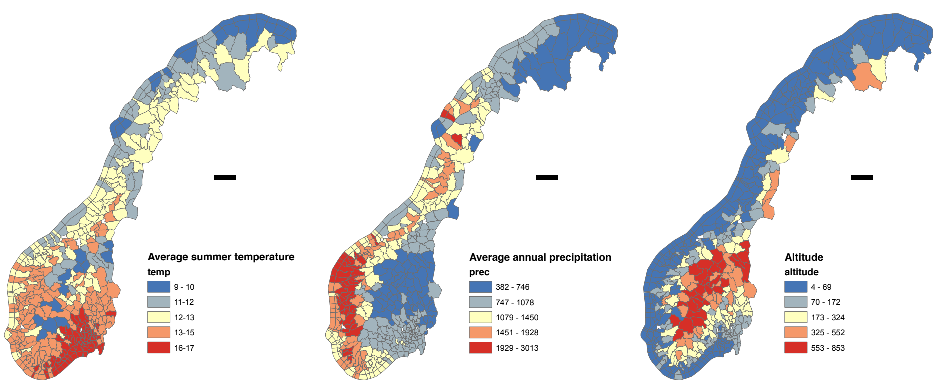


Figure 2.3: The climatic variations in Norway between 1960-1990

The Norwegian topography and the mountainous terrain increase uncertainties in climate change modelling, making it difficult to identify the actual impacts of the climate change. Nevertheless, research assume that temperature in Norway will increase between 1- 2.5 ^{o}C during the period 2030 -2049, compared to 1980 -1999. This change is predicted to be obvious in the inland and the north part of the country. Precipitation is expected to increase, especially during the autumn and in the western region of the country, and during the winter in the southern region (O´Brien et al., 2006).

Primary observed changes and trends in Norway as an impact of the climate change are summarised below (OECD, 2013):

Temperature: The annual average temperature has increased by about 0.8 ^{o}C during the past 100 years, resulting in increased winter and spring stream flow, earlier snowmelt and spring and autumn flooding. Heavy rainfall events have become more frequent since 1987.

Precipitation: The annual precipitation has increased by greater than 20% since 1900, with most of the increase in the precipitation occurring after 1980.

Predicted changes in temperature and precipitation resulting from climate change are summarised below (OECD, 2013):

Temperature: The annual mean temperature is estimated to increase by 3.4 ^{o}C with northern regions experiencing an increase of 5.4 ^{o}C.

Precipitation: Average annual precipitation is predicted to increase by 5%, 18 % and 31% by the year 2100 for low, medium and high climate projections respectively. According to the medium projection the annual precipitation will increase nation wide by about 20% in the autumn, winter and spring and by 10 % in the summer. Consequently, annual runoff is also expected to increase especially in the autumn and winter, and decrease during the summer with the exception of glacial runoff. Flood projections are uncertain due to the large climate variations within the country. Despite modelling uncertainties, increases in temperature and changes in rainfall patterns are predicted to increase flood events, especially in early spring, late autumn and winter.

2.3 THE URBAN DRAINAGE SYSTEM

The urban drainage system was first challenged due to the interactions between human activities and the natural water cycle, where this cycle was interrupted due to either (a) abstraction of water for drinking purposes and generating a wastewater also (b) increasing the impervious surfaces that causing rainwater diversion from natural drainage system and generating a considerable runoff. Consequently, both types of water are requesting immediate drainage (Bulter and Davies, 2011). In this thesis, the rainfall-generated runoff is only of concern and the urban generated wastewater won't be discussed further.

The runoff is a rainwater (can be also resulted from other forms of precipitation), which fallen on impermeable surfaces and caused distinguished damages, flooding and also further health risks due to the pollutants from air or the catchment itself ((Bulter and Davies, 2011),p.28)

The impacts of urbanisation and climate change on the urban drainage are discussed further in the following sections, in terms of quantity and to certain extend runoff quality also.

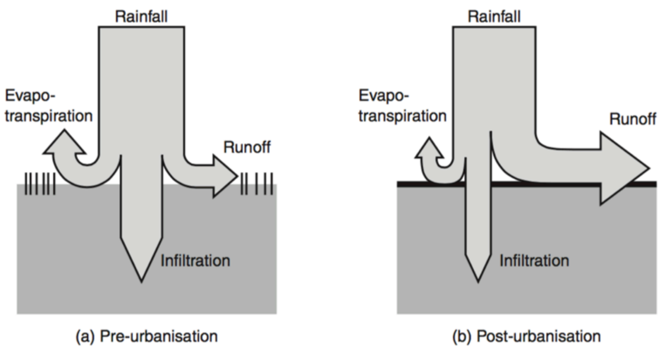


Figure 2.4: The impact of urbanisation on the rainwater- generated runoff (Bulter and Davies, 2011)

2.3.1 THE IMPACT OF URBANISATION ON THE DRAINAGE

There are significant differences between the natural water cycle and the urban water cycle; this is in terms of water losses and water - generated outcomes. Where in the natural water cycle there are a number of connected processes including evaporation, condensation, precipitation and formation of ground water (see *Figure 2.5*)[2]. However, the natural water cycle is included into the urban water cycle, but it is interrupted due to less natural infiltration and less groundwater formation due to increase of the impermeable surfaces (see *Figure 2.6*).

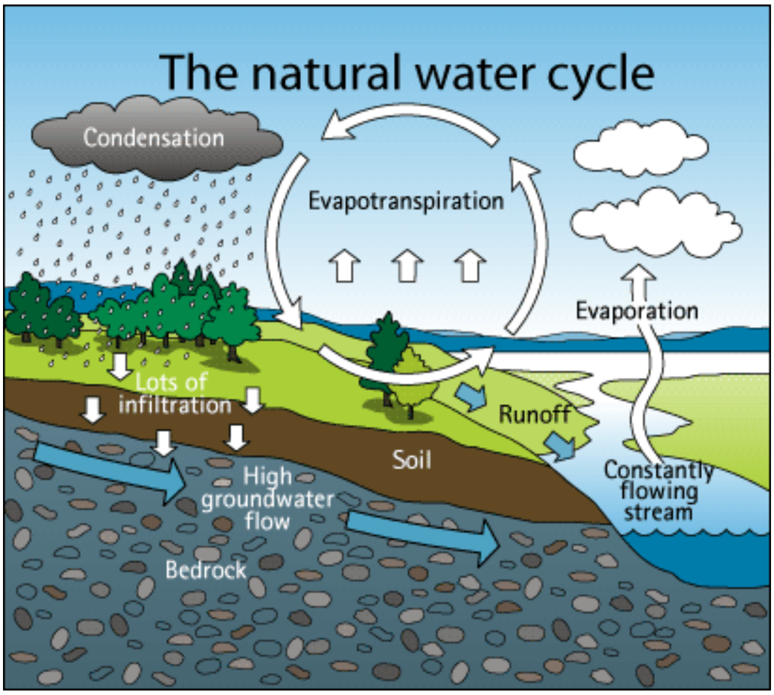


Figure 2.5: the Natural water cycle

[2] *Figures 2.5&2.6 were adopted from http://www.blueplanet.nsw.edu.au*

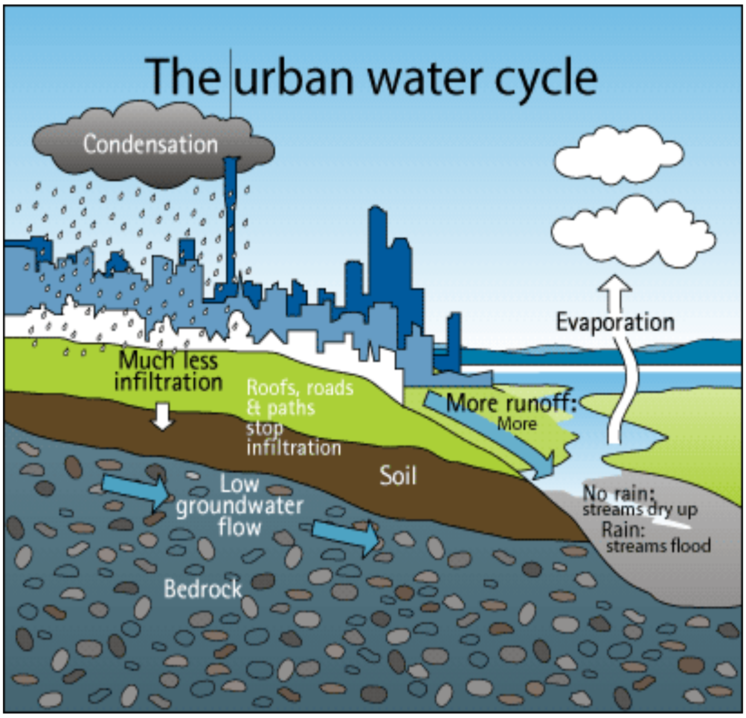


Figure 2.6: The urban water cycle

The main difference between the natural and urban water cycles is the degree of natural infiltration, where in both cycles the rainwater is subjected to losses due to evaporation to the air and transpiration by the plants. But in the natural areas, the surplus of the rainwater infiltrates in to form or/and recharge the groundwater. Still, a proportion of runoff can be formed (overland flow), but it is relatively less than the infiltration and depends on the surface permeability and soil type dominated, also is changeable under the storm event. In the urban areas, the same cycle is taking place, but the permeability degree of the surfaces influencing the ration between groundwater proportion and runoff proportion. Where the hard and impervious surfaces increases the surface runoff proportion comparing to the groundwater one. Further, the hard and impervious surfaces influencing the runoff to move faster on these surfaces than the natural surfaces causing flooding and surcharge on the sewers and recipient (Bulter and Davies, 2011).

Further, both of the groundwater and surface runoff end up in the recipient such as river and both contribute to the river but in different way. While the groundwater contributes to the base -flow of the river, the surface runoff increases the flow under the storm event and thereafter increases the volume in the river (Bulter and Davies, 2011).

In short, the urban runoff is a sensitive parameter, in terms of the amount and movement. Where as soon as the urban runoff with high speed and considerable amount entering either the sewers or the recipient will cause a significant increase in the peak flow in both systems causing eventually flooding (see *Figure 2.7*), also the instant runoff can carry pollutants and sediments to both sewers and river (Bulter and Davies, 2011).

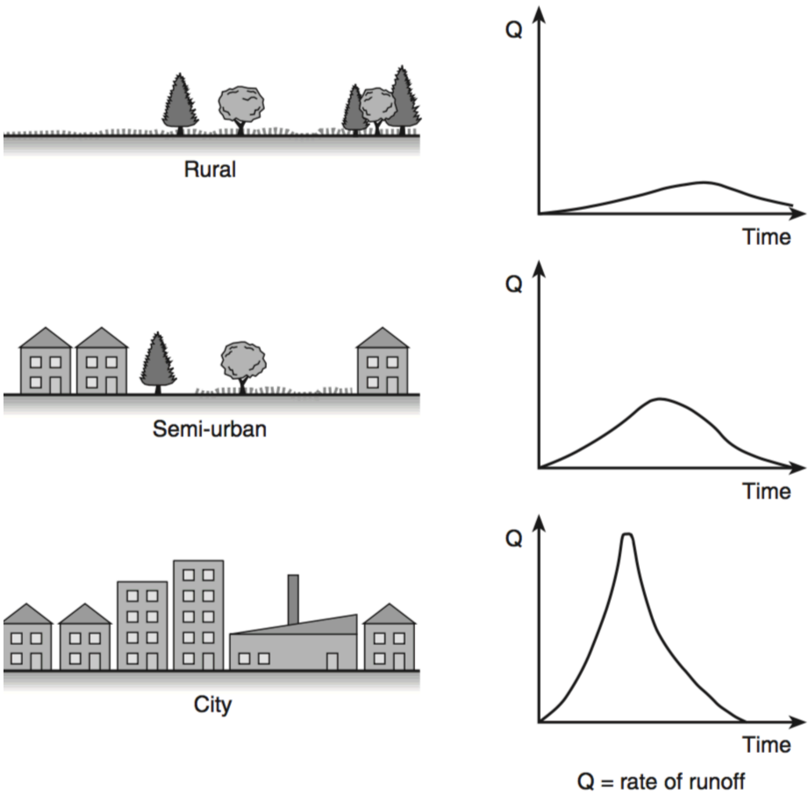


Figure 2.7: The impact of the urbanization on the peak flow of the runoff (Bulter and Davies, 2011)

2.3.2 THE IMPACT OF THE CLIMATE CHANGE ON THE DRAINAGE

Following the climate change studies, there are a number of potential implications of the climate change to the urban drainage. Hence, this mainly related to the expected increase in the total precipitation in terms of intensity and duration, which in return increases the generated runoff in the urban areas. Still, the impact of the increased temperature is difficult to assess.

The climate change implications are summarised with the followings (Bulter and Davies, 2011), p. 102):

- Increased volume and flow – rate that may exceed the capacity of existing sewer systems, which leading to more frequent surcharging, surface flooding and property damages.
- Greater deterioration of sewer due to more frequent surcharging
- Greater build-up and mobilisation of surfaces pollutants in summer
- More frequent CSO spills
- Poorer water quality in rivers due to extra SWO & CSO spills and reduced base flows in summer
- Increased flows of dilute wastewater at the WTP(s) due to higher rainfall and infiltration, potentially leading to poorer treatment by biological process.

2.4 THE URBAN RUNOFF

Following rainfall event, the amounts of generated runoff are of high concern when planning new urban areas, this in term of both quantity and quality. Hence, in this thesis there will be focus only on the quantity of the runoff.

2.4.1 THE STORMWATER RUNOFF GENERATION

The urban runoff generation can be described in different ways. However, *Figure 2.8* explains the different processes that lead to the surface runoff formation, where stormwater (A) runs over the impermeable surfaces and form the surface runoff (B) also the overland flow ((C) the surplus from infiltration) join together to the surface runoff and flow into the sewers (D). These different processes are mainly depends on the rainfall intensity and duration as well as on the nature of the catchment, nevertheless the nature of the surfaces (Bulter and Davies, 2011).

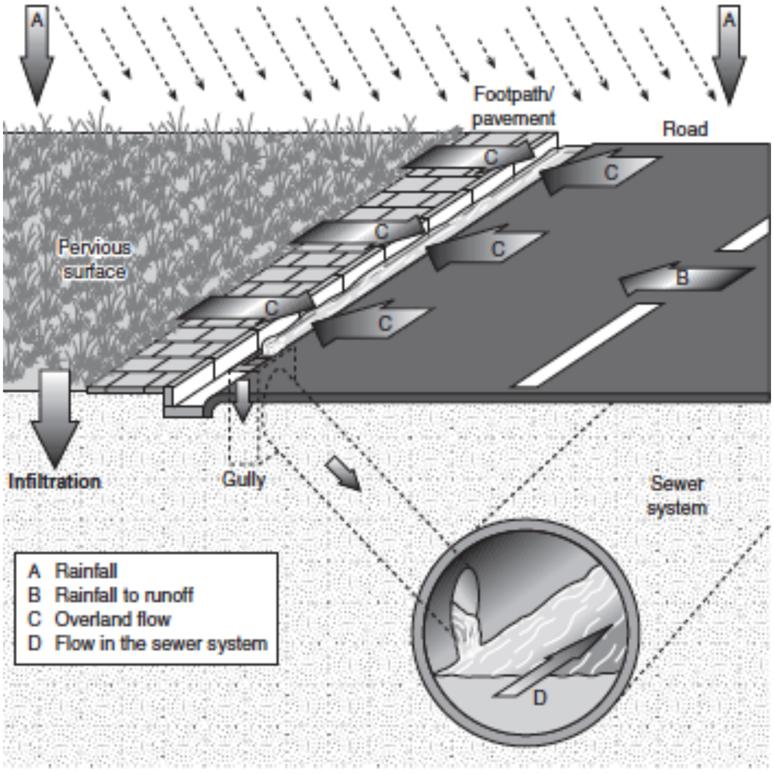


Figure 2.8: Runoff generation processes (Bulter and Davies, 2011)

2.4.2 THE URBAN RUNOFF CONDITIONS

In Norway the urban runoff is characterised according to the season, but the runoff generated form snow melting is not easy to assess yet. The different runoff conditions producing a considerable amounts of runoff that are varies due to the season, this can be explained as follows ((Ødegaard, 2012), p.62-63):

- Summer conditions: heavy rainfall on dry and impermeable surfaces, there is no runoff generated from semi- preamble and permeable surfaces
- Autumn conditions: long duration rainfall events on wet surfaces, this causing runoff from permeable surfaces also increasing the groundwater table.
- Winter conditions: rainfall events on frozen surfaces that are covered with snow, where runoff is generated from impermeable, semi-permeable and permeable surfaces.
- Spring conditions: snow melting and runoff form all types of surfaces and increase in groundwater table.

2.5 THE SEWER SYSTEM- COMBINED OR SEPARATE

In general, the traditional transport system of all forms of urban generated water (wastewater, stormwater, and overland flow) was mainly the combined pipe- based system. Where these forms of water were flowed into the same pipe system and mixed together to end up in WTP and thereafter into a water recipient. In addition, the groundwater might also infiltrate into the pipes and mixed up with the all other forms of urban water, but this water of a good quality and act as a diluent. Still, each form of water contributed in different proportion to the WTP, this depends mainly on the water consumption and precipitation patterns. But the overall load at the WTP is considered (Bulter and Davies, 2011).

Further, the separate pipe-based system is relatively new and used for transporting all forms of urban generated water. This system is based on two separate pipe systems, one aimed to transport stormwater and one for wastewater, where the stormwater can be easily bypass the WTP without any special treatment and transported directly into the water recipient, while the wastewater transported to the WTP. Still, this system also can be exposed to groundwater infiltration. Therefore, considering the pipes capacity is also significant in the separate system (Bulter and Davies, 2011). In addition, the stormwater sewer system must be designed after the perspective return period that can varies between 5-50 years (Ødegaard, 2012).

Further, one of the main disadvantages of the combined system is overloading the pipe system with stormwater under heavy rainfall events, consequently causing basement flooding.

Figure 2.9 illustrates the impact of combined and separate sewer systems on houses, where a) illustrate a basement flooding or inundation under a rainfall event while b) illustrate the water accumulation outside the basement when separate system is installed (Lindholm, 2013).

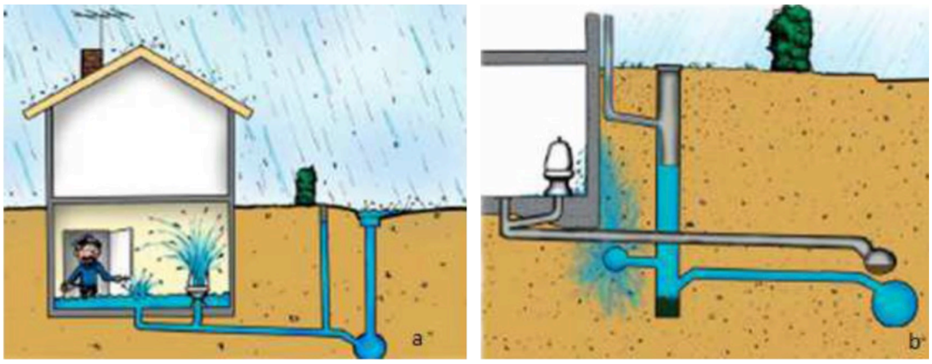


Figure 2.9: Illustration of impact of combined and separate sewer system (Lindholm, 2013)

Still, the separate system must be adjusted to avoid water accumulation outside the basement. One can install the stormwater pipe system about 90 cm under the basement level, this measure is recommended also to avoid potential basement flooding (Lindholm, 2008).

Table 2.1 summarised the advantages and disadvantages comparing the separate and combined systems ((Bulter and Davies, 2011), p. 24):

Table 2.1: Comparison between combined and separate sewer systems

System	Combined	Separate
Advantages	• lower pipe construction cost • economical in space • cheaper and simpler house drainage • limited treatment of stormwater • sediments exposed to flush out under storm	• no sewer flow • less pollution in the water recipient • enough with small- scale treatment • storm water can be pumped • optimum line can be valid • flow is maintained and less variation in flow – no overloading • no girt to be removed • flooding limited to stormwater only
Disadvantages	• sewer flow is valid and adjusted to the WTP size	• two pipe systems, is more expensive • additional space needed in urban

	• sewer flow causing serious pollution in the recipient • necessary large-scale wastewater treatment • high pumping cost • slow and shallow flow in large sewers, due to dry weather, causing sediments • wide variation in flow to pumps • must remove road grit • high risk for flooding and surcharge • high risk for basement inundation • compromised line for the different forms of water	areas • double house drains might increase risk of leakage • sediments can not be flushed out

2.6 STORMWATER MANAGEMENT

As mentioned above, the urban development is producing more runoff comparing to natural areas, which in return is leaving a further negative impact on the sewer system in these areas. Hence, more runoff is generated under heavy rainfall event and overloads the system-designed capacity, thereafter encouraging surface flooding and more frequent manholes surcharge.

Figure 2.10 explains the designed- sewer capacity as a constant but the runoff flow is a variable depending on both of the rainfall intensity and duration.

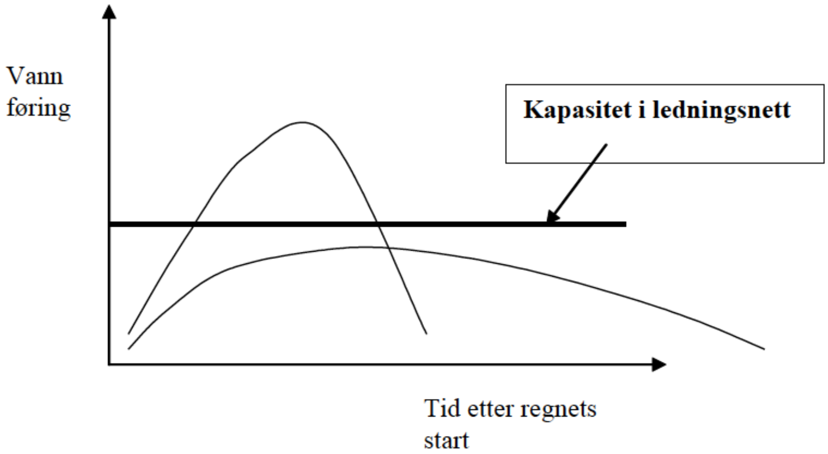


Figure 2.10: The designed – sewer capacity under heavy rainfall event (Lindholm, 2012b)

The 3- steps strategy is considered in the planning phase of the intended urban development in the catchment. Thereafter, aiming to minimise the impact of the urban runoff locally and to enrich the area profile with sustainable runoff mitigations.

This strategy implies the following stages for different rainfall events that vary from small – medium to extreme or heavy events. Hence, the suggested values should be adjusted to the design-storm for the catchment (Lindholm, 2012a):

Step 1: To locally retain and infiltrate the runoff from small rainfall events (exa. < 20 mm)

Step2: To locally delay and retained the runoff from medium rainfall events (exa. >20 and < 40 mm).

Step 3: To transport the runoff from extreme rainfall events (exa. >40 mm) far from the urban areas. Implement flood pathways.

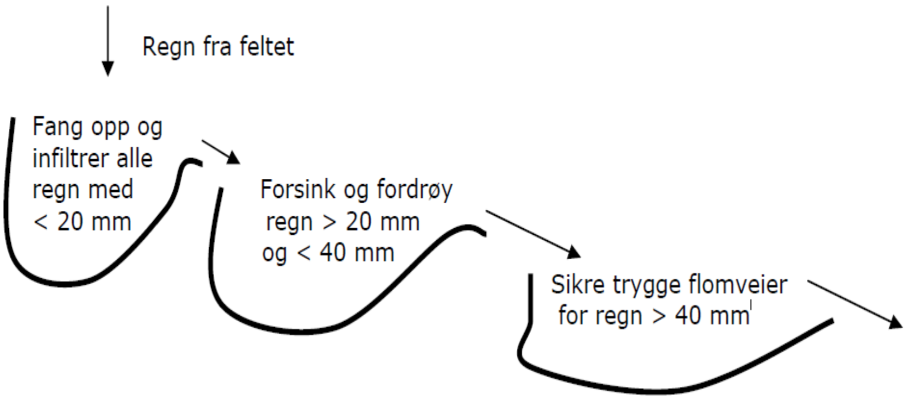


Figure 2.11: The 3-steps stormwater strategy (Lindholm, 2012a)

Further, the amount of runoff that needed to be detained is illustrated in *Figure 2.12 (COWI, 2013)*. Based on the Figure 2.12, this amount can be defined as the difference between the maximum runoff from the catchment and the allowed outflow to the recipient. This amount must be delayed or detained to avoid runoff undesirable consequences.

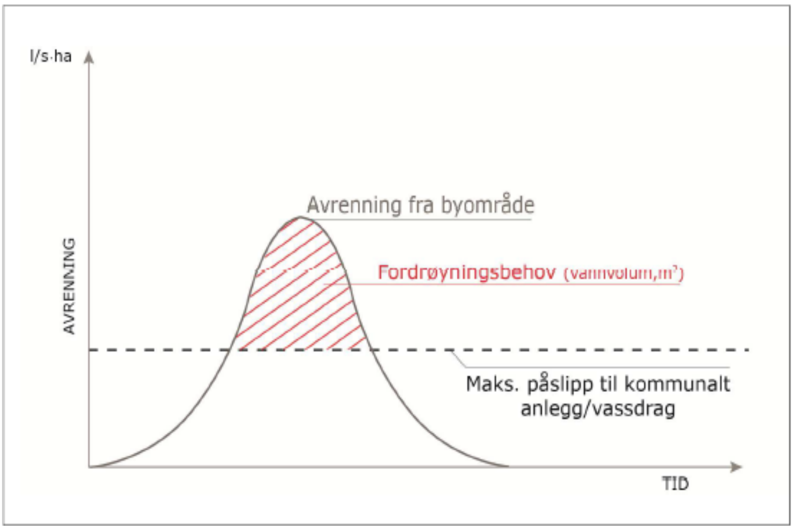


Figure 2.12: The amount of runoff needed detention (COWI, 2013)

In principal, most of the urban drainage infrastructures are based on the capacity to match a design discharge, such as 1- in 100- year is highly recommended to avoid flood consequences later on. In addition, one can consider good manhole that increase the hydraulic capacity for the system with about 15% in case of surcharge. Also, upgrading the pipe (conduit) diameter improving the hydraulic capacity of the drainage system (Lindholm, 2012a).

Table 2.2 explains the impact of upgrading the pipes diameter on the sewer capacity. Where by upgrading the 300-diameter pipe by only a 100 mm this will increase the capacity for tis pipe by about 114% in comparison to the original capacity.

Table 2.2: Impact of the diameter upgrading on the pipe hydraulic capacity (Lindholm, 2012a)

From diameter (mm)	To diameter (mm)	Capacity increase %
300	400	114
400	500	80
500	600	61
600	700	50
700	800	42

2.6.1 THE LOCAL STORMWATER MANAGEMENT

In general, the local stormwater management is a sustainable and environmental friendly alternative to the pipe-based drainage system, in which the urban drainage system is connected again to the natural water cycle, also with least possible impact. This alternative is important as it is considered a multifunctional and promising structure to meet the future climate change and the urbanisation challenges.

According to COWI recommendations, the general aims from the local stormwater management are summarised with the followings (COWI, 2013).

- To reduce flooding at the recipient

- To reduce pollutant transport to the recipient
- To minimise the load in the existed sewer system.
- To reduce runoff transport to the WTP in case of the combined system

Further, in order to meet the mentioned above aims, the followings should be considered first when planning stormwater management structures to adhere with Oslo Municipality stormwater management strategy for 2013-2030 (OsloMunicipality, 2013):

Adapting with the climate change:

The implemented stormwater management measures should minimise the damages caused by the climate change without having an impact on the human, buildings, infrastructure & environment. Also minimising the risk for flooding in the urban areas.

Securing a good water quality in the water resources:

The stormwater management measures should improve the water quality of the runoff, also reducing the pollution in the water resources

Utilizing the stormwater as a resources:

The stormwater should be utilized as an integrated part of the urban area landscape, also encouraging the recreational purposes as well as the biological diversity within the urban areas

The Oslo municipality is recommendations related to integration of stormwater management into the urbanisation plan (OsloMunicipality, 2013):

- Consider stormwater management measures at early stage of the plan
- Evaluate different stormwater measures aiming to implement multifunctional solutions
- Include the multifunctional stormwater management in the municipality own specification

2.6.2 THE STORMWATER MITIGATION STRUCTURES

The stormwater management structures are significant in urban areas, hence in terms of runoff mitigation. These structures are able to reduce the negative impact of the stormwater on the sewer system and maintaining the pipes capacity under different climate conditions, i.e. reducing risk for flooding and surcharges. In addition, improving the water quality of the runoff before reaching the recipient.

These structures can varies according to their function, but also they can be used in different combinations to establish a pipe-free drainage system that is based only on these structures.

Still, the selections of the appropriate and most feasible combination is influenced by the dominated land use, site and the catchment characteristics, nevertheless the expected performance. *Table 2.3*includes the most common structures, also providing with several examples and their purposes (Bulter and Davies, 2011).

Table 2.13: List of most common stormwater management structures (Bulter and Davies, 2011)

Structure	Example	Purpose
Inflow control	Rooftop ponding Green-roofs Storage connected to downpipes	Retain or delay runoff Improve runoff water quality
Infiltration and detention	Infiltration devices: - Infiltration trenches - Soak -away (stones/plastic filed boxes)	Diverting the runoff for later infiltration or evaporation
	Vegetated surface: - Grass- lined channels (Swales) -Filter stripes	Transporting, storage, infiltration and treatment of runoff. Also used for pre-treatment Delay runoff peak and reduce runoff volume
	Pervious pavements: - Porous or permeable surfaces. Exa. Porous asphalt	Encourage runoff filtration, sedimentation, adsorption, and chemical/biological treatment, also storage

	Infiltration basins	High capacity infiltration, used for small catchment
	Constructed/artificial wetlands	Improve runoff quality Reducing runoff flow
Storage structures	Detentions basins	High storage capacity
	Detention ponds	Storage and treatment, recreational (sailing, fishing) and environmental values. Flood control function also as a Reservoir

Figure 2.13 illustrating what is called the stormwater mitigation -train where each structure contributes to reduce the impact of the stormwater on the sewers and recipient ((Bulter and Davies, 2011), p. 531).

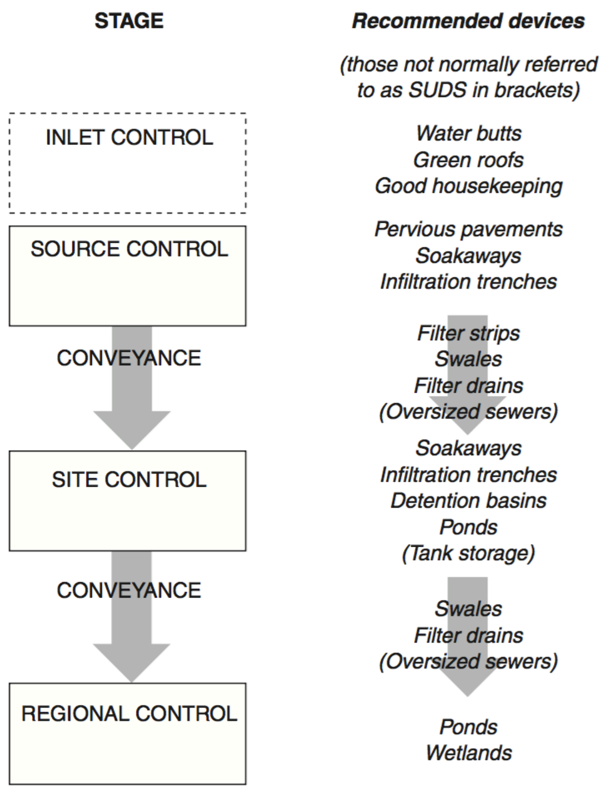


Figure 2.14: Illustration Stormwater mitigation/management train (Bulter and Davies, 2011)

2.6.3 STORMWATER MANAGEMENT LAWS AND REGULATIONS

Since there is no law directly regulating the water and sewer sector in Norway, the implementation of stormwater management practices is regulated under different laws. This includes the planning and building law, water resources law and pollution law. Hence, there are different departments deals with the implementation of these practices (Langeland, 2011).

The followings listed regulations related to stormwater management are considered at VAV at Oslo Municipality ((Brennhovd, 2014), p.11):

Table 2.4 Stormwater management related regulations at *VAV (Brennhovd, 2014)*

Law	Reference document
Plan – and building law § 27-2	City ecological program
Building technical regulation (TEK10) chap.15 /sec.3	Guidelines about stormwater management for developer/builders
Pollution law	Action Plan for environment and climate 2013-2016
Pollution regulation chapter 15A	
Water resources law § 7	Main plan for wastewater and aquatic environment 2000-2015
Subscription terms worked at Oslo Municipality	

2.7 DETERMINATIONS OF GENERATED RUNOFF – PEAK SURFACE RUNOFF

Regardless the nature of the catchment area, there are several methods used to determine the amounts of the generated runoff, or what is called the peak of the surface runoff. This is important to be determined in advance; therefore the drainage system actual design should meet the expected runoff.

2.7.1 CLIMATE CHANGE FACTOR (C_F)

The climate change factor (C_f) is a dimensionless additional value, usually used to represent the expected future changes in precipitation extremes. The C_f -value depends on geographical location, the extreme precipitation both in terms of duration and frequency (Willems et al., 2012).

Still, using the current IDF-curves to extrapolate the future rainfall extremes required consideration to the climate factor. Following to the Swedish and Danish reports, the future precipitation patterns are subjected to increase of about 20-50% more than the current patterns. Thereafter, other measures must be considered to meet the future climate challenges when planning /designing water related new technical projects and also for renewal and rehabilitation of old systems. Hence, the C_f -value is influencing the lifetime for water related technical facilities, therefore the DANVA recommended to use this value as a reference to explain the impact of the climate change, I addition considering a long-term plan up to 100 years. ***Table 2.5*** explains if the precipitation intensity will change over time interval, one can estimate the lifetime of the technical facility (NorskVann, 2012).

Table 2.5: Example of increase of precipitation intensity over time (NorskVann, 2012)

Time interval	Increase in precipitation intensity
Today	0%
In 10 years time	5%
In 25 years	12.5%
In 50 years	25%
In 100 years	50%

Still, the DANVA recommended to consider a climate factor between 1.2-1.4 (see ***Table 2.6***) adjusted to the return period for the designed rainfall (NorskVann, 2012).

Table 2.6: Danish water and wastewater association Cf -value recommendations for 2008 (NorskVann, 2012)

Return period	2years	10 years	100 years
Climate factor (C_f)	1.2	1.3	1.4

In order to meet the changes in the future precipitation pattern, the NWA recommended considering an increase of about 30-50% of today's precipitation patterns when designing water related facilities (NorskVann, 2012).

Following the NPRA recommendations, each technical installation with expected lifetime of 100 years a C_f –value must be equal to 1.3 for 10 years return period and 1.4 for 100 years return period. However, the municipalities in Norway are granted the freedom to adopt their own strategy to meet the climate changes. Thereafter, municipalities decided independently different C_f –value for their perspective projects (StatenVegvesen, 2011). The municipality of Oslo considering 1.5 is an ideal C_f –value of, hence considering this value when designing all new technical installations and also for renewal and rehabilitation of the old one (Engan, 2014).

2.7.2 RUNOFF COEFFICIENT: Φ- VALUE

The runoff coefficient (φ value) is a dimensionless empirical- constant value that represents the percentage of the rainfall that becomes runoff ((Rossman, 2010), p. 30). It assumed to varies according to time and rainfall intensity.

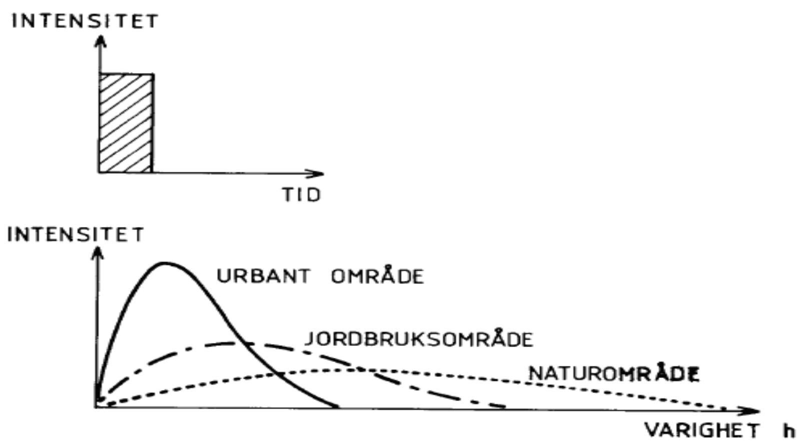


Figure 2.15: Runoff from different surfaces (Lindholm, 2014)

The φ-value varies and depends on permeability of the surfaces (see *Figure 2.14*), where the areas with low infiltration capacity (impervious surfaces, urban areas, steeped gradient) compromise high φ-value comparing permeable surfaces (forest, cultivated land, flat surfaces, pervious surfaces). In another word, impermeable surfaces produce more runoff than the permeable one (Lindholm, 2014).

The high φ-value means low infiltration capacity of the surfaces and increase the risk for urban flash/surface flooding[3]. The municipality of Oslo comply with NWA and considering the following φ-value for runoff calculations. ***Table 2.7*** listing the different φ-value according to the nature of the surface (NorskVann, 2012). The pervious surfaces are assumed to have φ – value equals to ZERO (Rossman, 2010).

Table 2.7: Runoff coefficient φ-value (NorskVann, 2012)

Type of surface	Runoff coefficient
Impervious surfaces (rooftop, concrete t/asphalt, mountain	0.85-0.95
Urban centre- dens inhabited areas	0.7-0.9
Apartment/townhouse	0,6-0,8
Detached/family houses area	0.5-0.7
Gravel/ unpaved road	0.5-0.8
Lawns, cultivated land, parks, cemeteries	0.3-0.5
Industrial areas	0.3-0.9
For flat area and permeable surfaces low values are considered	

However, the NPRA is taking into account the seasonality in addition to the permeability, when considering the φ-values. ***Table 2.8*** Includes φ-value for frost-free surfaces (StatenVegvesen, 2011).

[3] *Flash flood: defined as a short term event within 6 hours causative event(heavy rain, dam break, snowmelt and ice jams) and often is taking place within 2 hours at the start of a high intensity rainfall. It can be produced when slow moving or multiple thunderstorms occuer over the same area (nws.noaa.gov)(FLOODING, F. Flash Floods [Online]. National Oceanic and atmospheric adminstration Available: www.nws.noaa.gov [Accessed 10.04 2015].)*

Table 2.8: The φ-values for frost free surfaces (StatenVegvesen, 2011)

Type of surface	Runoff coefficient
Asphalt, concrete or mount like surface	0.6-0.9
Gravel roads	0.3-0.7
Lawns and cultivated land	0.2-0.4
Forest	0.2-90.5

Still, the high φ- value must be considered when the design aimed for urban areas to meet extreme rainfall situations, this in term of intensity and duration. This will enable the system to reduce the risk for surface/flash flooding, as the runoff will move much faster on paved surfaces.

In addition to φ- value considerations, Norsk Vann recommended few other issues to be considered to avoid flash flooding (NorskVann, 2012):

- In the summer: runoff from dry land after an intensive rain
- In the autumn: runoff after a prolonged rain and high ground water level after a intensive rain
- In the winter: runoff on frozen overland with rain in the autumn or winter

2.7.3 TIME OF CONCENTRATION:

Time of concentration or what also called Travel- time is defined as the time between the occurrences of rainfall event until excess water leaves the catchment at the very most downstream outlet (Laurenson, 1964).

$$T_c = t_t + t_s$$

Where:

Tc: concentration time (s)

t_t: surface inflow time or overland flow (s)

t_s: flow time into the pipes (s)

The flow time into the pipes (t_s) depends on the length of drainage pipe and the water velocity.

$$t_s = {}^{L}/_{V}$$

Where:

t_s: the flow time into the pips (s)

L: drainage length until the most downstream point (m)

V: flow velocity into the pipes (m/s)

It is recommended that the flow velocity must be between 1.5-2 m/s (Ødegaard, 2012).

Following Svenskt Vatten guidelines for 2004 (P90) recommended the following flow velocity for different drainage system (see ***Table 2.9***) (NorskVann, 2012):

Table 2.9 Recommended surface Flow Velocity (NorskVann, 2012)

Drainage type	Flow velocity (m/s)
Sewer system	1.5
Tunnel and mega size	1.0
Trenches and drains	0.5
Fields	0.1

The surface inflow time (***t_t***) occurs following a rainfall event and defined as the travel time from the outmost point of the catchment to the nearest outlet. Hence, it is proportional with the distance and decreases under a high rain intensity, also depends on the overland surface conditions. It can be ranged between 5-7minutes in urban areas with limited size; these values can be projected to range between 3-15 min (NorskVann, 2012).

In order to obtain appropriate and more accurate values for surface inflow time, one can use a Nomograme diagrams. *Figure 2.15* explains the surface inflow as a function of flow distance (BergenMunicipality, 2005).

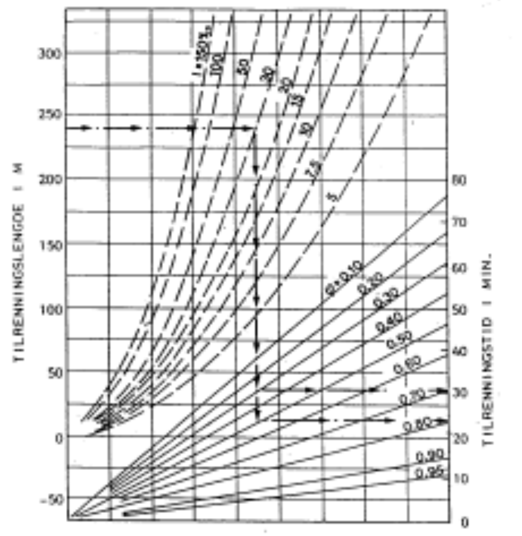


Figure 2.16: Nomograme for surface inflow time (t_t) (BergenMunicipality, 2005)

The NWA is adopting values for the surface inflow time (t_t) from the Northern Virginia BMP handbook (1992). Where t_t is specified according to the location.

Table 2.10: Time of concentration in min for urban areas (NorskVann, 2012)

Zone	Consecration time in min
Town centre	About 5
Residential area with multi- man homes	5-10
Residential area with villas and gardens	10-12

Further, the time of concentration is significant to avoid flood events. In case the storm duration is equal or larger than the time of concentration for that specific catchment, then a flood event is expected to take place. But if the storm duration is less than the time of concentration, so the storm can end and the surface flow rate decreases before approaching the most down stream outlet. The ideal scenario is when the rainfall rate is relatively constant; a surface runoff flow will be generated soon after the catchment storage capacity had been met. In another word, equilibrium between the storm duration and concentration time is established and enable all parts of the catchment to contribute simultaneously to the outlet (KristiansandMunicipality, 2014).

2.7.4 RETURN PERIOD –REOCCURRENCE OF FLOOD:

In order to determine the rainfall return period or what is called the reoccurrence of flood event, the Oslo municipality is following the Norwegian national standards (NS EN 752-4, 1998)[4].

The return period is an important parameter to be considered when designing stormwater drainage system, it is significant to adjust the hydraulic capacity of the drainage system, just to be sufficient enough to avoid flooding and manholes surcharges. Based on this, the municipalities can develop their own guidelines (VA-norm) according to their need also can adjust their own return period for designed rainfall events (NorskVann, 2012) .

Table 2.11: Requirement of minimum return period in years (NorskVann, 2012)

Design rain [5] (1 in ¨n¨ years)	Type of area	Design occurrence of flooding [6] (1 in ¨n¨ years)
1 in 5 years	Low damage potential areas (Rural areas)	1 in 10 years
1 in 10 years	Residential /industrial areas	1 in 20 years
1 in 20 years	City center /	1 in 30 years

[4] *The Norwegian national standard for drain and sewer-systems outside buildings-hydraulic design and environmental considerations (NS EN 752-4, 1998).*

[5] *No surface ponding/surcharge is allowed above the top of the pips*

[6] *No surface ponding/surcharge is allowed at the basement level, 90 cm requirement above the sewer top*

	industrial/ commercial areas	
1 in 30 years	Very high damage potential areas	1 in 50 years

In general, the municipalities in Norway are using ***Table 2.11*** as a reference to design new water related facilities, also when rehabilitating or renovating old ones. The table includes values of the frequency of rainfall reoccurrence where no flooding or surface ponding /surcharge is allowed in the respective area.

2.7.5 TIME-AREA CURVE

The time-area diagram is a graph of a cumulative drainage area contributing to discharge at the watershed outlet within a specified time of travel ((Muzik, 1996) ,p. 1401).

The GIS is mainly used to estimate potential flow network of the catchment areas, also it is used to calculate both the distances and the runoff traveling times to the outlet for different points within the catchment area (Muzik, 1996). GIS is also used to create an Isochronal or travel-time maps, where accumulative runoff contributions from each point is highlighted.

Figure 2.16 explains IsoChronal or Travel-time map of a Catchment area with an accumulative drainage (runoff) from A to B (Kitterød, 2013).

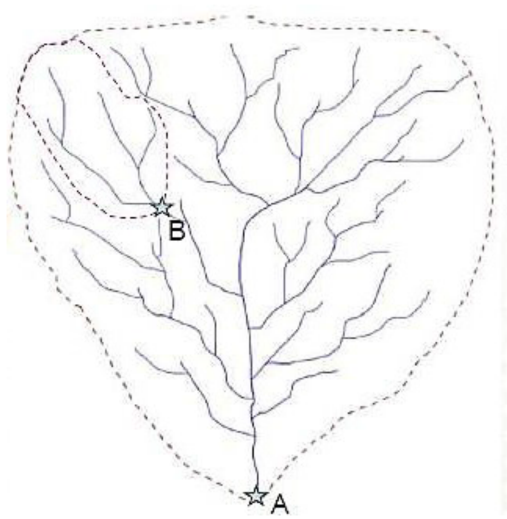


Figure 2.17: IsoChronal or Travel-time map of a Catchment area with an accumulative drainage (runoff) from A to B (Kitterød, 2013)

The time-area diagram can be obtained by computing the time for various points on the catchment area to draw isochrones (line of equal travel), or time contours on the catchment area map. Then the area between adjacent isochrones can be calculated and plotted against travel time in the form of Histogram (Laurenson, 1964).

Still, the runoff generated from each respective point within the catchment area is mainly dependent on rainfall conditions, evapotranspiration, size and shape of the catchment area, soil type dominated, percentage of vegetation cover and urbanisation. However, the runoff direction depends on the topography and follows the gravity direction (Bjerkholt, 2012).

The main characteristics for the runoff from small size catchment area (Bjerkholt, 2012):

- Quick reaction under short and intense rainfall
- The rainfall can be received uniformly and at the same time
- Minimum detention
- Floods are over with a short time period

2.7.6 THE RATIONAL FORMULA METHOD

The peak surface runoff method is widely used to estimate manually the rate of peak surface runoff. It also called time-area method. The method is used when designing drainage systems for runoff flow in small urban areas with an area less than 20-50 hectares (ha). Hence, for areas for more than 50 ha, other hydrology models (f. ex. SWMM & Mouse) can be used (Ødegaard, 2012).

The formula assumed that the drainage area is a single unite receives a rainfall that is distributed evenly over that area. In addition, the flow is estimated only at the very end downstream point, where all parts of the watershed are contributing to the outflow and the rate of overall maximum runoff can be best estimated. Hence the concentration time is approached (www.itc.nl/ilwis/applications).

The rational formula for Peak surface runoff rate is expressed with the following formula (Ødegaard, 2012):

$$\boldsymbol{Q} = \boldsymbol{\varphi} \times \boldsymbol{A} \times \boldsymbol{I} \times \mathbf{Cf}$$

Where

Q: Peak runoff rate, runoff flow (L/s)

φ: Runoff coefficient

A: drainage area (ha). 1ha=10 000 m^2

I: rainfall intensity (L/s.ha)

C_f: climate change factor

The formula is mainly dependent on a pre-defined drainage area, runoff coefficient, and an average of rainfall intensity (obtained from IDF- curves). Climate change factor is significant also, since changing in the future precipitations patterns will influence the projected runoff flow estimations.

The formula follows the following assumptions [7]:

- The peak probability to happen (return period) is equal to the rainfall intensity
- The runoff coefficient φ is constant during the rain storm
- The concentration time is approached

2.8 DETERMINATIONS FOR THE DRAINAGE SYSTEM DESIGN

2.8.1 URBAN RUNOFF: THE SURFACE -DRAINAGE ALTERNATIVE

The Norwegian Water Association (NWA)(in Norwegian –Norsk Vann) is a national association representing the Norway´s water related industry. It is the only authorised institution to act on behalf of member´s municipalities and their companies. Thus the municipality of Oslo comply with NWA water and sewer related norms (VA-norm).

[7] *The assumptions were adopted from (www.itc.nl/ilwis/applications)*

The VA-norm is adhering with the 3-points strategy, in which the urban runoff should not be discharged into the sewer system, but instead it must be handled locally through encouraging stormwater mitigations, and conveyed further through floods pathways in case of extreme storms. In addition, the system must be designed with long lifetime as possible, also with considerations to the cost-effectiveness and population growth demands (NorskVann)

Figure 2.17 illustrate the different situations for urban runoff generation and the recommended mitigation that must be conducted. Where the runoff from small rainfall event should be collected and infiltrated through inflow control structures such as Green-roof. Further, the runoff generated from medium rainfall events should be retained and delayed through Rain-bed and open detention basins for example, however the runoff generated from heavy rainfall events must be transported outside the city centre and flood pathways must be implemented (OsloMunicipality, 2013).

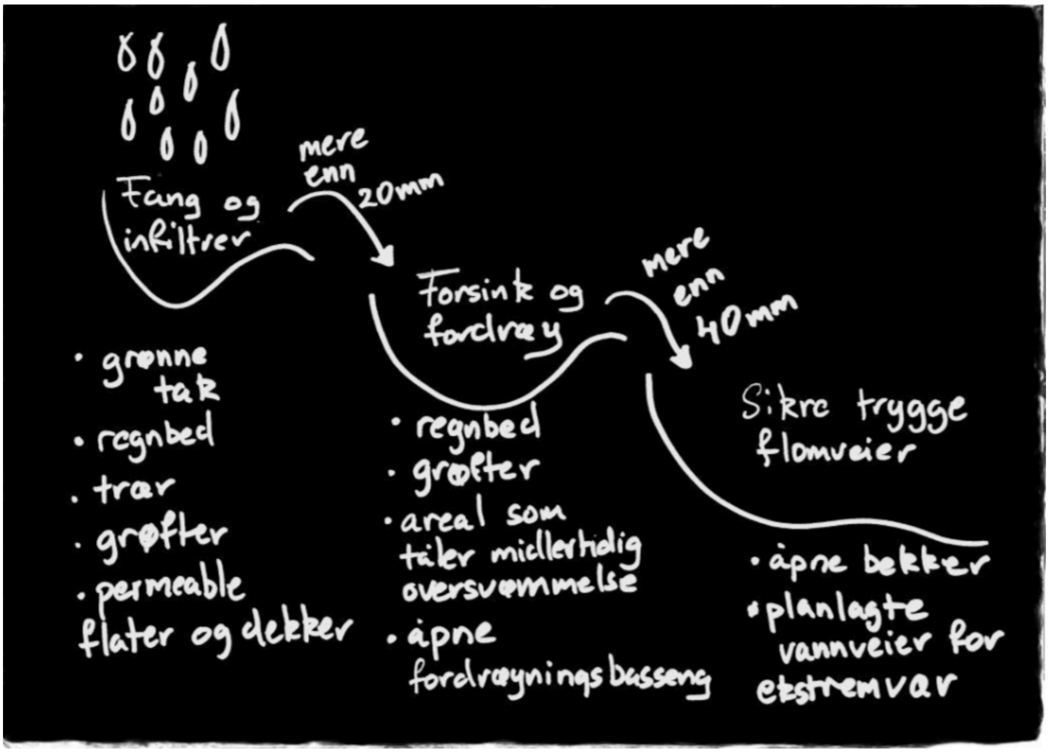


Figure 2.18: Illustration of the urban runoff management (OsloMunicipality, 2013)

2.8.2 URBAN RUNOFF: PIPE-BASED ALTERNATIVE

Following the VA-norm, the design requirements for both stormwater and wastewater are the same. In both cases the minimum pipe´s diameter must be not less than 150 mm, despite the expected flow (Lindholm, 2013). The Norwegian Public Road Administration (NPRA) required 200 mm as the minimum diameter for pipes aimed for stormwater road flushing into conventional system (StatenVegvesen, 2011).

Moreover, it is obligatory to apply the ¨gravity flow ¨ method by following the available natural drainage. This is an important measure to avoid including pumps and pumps stations to the network. Further the VA-Norm recommended water velocity 0.6-0.8 m/s or pipes must be with minimum boundary shear stress with 3-4 N/m^2 along the pips bottom (Ødegaard, 2012). These conditions aimed to ensure that sewers will carry suspended sediments and minimise accumulation of settles deposits due to long residence times or low water velocity during low flow periods, thus, ensuring self-cleaning and full hydraulic capacities of the pipes.

The stormwater pip´s material depends on different factors including the rainfall chemical composition, bedrock, soil type and structure and nature of the terrain surfaces, also the human activities in the catchment area. The stormwater quality can varies due to different factors such pH-value and carbon contain, that affect the water quality and consequently influencing the choice of transporting pips, but plastic pipes (for example PVC &PP) considered one of the best options against pipe´s corrosion than metal (COWI, 2010).

As for the main characteristics for stormwater manholes/nods, the Norsk VA-norm specified the diameter for the downstream manholes should not be less than 1000 mm, where manhole´s inflow pip should be the same material as the stormwater pipe´s material. For example if one decided to use PVC pipe the manholes inflow pip must be in plastic also such in PP (NorskVann). Moreover, the distance between two manholes must not be less than 80 m, regardless the expected flow (Lindholm, 2013).

In order to avoid sealing of the sewer system, the stormwater must go through pre-treatment measures. This can includes mechanical purifying instruments such as screen or sand trap (NorskVann).

2.8.3 SECURING THE SYSTEM AGAINST FROST

In general, the NPRA is taking into consideration the weather conditions in Norway and recommended to establish all water related facilities in frost-free zones. This aimed to reduce damages caused by frozen water. In case of stormwater, the pipes can be placed deep enough to avoid frozen water in the winter, however the actual depth can be decided for each individual situation. Also it is recommended to avoid that water from frost-free zone run into frost vulnerable system (StatenVegvesen, 2011).

Alternatively, one can isolate the pipes against frost. There are wide range of materials can be used and function as isolation. The NPRA is recommending the following materials (StatenVegvesen, 2011):

- Rock fill and rubble
- Sand and gravel materials
- Leigh weight aggregate
- Foamed plastic plates (typically extruded polystyrene XPS)

2.9 TOWARDS THE BLUE –GREEN FACTOR (BGF):

The Blue-Green factor is a significant tool when planning/designing a new urban area. This tool is used to define the urban profile in terms of enriching the outdoors qualities in the urban areas with more water and vegetation (COWI, 2013).

Further, the BGF is based on scoring system from 0-1. Where the water, vegetation cover and previous surfaces compromising the highest score of 1, while the impervious surfaces and less vegetated surfaces compromise the lowest score of 0 (COWI, 2014).

There are few measures can be considered to increase the BGF score, such as stormwater management solutions, increasing green covers and vegetation, also encouraging the biological diversity in urban development projects. Further, these measures can be implemented locally by compromising the impervious surfaces (asphalt and concrete) in the urban area with blue-green infrastructures and with more pervious surfaces (COWI, 2013).

Moreover, the BGF is corresponding with the Natural Diversity law and used as an additional tool to the already existing legal framework. Thus, the GBF is used to secure that the blue-green infrastructures are implemented in the development plan (COWI, 2014).

3. RAINFALL DATA:

3.1 INTENSITY DURATION FREQUENCY: IDF-CURVES:

The Norwegian Meteorological Institute (met) has developed a web portal (eklima.no), aimed to provide the public with an access to the climate database that was based on historical and present data.

The rainfall data used for calibration was obtained from Vestli/Oslo weather monitoring station number 18270. The station is located 200 meter above the sea level and was in operation since 1974, where climate data such as temperature, rainfall, water temperature, and snow melting, runoff and humidity variables were registered and continuously updated between 1974-2013. *Figure 3.1* is providing with IDF curve of Vestli (eklima.no). The rainfall intensity observations were complied into intensity duration frequency –IDF curves[8], including the long-term rainfall records for each respective return period that varies between 2-200 years (Eklima).

[8] *The IDF curve is a graphical representation of the probability of a rainfall event to happen, that varies between 2-200 years EKLIMA. Intensity Duration Frequency curves (IDF) [Online]. Norwegian Meteorological Institute. Available: http://sharki.oslo.dnmi.no/portal/page?_pageid=73,39035,73_39049&_dad=portal&_schema=PORTAL [Accessed 20.01 2014].*

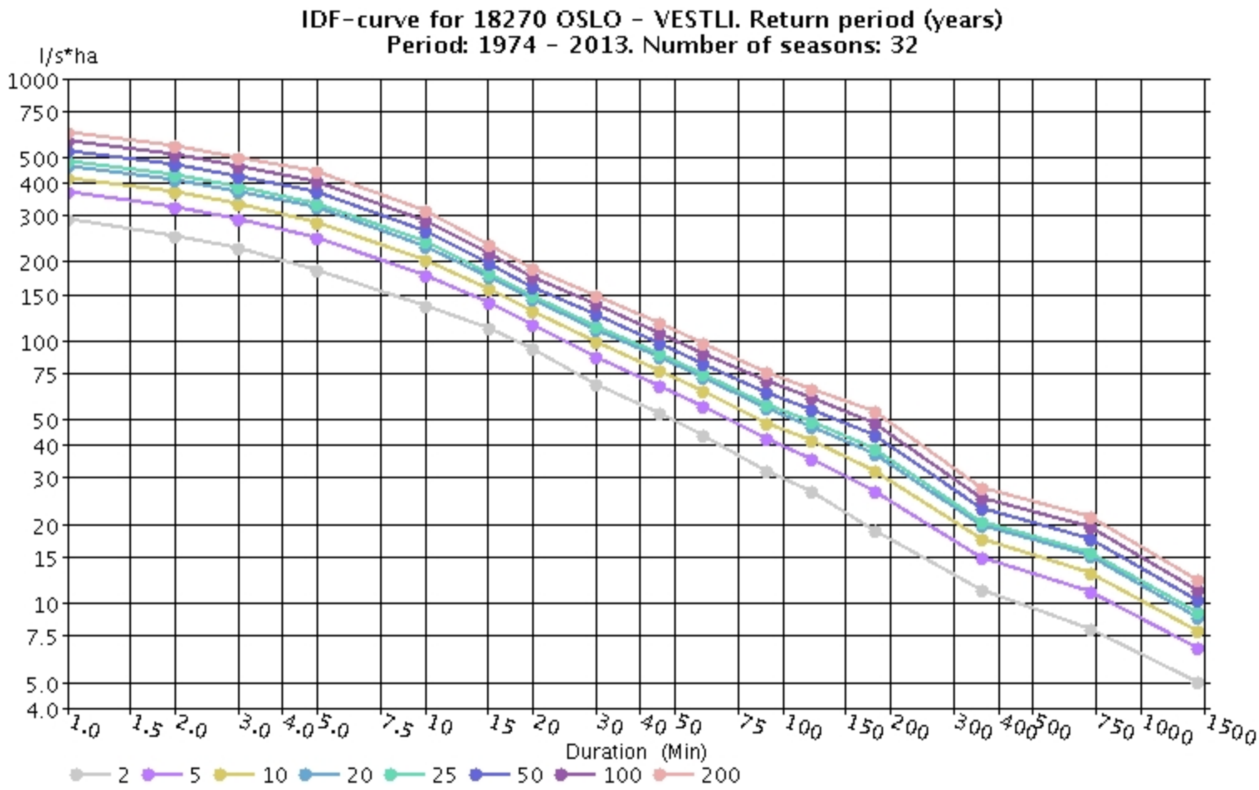


Figure 3.1: IDF-Curves for Vestli /Oslo (Eklima.no)

The rainfall intensity is given either in mm/ min or in l/s. ha

$$\text{Where} \quad \frac{l}{s.ha} = 60 \times 10^{-4} \ \frac{mm}{min}$$

3.2 THE RAINFALL DATA UNCERTAINTY:

The rainfall pattern is not a constant parameter in terms of intensity and variation due to duration and distance between respective areas, even within the same geographical area. *Figure 3.2* explains how three hydrographs can be developed of the same rainfall that was collected from three different pluviographs[9] were placed just few kilometre from each

[9] *Pluviograph: is an instrument for measuring the amount of water that has fallen (i.e. Rain gauge), with a feature to register the data in real time to demonstrate rainfall over a short period of time, often an automated graphing instrument (http://en.wiktionary.org).*

another. It is also confirmed that, if the rainfall allowed to follow the sewer -system main direction and with flow velocity of about 2 m/s, this can increase the surface flow of about 10% comparing of stationed rainfall (Lindholm, 2013).

.

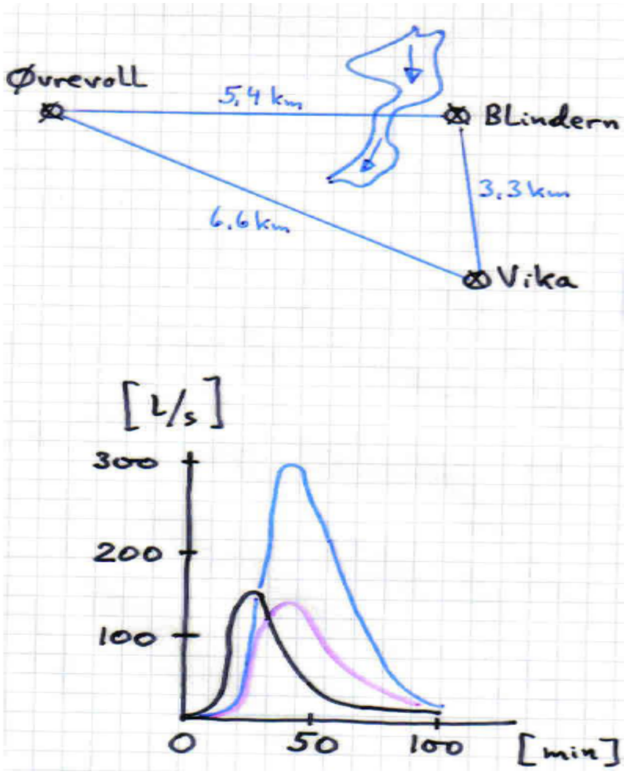


Figure 3.2: Different rainfall intensities varies over duration and distance (Lindholm, 2013)

The NWA recommended not to consider a rainfall data form a rainfall monitoring station that is located more than 3-5 km distance from the respective study area. Therefore, the NWA recommended the Norwegian municipalities to install several rainfall monitoring stations and within the recommended distance to provide with more accurate rainfall data (within 3-5 km in between) (NorskVann, 2012).

3.3 RAINFALL – RUNOFF TRANSFORMATION

With a high uncertainty of temporal rainfall pattern, it is a challenge to predict a realistic data for a future rainfall, thereafter to design a drainage system for generated urban runoff. Still, considering developing a hydrograph based on past rainfall events will provide with more realistic image about runoff expectations for the respective area.

The hydrograph is usually determined by mean of statistical analysis of observed mean rainfall intensity based on IDF-curves. The method can provide with a realistic data of rainfall patterns includes return period, as well as data about the runoff sequences (Hyetograph). This helps to establishing a realistic design for the drainage system (Grimaldi and Serinaldi, 2010).

.

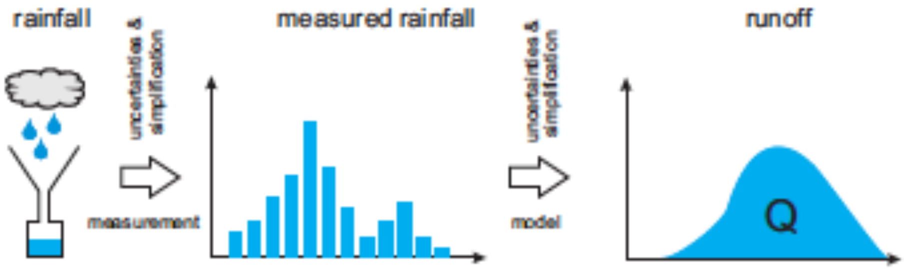


Figure 3.3: Rainfall transformation into runoff (Kleidorfer, 2009)

Figure 3.3 shows the transformation of rainfall into a runoff, where the average rainfall intensity of a storm can be estimated, as well as the time of rainfall distribution over the catchment area. The average rainfall intensity is expressed as a function of time called Hyetograph and the runoff distribution called Hydrograph (Ødegaard, 2012).

The symmetric curve of the hygrograph includes the history of the flow rate, where the peak flow (maximum intensity) represented by the rational formula with climate change considerations the formula is follows: $\boldsymbol{Q = \varphi \times A \times I \times \mathrm{Cf}}$.

Figure 3.4 explains using what is called in Norwegian a ¨Kasseregn¨ or ¨rain box ¨ as a starting point to establish a hyetograph for a rainfall event. Where rainfall intensity (l/s. ha) varies over a time period with respective return time (Lindholm, 2013).

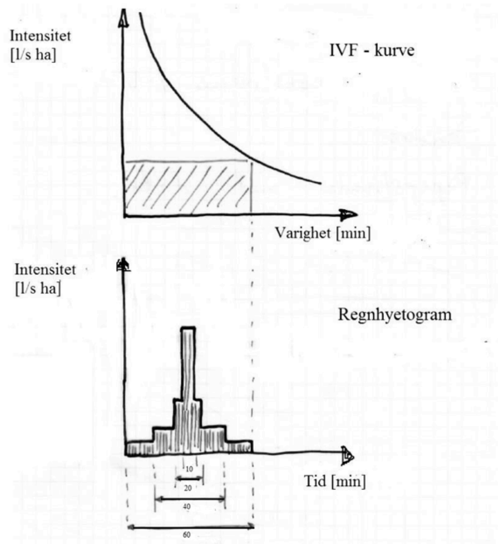


Figure 3.4: Principals of constructing a Hyetograph from IDF-curve (Lindholm, 2013)

3.4 DETERMINATIONS FOR SIZING DETENTION STRUCTURES:

In general, the detention volume calculations are mainly based on calculating the difference between the inlet- Hydrograph and the outlet –Hydrograph in a volume unit. Where the volume balance of high considering (NorskVann, 2012).

Figure 3.5 explains the main principals of detention volume calculations. Where the inlet-Hydrograph is the summation of runoff´s volume from the respective catchment area, and the outlet-Hydrograph is the volume that flows out from the detention structure. Both are given as a function of time with a peak flow obtained form the rational formula $\boldsymbol{Q} = \boldsymbol{\varphi}\times\boldsymbol{A}\times\boldsymbol{I}\times\mathbf{Cf}$.The estimated detention volume is calculated as a difference between the inlet and outlet curves (NorskVann, 2012).

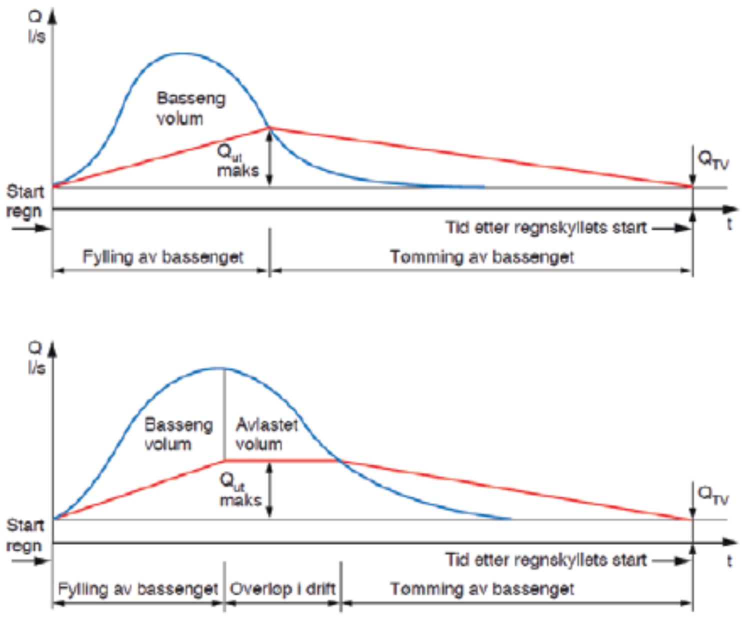


Figure 3.5: The inlet and outlet -Hyetographs with and without (NorskVann, 2012)

The outlet-Hyetograph (Q_{ut}) volume calculations depend on the maximum allowed water level (h) for the detention structure. Where the detention structure shape and depth are also considered. Considering all these variables, the NWA is recommending using advanced computer software (such as SWMM, MIKE URBAN/MOUSE…etc.) to estimate the ideal basin depth. In addition, the manual calculation methods (such as rainfall envelops) are highly recommended to achieve more accurate results (NorskVann, 2012).

3.4.1 RAINFALL- ENVELOPE METHOD:

The rainfall envelope method is based on calculating the inlet and outlet flow in the detention structure, where the flow balance and volume are of concern. Both of the accumulative volume and flow estimations depend on the rainfall intensity that obtained from IDF –curves, where further consideration for rainfall patterns and duration are also significant. Thus, in case of short time period between two or more rainfall events, the structure won´t be able to function as expected (NorskVann, 2012).

However, the NWA recommended three different alternatives to conduct the calculation for detention volume. Still, the alternative B is the most recommended (NorskVann, 2012):

Alternative A: considering the outlet volume is constant over the time period.

Alterative B: considering the outlet volume is variable over the time period and depends on the depth of the attenuation structure.

Alternative C: the Aron and Kilber method (1990) that considering the runoff hydrograph as a trapezoid (see *Figure 3.6*), where the peak flow is represented with rational formula ($\boldsymbol{Q} = \boldsymbol{\varphi} \times \boldsymbol{A} \times \boldsymbol{I}$) (Akan, 2002).

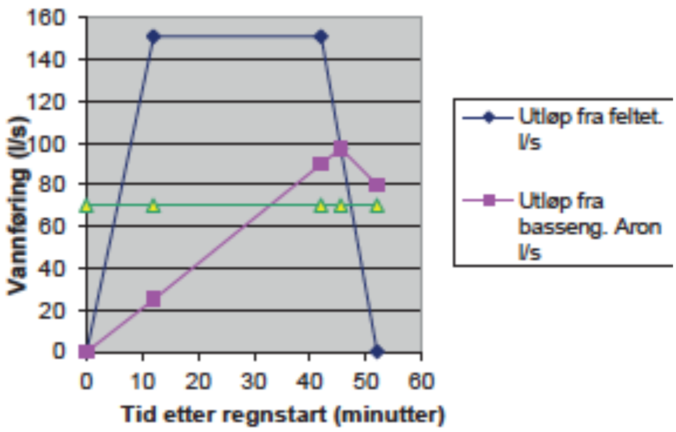


Figure 3.6: Hyetograph for inlet and outlet volume of detention basin (Akan, 2002)

<u>Alternative A</u> with constant outlet volume is considered in this thesis to estimate the size of perspective structures for detention and infiltration purposes. *Figure 3.7* Rainfall envelop for a rainfall event that varies between 0-120 minutes with a constant outlet volume of 70l/s (Lindholm, 2012b).

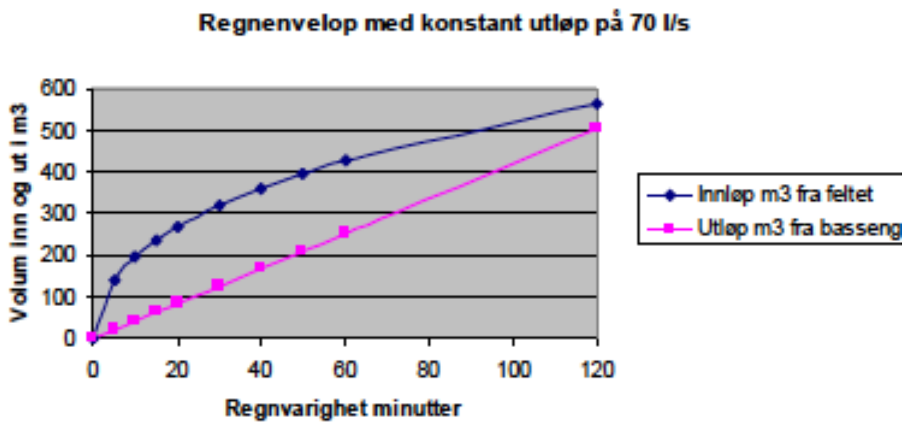


Figure 3.7: Rainfall envelope with a constant flow rate of about 70 l/s (Lindholm, 2012b)

The method depends on runoff flow from the catchment area, and rainfall intensity data that obtained from IDF-curve. The volume is calculated as a function of time (Lindholm, 2012b).

$$V_{inlet} = QXt$$

Where:

Vinlet: Inlet volume (m^3)

Q: Runoff flow rate after rainfall event (l/s)

t: Rainfall variation time (min)

In order to avoid the risk of loading the drainage system, each municipality predefined the maximum allowed limits for runoff discharge. The discharge can be either to the nearby water body or to the conventional system.

Table 3.1 shows the requirements for the maximum allowed runoff discharge (Qout). Hence, these values must be considered when designing drainage system in new urban area or expansion an existing one (Ramboll, 2015).

Table 3.1: General assessments related to municipal runoff provision (Ramboll, 2015)

Area	Requirement	Comment
Natural areas- non urbanised	Maximum runoff 10-15 l/s.ha under 25years return period (similar to natural runoff)	In natural areas the runoff follows the natural topography patterns
Urban areas	Urban expansion must not induce increase in runoff	
General	Urban expansion must not increase the runoff. Demands attenuation structures aiming that runoff not exceeds 10 l/s.ha	

The maximum allowed discharge is summarised with the following formula:

$$\boldsymbol{Q_{outlet\ vol.}} = \boldsymbol{Q_{discharge}} \times \boldsymbol{total\ area}$$

Where:

Q_{outlet}: Constant outlet runoff flow (l/s)

$Q_{discharge}$: Allowed runoff flow discharge per unit area that varies from municipality to another (l/s.ha)

Total area: Catchment area (ha)

Thereafter the volume of allowed runoff discharge, is express as a function of time, with the following formula:

$$\boldsymbol{V_{outlet}} = \frac{\boldsymbol{Q_{outlet\ vol.}\ X\ t\ X60}}{\boldsymbol{1000}}$$

Where:

V_{outle}t: Outlet runoff volume after rainfall event (m^3)

$Q_{\mathbf{out}}$: Allowed runoff flow rate (l/s)

t: Rainfall duration (min)

Also the inlet volume must be considered and can be calculated using the following formula:

$$\boldsymbol{V_{inlet\ vol.}} = \frac{\boldsymbol{Q_{inlet\ vol.}\ X\ t\ X\ 60}}{\boldsymbol{1000}}$$

Where:

$V_{inlet\ vol.}$: Calculated inlet volume (m^3) from the catchment to the retention structure

t: Rainfall duration time (min)

And

$$Q_{inlet\ vol.} = \mathbf{A\varphi I}$$

Q $_{inlet\ vol.}$: Runoff flow from the catchment

A: Area of the catchment (ha)= 30 ha

φ: Runoff Coefficient

Rainfall envelope with a constant flow rate is summarised with the following formula:

$$V_{reten.vol.} = V_{\ inlet\ vol.} - V_{\ outlet\ vol.}$$

Where:

$V_{reten.vol.}$: Volume of retention (m^3)

V $_{inlet\ vol.}$: inlet volume into the pond (m^3)

$V_{outlet\ vol.}$: outlet (discharge) volume from the pond (m^3)

In both of the cases, the inlet volume and outlet volume, the peak flow is expressed using the rational formula ($Q = \varphi \times A \times I$).

4. MATHEMATICAL MODELS

4.1 BACKGROUND

The word ´model´ used in this study is referring to the US Environmental Protection Agency Storm Water Management Model, SWMM version 5. SWMM is developed to evaluate dynamics of the rainfall-runoff for a single rainfall event or continuous long-term rainfall events, hence, the model simulate the runoff quantity and quality from post-urbanized areas (Rossman, 2010).

Ever since SWMM was developed in 1971, it went through several upgrades phases in order to meet the climate changes and the users needs. The latest upgraded version of SWMM is called Storm Management Model Climate Adjustment Tool (SWMM-CAT) version 5.1.007, also with Low Impact Development (LID) controls such as green infrastructures and permeable pavement, rain gardens, green roofs, street planters, rain barrels, infiltration trenches and vegetative swales. The new version incorporate the future climate change projections into the classical version of SWMM (EPA, 2015)

In addition, using SWMM for flow simulation in a specific catchment area required more specific information about the surfaces of the catchment and also the sewer system. Further, the developed model also required checking/testing, calibration and later verification for the model to be considered (Bulter and Davies, 2011).

4.2 DESIGN STORM

Due to the non-stationarity in rainfall records, the rainfall data was extrapolated in order to establish potential rainfall scenarios that might valid also in the future. The synthetic rainfall data based on historical rainfall data (1974-2013) obtained from IDF-curves for Vestli/Oslo area. Also both time series and return period were predefined. The climate change factor was also considered.

Further, SWMM will be used for runoff simulation in both proposed drainage systems (see Alternative 1 &2). Hence, the runoff simulation will estimate the catchment's response to design storm.

4.2.1 DESIGN STORM CALIBRATION

In general, one can define calibration as the collection of acts that under certain conditions create a relationship between values were given by a measuring instrument, measuring systems or any other method of measuring, where material goals and values of the realized norms (Engan, 2011). But in order to conduct an actual calibration for a model, historical data must be considered including measurements were taken over a long period of time for a catchment; this can includes also rainfall and measurements for stormwater flow into the pipes. Further, a comparison between results achieved from measured and calculated data can be conducted followed with an adjustment for different parameters that influence the results such as runoff coefficient (φ-value), pipe roughness, flow time…etc. (Lindholm, 2013). In this regard, the maximum flow peak (Q-max) into the pipe is of concern to avoid inundation and flooding in the drainage system (Barton, 2010).

The following formula gives the average difference between the measured and calculated runoff. In case $Q_b < Q_m$ then Q_{max} is negative, but if $Q_b > Q_m$ this gives a positive Q_{max}, in another word Q_{max} is sensitive parameter and depends on the both of the calculated and measured values of runoff flow.

$$Q_{max} = \frac{(Q_m - Q_b)}{Q_m} X\ 100\%$$

Where:

Q max: Maximum runoff

Q_m: Maximum measured runoff

Q_b: Maximum calculated runoff

4.2.2 DESIGN STORM VERIFICATION

The verification is following the calibration and is subjected to the trial- and error process and the personal judgment for the modeller (Bulter and Davies, 2011). Hence, this process is enable the modeller to adjust /verify the different physical features and details for the sewer system, in another word weather these features are integrated correctly into the model and in case they are having any significant impact on the model results or not (Bulter and Davies, 2011).

4.2.3 DESIGN STORM –SENSITIVE ANALYSIS

In short, the sensitive analysis is a tool used for mapping the relevant parameters hat have influence on the model, the relation between the different parameter is illustrated in form of Star –diagram (Buhler, 2013)

Moreover, this analysis is conducted for each individual parameter in order to estimate which parameter has the most influence on the model (Lindholm, 2013).In order to establish the model of a catchment, there is a wide range of parameters were used as an input in SWMM, such as pipe roughness, the width of the catchment, the slope, roughness and proportion of the impervious surfaces, in which each of these parameters is dependent on the other parameters. But in case of designing a runoff drainage system the maximum flow runoff (Qmax) is the most of concern, but it depends on the runoff- time that is depends in return on the width of the catchment.

The following star- diagram was adopted from Lindholm (2012), where sensitive analysis was conducted for a calibrated model for a rainfall event from 12.08.2009 that was used for simulation.

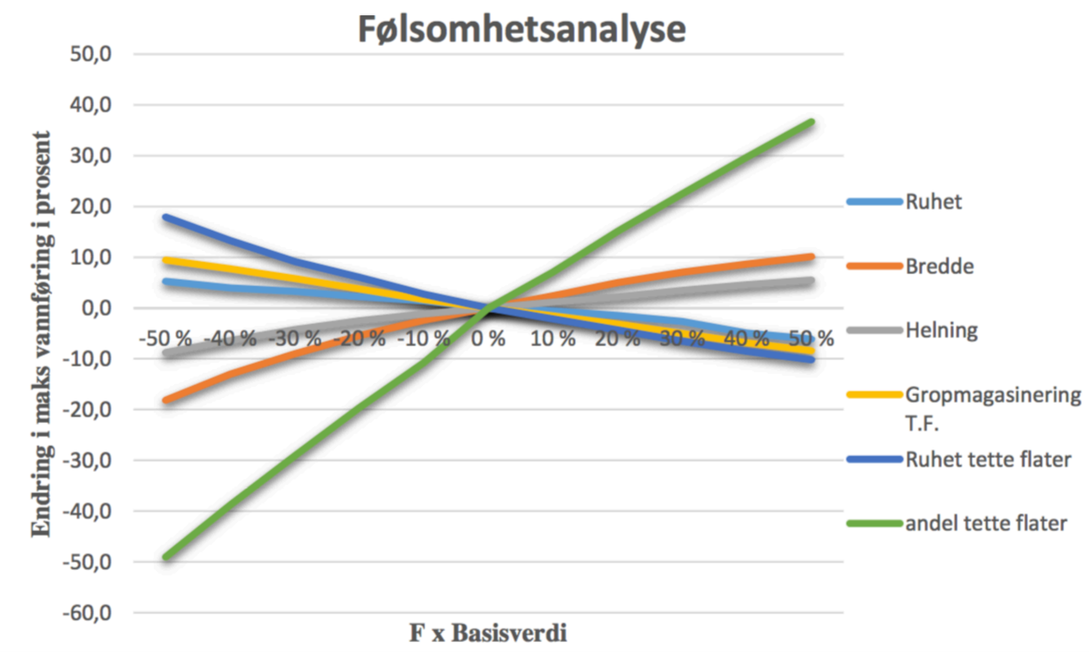


Figure 4.1: example for Sensitive analysis (Lindholm, 2012b)

4.2.4 DESIGN STORM CHECKING/TESTING

Checking/testing the model is aimed to check weather the model behaviour is satisfactorily in terms of mathematics or not, also it helps to locate the mistakes in the input data and to assess the volume losses in the drainage system (Bulter and Davies, 2011).

Further, the good calibrated model is given as a *F-value*. This value is based mainly on estimation for the maximum runoff flow (Qmax), the total runoff volume and time for occurring the maximum runoff flow. The relationship between these parameters is illustrated in the following formula, where in case the calculated parameters are equals to the measured ones then the F-value is equals to ZERO; i.e. the model was good calibrated. However, the high or low F-value is requiring re-evaluation of the parameters.

$$F = V_1 \sum (Qm - Qb)^2 + V_2 \sum (Vm - Vb)^2 + V_3 \sum (Tm - Tb)^2$$

Where:

F: is the object function (parameter) that needs to be reduced, summation for the actual rainfall.

V: is weight number. Total of V_1, V_2, V_3 = 1.0

m: measured value

b: calculated value

Q: maximum flow peak in l/s

V: water volume in m^3

T: time to approach Qmax

4.3 PARAMETERS IN SWMM

4.3.1 SUBCATCHMENT

SWMM required subdividing the catchment area into a number of subcatchment. Each subcatchment is considered as an independent hydrologic unit with predefined topography and drainage system. Where the runoff can be routed through nodes in the drainage system or to another subcatchment.

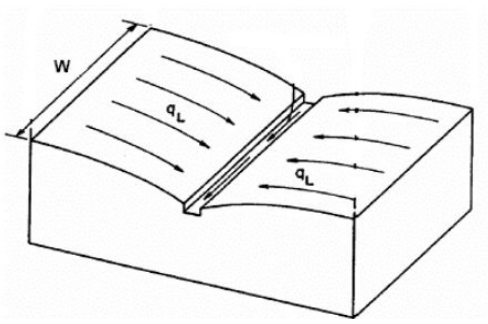


Figure 4.2: Idealised subcatchment (Rossman, 2010)

Figure 4.1 shows how the overland flow can be estimated at SWMM, where the idealised subcatchment is illustrated as a rectangular and with a uniformed slope along the subcatchmnet and the flow drains into a single outlet channel.

The features of each subcatchment must be also assigned (Rossman, 2010):

- Assigned Rain Gage (s)
- Outlet node or subcatchment
- Assigned land uses
- Tributary surface area
- Imperviousness
- Slope
- Characteristic width of overland flow.
- Manning´s for overland flow *n* on both pervious and impervious portions
- Depression storage *n* both pervious and impervious areas
- Percentage of impervious area with no depression storage

Slope: Using the GIS tools and the available online mapping software helps to localize the catchment area on the map, thus defining the location X and Y- coordinates as well as the average percentage slope (head loss per unit length). Defining the catchment slope is important to evaluate the natural drainage pathways (Rossman, 2010).

$$\%Slope = \frac{altitude\ 2 - altitude\ 1}{Length} * 100\%$$

The imperviousness of the subcatchment: it is important to divide each subcatchment into pervious and impervious portions, depends on the surfaces permeability. The pervious portion is the most permeable; means the runoff is subjected to losses to the unsaturated upper soil zone due to infiltration, but the impervious portion has lower infiltration capacity and can relatively produce 100 % runoff after a rainfall event.

<u>The Manning's roughness coefficient n</u>: reflects the degree of resistance that the overland flow might meet while running on the surfaces, therefore it is important to be defined for impervious and pervious portions (see ***Table 10.3*** in appendix A).

<u>Depression storage:</u> the depression capacity varies according to the surfaces, in another word the ability of the surfaces to fill up the pores before the runoff takes place. Is about 1.2 mm for impervious surfaces and 7.62 mm for green areas (Rossman, 2010), (see ***Table 10.1*** in appendix A).

<u>Subcatchmnet width:</u> SWMM manual is defining the subcatchmnet width as

$$Width = \frac{\text{Subctahcmnt area}}{\text{length of the longest overland flow path}}$$

Where the overland flow length, is defined as the longest flow path that the water can approach ((Rossman, 2010), p17). The maximum lengths from few possible pathways must be averaged and can be used to determine the subcatchmnet width. These pathways must reflect slow flow rather than rapid flow for attenuation purposes (Rossman, 2010).

The runoff infiltration in the pervious portion can be modelled through Horton, Green-Ampt or Curve Number infiltration models. However, the snow accumulation, re-distribution and melting can be also modelled but first after Snow Pack object is assigned. Also one can model the groundwater flow but a set of Groundwater parameters must be assigned first. The Land Uses ´s associated pollutants and wash-off from subcatchment can be also modelled as well as different types of LIDs can be modelled by assigning a set of predesigned LID controls to the subcatchment (Rossman, 2010).

4.3.2 CONDUIT:

The runoff can be transported into conduits/ pipes or channels from one node to another in the drainage system, where the natural flow is considered. SWMM provides with a variety of open and closed pips depends on cross-sectional shapes of each (see ***Table 10.5*** Appendix A) (Rossman, 2010).

SWMM used the Manning Equation to estimate the flow rate in each conduit, where the relationship between the runoff flow – rate (Q), cross sectional area (A), hydraulic radius and slope (S) are expressed in the following equation. Hence, the conduit roughness coefficient is variable value and depends on the material of the conduit. The *Manning's roughness coefficient (*$\boldsymbol{n}$*)* expressed the conduit´s roughness (see ***Table 10.2 & Table 10.4*** in Appendix A).

$$\boldsymbol{Q} = \frac{\mathbf{1.49}}{\boldsymbol{n}}\ \boldsymbol{A}\boldsymbol{R}^{\mathbf{2/3}}\boldsymbol{S}^{\mathbf{1/2}}$$

Where

Q: Flow rate (l/s)

A: Cross-sectional area of the conduit

R: Hydraulic radius

S: Slope of the conduit or friction (%)[10]

n: Manning's roughness coefficient

In order to enable SWMM to follow the flow rate in conduits, there is few other input parameters must be predefined (Rossman, 2010):

- Name of the inlet and outlet node
- Offset height or elevation above the inlet and outlet node inverts
- Conduit length
- Manning´s roughness
- Cross-sectional geometry

---

[10] *S: is the head loss per unit length. It can express either the conduit slope in case of stable flow and under kinetic routing, but it also can express the friction slope in case of dynamic routing (Rossman, 2010).*

Also as an optional parameters:

- Entrance/ exit loss. As an optional parameter
- Presence of a flap grate to prevent reverse flow

4.4 CALIBRATION DATA

By using SWMM one can compare simulation´s results with measured data from the field in its Time Series. Still, the calibration data must be entered into specially formatted text file to be registered with the project (Rossman, 2010).

The Calibration files includes different measurements of a single parameter for one or varies locations, this data can be compared with simulated values in Time Series Plots. Different files can be used to register different parameter, few to be mentioned here (Rossman, 2010):

- Subcatchment Runoff
- Groundwater Flow
- Node Depth
- Node Flooding

4.5 COMPUTATIONAL METHODS ON SWMM:

SWMM is based on few physical processes, where principals of mass and energy conversations are employed. These processes are aimed to estimate the stormwater runoff in terms of quantity and quality (Rossman, 2010). Considering the previous mentioned assumptions, this thesis is considering the runoff quantity only. Hence, the following physical processes are considered:

- Surface (overland) runoff
- Infiltration
- Surface ponding

4.5.1 SURFACE (OVERLAND) RUNOFF:

In principle, the surface of each subcatchment is considered as a non-linear reservoir, where the inflow followed a precipitation event or can be form another upstream subcatchment. The outflow can be several of them, for example in form of losses due to infiltration and evaporation or surface runoff. *Figure 4.2* explains this principal, where the capacity of this reservoir considered as the maximum depression storage or surface storage what is called (Rossman, 2010).

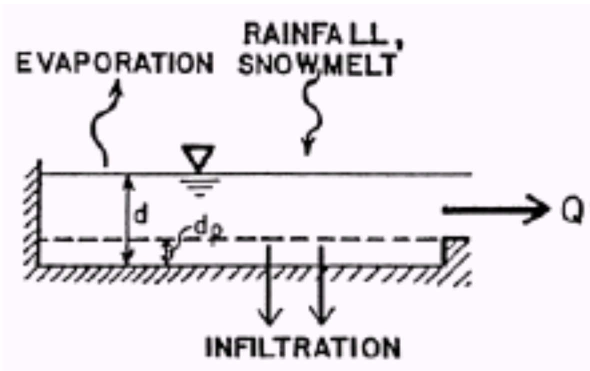


Figure 4.3: Conceptual view of surface runoff (Rossman, 2010)

According to Butler, the depression storage accounts for rainwater that has become trapped in small depressions on the catchment surface, preventing the water from running off (Bulter and Davies, 2011), p.108)

However, the depression storage depends on the permeability of the surfaces. ***Table 4.1*** includes different values for depression storage that are considered on SWMM given in mm (Rossman, 2010).

Table 4.1: Depression storage (Rossman, 2010)

Depression storage	Capacity (mm)
Impervious surfaces	1.2-2.5
Lawns	0.10-0.20
Pasture	5.08
Forest litter	7.62

Hence, the surface runoff Q can take place only when the water depth (d) exceeded the maximum depression storage (d_p) means d> d_p, thus the Manning's equation is applied. The surface depth is consciously updated(Rossman, 2010).

$$\boldsymbol{Q} = \frac{\mathbf{1000} \times \boldsymbol{A} \times \boldsymbol{R}^{2/3} \times \boldsymbol{S}^{1/2}}{\boldsymbol{n}}$$

Where:

Q: surface runoff per unit area (l/s)

A: area of the subcatchment

R: Hydraulic radius

S: Slope of the conduit or friction (%)[11]

n: *Manning's roughness coefficient. i.e.* (see ***Table 10.3*** Appendix A- *Manning's n* for overland)

4.5.2 INFILTRATION:

Infiltration is a process that follows a rainfall event, where water penetrates the ground surface into the unsaturated soil zone. The infiltration capacity of a soil is defined as *the rate at which water infiltrate into the soil*, hence it depends on the surface permeability, soil type, soil structure and compaction, the initial moisture content, surface cover, and the depth of the water on the soil (Bulter and Davies, 2011).

Considering the mentioned above features that deciding the infiltration capacity, still the soil hydraulic conductivity is an issue in order to meet the perceived amounts of water and infiltrate it as quick as possible. Infiltration is given in m/s or m^3/ day. *Figure4.3* explains the

---

[11] S: can express either the conduit slope in case of stable flow and under kinetic routing. but it also can express the friction slope in conduit of dynamic routing (Rossman,2010)

different types of soil and their hydraulic conductivity. It is recommended to consider the soil type when planning infiltration zones within the urban areas (Endresen, 2009).

Figure 4.4: Soil diagram (Endresen, 2009)

Further, it is recommended to consider the seasonality in Norway to avoid the flash floods. Where seasonal frost cause blockage of soil pores, consequently reduce the soil´s infiltration capacity and increase the risk for flash flooding. Instead one can avoid surface infiltration in the frost season and consider the infiltration deeper enough under the frost zone (Endresen, 2009).

For environmental and economic considerations, infiltration is convenient (Endresen, 2009):

- Infiltration helps to minimize pipes within the stormwater network
- Infiltration can reduce the risk of floods, as routing the runoff into pipes will increase the flood in the recipient
- Infiltration influence the water balance in the respective area, as routing the runoff through pipes will influence negatively both the groundwater balance and vegetation cover

- In general the stormwater contains pollutants, specially the water from roads. Hence, infiltration function as a natural filter. This can varies and depends on the soil type

As mentioned above, SWMM is using different models, however in this thesis Horton's infiltration model is adopted. Where the parameters used by this model are the followings ((EPA, 2009), p.19):

- *Maximum infiltration rate*: is considered the initial rate as the storm starts. Cannot easily estimate since it depends on the soil moisture content.
- *Minimum infiltration rate:* is the limiting infiltration rate and is equal to the soil hydraulic conductivity.
- *Decay coefficient:* is significant parameter in determining the infiltration ¨decays¨, that range between the initial maximum values to the minimum values.

The Horton's Equation mainly describes the reduction in infiltration from initial maximum rate to a minimum rate over a period of time. Thus the infiltration process at a high and constant rate (f_0) then decreases exponentially over time period (t) until it stabilised it self at – a steady state (f_c) - and becomes constant as the soil approaching saturation state (Bulter and Davies, 2011).This can be expressed with Horton´s Equation:

$$\boldsymbol{f_t = f_c + (f_0 - f_c)e^{-kt}}$$

Where:

f_t: Infiltration rate at time t (mm/h)

f_c: Final (steady state)infiltration rate or capacity is constant (mm/h)

f_0: Initial infiltration rate (mm/h)

K: Reduction constant for soil infiltration (decay coefficient) [12] (h^{-1})

---

[12] *K: decay coefficient describes how fast the infiltration rate decreases over time period (Rossman,2010)*

t: time required to approach the fully saturation state to completely dry state

SWMM demands to predefine the infiltration parameters including maximum and minimum infiltration rate also the decay coefficient also the time that soil takes to dry off completely (see Appendix A both ***Table 10.5*** in - The infiltration parameters for Horton infiltration also ***Table 10.7***-Decay constant values and soil drying time).

Figure 4.4 explains the Horton´s model, as more rain received the infiltration decreases exponentially over a period of time. Also, explains the concept effective rainfall ¨, i.e. the rainfall increases and approaches the infiltration capacity of the soil thereafter started to flow on the surface and contributes to the surface runoff (Sælthun, 2013). Still, the Horton's Equation is limited to infiltration surfaces *i.e.* the pervious surfaces.

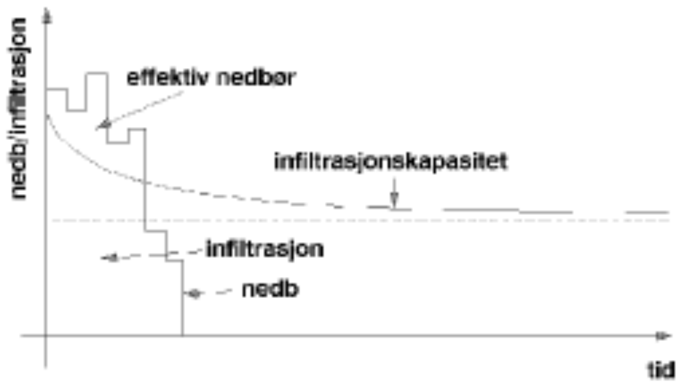


Figure 4.5: Effective rainfall event -Horton's infiltration model (Sælthun, 2013)

4.5.3 SURFACE PONDING-MANHOLES SURCHARGE

The surface ponding is taking place when the flow continuing until exceeds the available capacity of the system; thereafter reduce the capacity to transport the water further to the downstream points. The overflow volume ponds through junctions and might be lost from the network (ponding off). *Figure 4.5* explains the ponding -off and ponding- mechanisms, however both mechanisms are caused by the same reason (Lindholm, 2013).

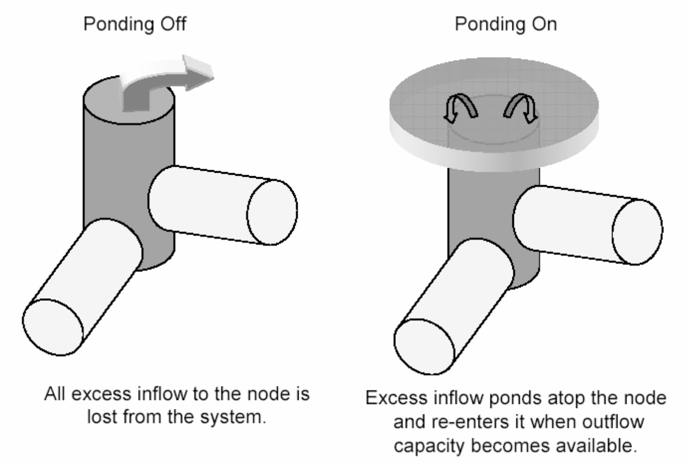


Figure 4.6: The node/junction flooding options (Lindholm, 2013)

In pipe-based alternatives, the overflow atop the junction (manhole) is taking place when the flow exceeding the designed capacity of the system. But while the ponding- off mechanism leading to total inflow losses from the system, the ponding-on allowing overflows routing within a predefined ponding area and to retain later into the system when the capacity permits.

But in pipe-free alternatives, where the overflow volume can be introduced into channel systems such as road overflows, vegetation area (Swales) and floodplain storage areas. However in close conduit network the overflow volume can be routed into flood pathways such as down streets to next available downstream stormwater inlet or to either to an open channel or a depression storage unit (Rossman, 2010).

In this thesis the flow routing is based on what is called the Dynamic Wave routing to avoid loosing the overflow volume from the system. This routing depends mainly on the water depth of the nodes/junctions; it is possible that the overflow volume can be assumed to pond over the node but within a constant surface area that is predefined. In this thesis the ponding area in the closed conduits system is predefined with 100 square meters around each node to secure preserving the overflow volume. Alternatively Swales and detention basins are introduced.

4.6 FLOW ROUTING:

Equations of mass and momentum conservation are significant at SWMM, hence flow routing within each conduit link is regulated by these equations. One can choose one of the followings to solve these equations (Rossman, 2010):

- Steady Flow Routing
- Kinematic Wave Routing
- Dynamic Wave Routing

This thesis is considering the Dynamic Wave Routing to control the routing of conduit flow within the drainage system.

4.6.1 DYNAMIC WAVE ROUTING:

The Dynamic Wave Routing is used on SWMM to estimate the unsteady flow in open channels, part-full conduits, channel storage, backwater and for both reversal and pressurized flow. It can be applied in any network layout in general (Rossman, 2010).

This routing is able to solve the complete one-dimensional flow equations of Saint –Venant, both of the continuity equation and the dynamic (momentum) equation and a volume continuity equation at nodes for the entire conveyance system. This method is used to simulate all available flow conditions. Hence, it provides with a realistic data about the flow in conduits and the volume at nodes.

The Saint -Venant continuity equation (Kleidorfer, 2009):

$$\partial y/_{\partial t} + D_h \, \partial V/_{\partial x} + V \, \partial y/_{\partial x} = 0$$

Also

The Saint- Venant momentum equation (Kleidorfer, 2009):

$$\partial V/_{\partial t} + V \, \partial y/_{\partial x} + \partial y/_{\partial x} = g(S_0 + S_f)$$

Where:

V: flow velocity

y: flow depth

D_h: Hydraulic depth, A/B

S_0: Bed slope

S_f: friction slope

X: distance

t: time

G: acceleration of gravity

Using the Dynamic Wave routing provides with accurate information about flow conditions; hence weather the conduit is full or about to become full. This is also significant measure to avoid flooding and surface ponding flooding/inundation, as the water depth at a node is constant and flow can continue and exceeds the available depth, thereafter causing surface ponding.

4.7 MATHEMATICAL MODELS RELIABILITY:

Using the mathematical models gives an insight of the system and overview over all available variables, also the achieved results. With the available modern technology, one can conduct several operations such as calculate, analyse, design and simulate in such a short time comparing to the method conducted manually. Thus, including wide range of alternatives and factors to include in the operations (Lindholm, 2013).

Still, the mathematical models can be subjected to errors that increase the risk to reduce the results reliability. Still, the sensitive analysis is recommended to reduce the risk of data uncertainty (Lindholm, 2013).

Part2

Analysis part

5. STUDY SITE DESCRIPTION –KJELSRUD CATCHMENT

This chapter is aimed to give an overview of the Kjelsrud catchment and the intended urbanisation plans.

5.1 POPULATION AND URBANISATION IN ALNA DISTRICT ADMINISTRATION

Kjelsrud is located within the Oslo municipality in the Alna district administration area in Grorud Valley, Figure 5.1, and highlighted Alna in red colour. Alna is undergoing tremendous population growth and urban development.

The total population in Alna as of 1 January 2015 was approximately 48 770 inhabitants including the sub-district administration areas of Furuset, Ellingsrud, Lindeberg, Trosterud, Hellerudtoppen, Tveita and Teisen. Alna is expected to grow to about 54 847 inhabitants by the end of 2020 (OsloMunicipality, 2015a).

Figure 5.1: The location – the Oslo municipality and Alna district administration. (http://no.wikipedia.org)

Urban development plans are being developed for the Alna area in order to address expected population growth. ***Table 5.1*** shows the gradual urbanisation development in Alna during the last four years, thus this expected to escalate in the coming years (OsloMunicipality, 2015b)

Table 5.1: Urbanisation growth in Alna district administration (OsloMunicipality, 2015b)

	2011	2012	2013	2014
Housing unit per year	7	64	28	142

5.2 THE LOCATION: KJELSRUD

The catchment, Kjelsrud is located northeast of center of Oslo. It is bordered by Alna park on the north, Strømsveien on the south, Nedre Klabkkvei on the west and Gamle Leirdalveien on the east (se *Figure 5.2*).

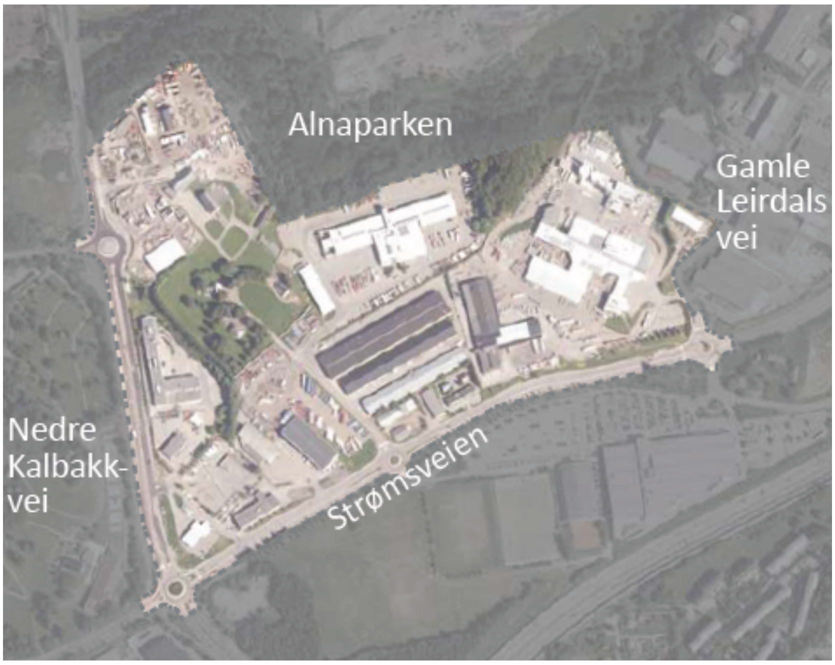


Figure 5.2: Aerial photo of Kjelsrud(ARC, 2013)

Figure 5.3: Map of Kjelsrud catchment (Norgeskart.no)

5.2.1 HISTORICAL BACKGROUND

The oldest tracks of Grorud Valley´s history are 6000 years old and were found in Stig and Årvoll areas. Agriculture was introduced to Norway between 4000 and 1700 BC. Traces of agricultural activities found in the valley date back to this period. The waterfalls in Alna River were used as source of energy in the valley, both for operating carpentry and mills and later for the production of bricks. The workshops along Alna provided building materials to Oslo and encouraged urban development in the region. Strømsveien was one of the oldest and most important streets in Grorud Valley. From the 1600s -1700s the street played a central roll in transporting building materials to Christiana until 1854. The street became less important as when the railway was established (ARC, 2013).

Figure 5.4: Photos of Kjelsrud Farm (Groruddal)

The original name of the Kjelsrud area was Nordre Leirdal. A reference to Leirudalr was found in Bishop Eystein's land book (The Red book) from the 1390's. Bishop Eystein Aslakson was responsible for documenting church income and land ownership in southern Norway. At that time St. Clement's Church in Oslo owned parts of the Kjelsrud (Leirdalgård) farm, therefore the farm was mentioned (ARC, 2013)

Between 1832-1837, 2/3 of the farm was bought by Lard Halvorssøn and 1/3 of the farm was bought by a gardener Carl Christian Fredrik Nielsen., hereafter it was owned by Halvorssøn and Nielsen families (Groruddal)

5.3 URBANIZATION PLAN IN KJELSRUD:

5.3.1 BACKGROUND:

The location and the potential for urban development in Kjelsrud have attracted the interest of the real estate company NHP. The company decided to invest in the long-term development for the area, which may transform Kjelsrud from being an industrial area to a modern urban area. However, the implementation of the project is awaiting the approval of Oslo municipality approval. It is hoped that the cooperation between the NHP and the architecture firm Arc Arkitekter AS will result in a multifunctional area with emphasis on solid blue-green stormwater infrastructure such as stormwater mitigations and infiltration areas.

5.3.2: LAND-USE IN KJELSRUD

The current land-use in Kjelsrud is industrial, consisting of small-scale warehouses and production facilities to larger specialised industrial facilities. Population growth in Kjelsrud is currently stagnant. However the urbanisation plan is expected to add additional 2500 houses to the catchment in addition to the Norwegian Police University Collage in field D (see *Figure 5.2*). The construction plan will be implemented over three phases that started within 5 years and will last up to 30 years (ARC, 2013).

Figure 5.5: The urbanisation plan in Kjelsrud (ARC, 2013)[13]

[13] *For better resolution see Figure 10.2 in Appendix B*

The total area of the catchment compromise 38 ha. *Figure 5.5* shows urbanisation plan that NHP and Arc are intending to implement. ***Table 5.5*** listing the planed land-use for the catchment as part of the urbanisation plan. Both ***Table 5.5*** and *Figure 5.5* gives an overview about the expected transformation of Kjelsrud from an industrial area into housing area.

Table 5.2: The planned land-use for Kjelsrud (ARC, 2013)

Land use	Field name	Area (m2)	Area (ha)
Commercial	A1,A2,B1,B2	60020	6,00
Residential	A1,A2,B1,B2,C1,C2,E1,E3, G1&G2,G4-G6, H1-H3	176960	17,70
Offices	E1 &E3, G6	29700	2,97
Existing industrial	F	12280	1,23
Hotel& conference	E1	17440	1,74
School	G3	16500	1,65
Training center	E1	5790	0,58
The Norwegian Police University College	D	50000	5,00
Cultural center	A3	5220	0,52
Parking		4790	0,48
Divers		4220	0,42
Total		382920	38,292

5.3.3 BLUE-GREEN STORMWATER INFRASTRUCTURE

Figure 5.6 shows natural existed and the planned Blue-Green structures[14]. This includes Alna River, Kjelsrud Water stream, Kjelsrud Park, Ravine Forest, this will significant in providing the area with Blue-Green profile (see under 2.10).

Figure 5.6 Blue-Green infrastructure in Kjelsrud (ARC, 2013)

[14] *Blue-Green stromwater infrasturcture: is to design a system to capture and collect runoff after rainfall event for either storage to use purposes or for infiltarte it back into the ground. The goal is to keep the stormwater out from the sewer system to creat conditions similar to those before urban development (CITYOFCHIAGO. 2014. City of Chicago- Green Stormwater Infrastructure Strategy [Online]. Chicago: City of Chigao. Available: http://www.cityofchicago.org/content/dam/city/progs/env/ChicagoGreenStormwaterInfrastructureStrategy.pdf [Accessed 29.07 2015].*

6. STORM WATER –RUNOFF TRANSFORMATION

This chapter is focused on establishing a design storm limited to the catchment, thereafter estimating the amount of generated runoff that needed to be reduced.

6.1 DESIGN STORM:

The design storm is considered as a representative input for the change in the precipitation pattern limited to the catchment. But due to the instability of the rainfall in terms of both intensity and frequency, the design storm was represented through two symmetric hyetographs associated with a return period determined by IDF-curves that was obtained from Vestli gauging station, where the data was of a good quality and updated contentiously.

Further, both of the hyetographs were based on a 20- years rainfall data with 60 minutes duration. Still, the second hyetograph considered the impact of future climate change that was represented with an additional value (C_f-value) to today's rainfall data.

Both of the hyetographs were assigned at SWMM under (Rain Gage), both were also considered the catchment as a uniform unit and both were applied evenly to the catchment for further runoff simulation.

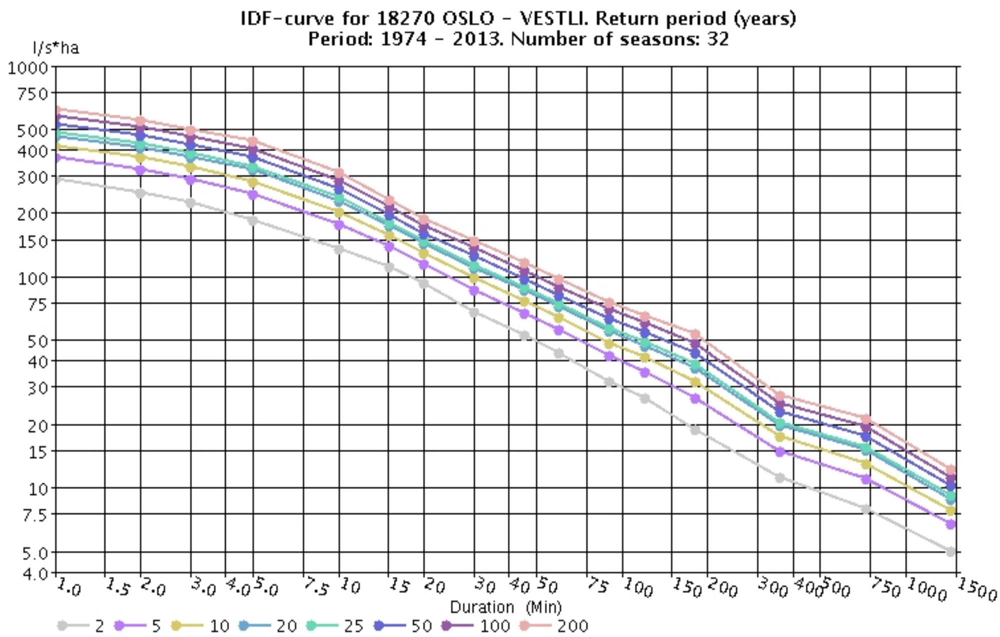


Figure 6.1: IDF-Curves for Vestli /Oslo (Eklima.no)

6.1.1 DESIGN STORM CALIBRATION

Following to Oddvar Lindholm recommendations, since the development plans for the catchment are yet not finalised; including the drainage system, one cannot conduct a relevant calibration for the applied model. Still, calibration for the design storm can be conducted later on to achieve a relevant data for the representative storm. Further, the other relevant model requirements including model verification and checking/testing as well as sensitivity analysis are all dependent on the calibration results, therefore wont be conducted at this stage for the same reason..

6.1.2 THE RAIN HYETOGRAPH METHOD:

The rain hyetograph method is based on an actual data from IDF-curves, where the instability in the rainfall intensity is distributed over time period. Hence, the rainfall intensity is a significant factor and develops an idea about the expected runoff from the rainfall event.

Following to Lindholm (2008), the principals behind the construction of rainfall hyetograph are the followings:

- The construction of the rain hyetograph is based on the IDF- curve of the perspective area.
- Presumed the rainfall is a symmetric at its central axis
- The rain volume within the x most intensive minutes (X/2 minutes at each side of the middle line) in the hyetograph is compromise the rain volume from a rain box at the same presumed X minutes
- The time- interval variation is between 2-5 minutes, where only 1 minute can give substantial high peak intensity. In this thesis 5 minutes time- interval variation is considered.

In addition, there are a number of parameters should be predefined:

- Assigned return period
- Calculating the time of concentration
- Decide the rain intensity based on IDF- curves

- Consider symmetric distribution of the data

The return period or the re-occurrence of flood event is an important parameter to be considered to adjust the hydraulic capacity of the intended drainage system to the expected runoff. Since the drainage system is aimed for urban area, a return period of 20 years is considered. ***Table 2.11*** was adopted from Norsk Vann (2012) also applied as a general practice in Oslo municipality in designing or renovating new water related facilities.

The concentration time or the travel-time is another important parameter to be calculated. It is defined as the time needed for a drop of rain to leave the catchment at the very down stream outlet. In principal, one will consider the longest flow time from the outermost point to the most down stream outlet.

Were the length of longest drainage pipe in the catchment is 650 meter. The recommended flow velocity into the pipes is ranged between 1.5-2 m/s.

The surface inflow time (t_t) to the closest drain is presumed between 3-7 min.

Using concentration time formula: $T_c = t_t + t_s$

Where the assumed surface inflow time t_t = 3 min

And $t_s = {}^{L}/_{V}$= (650 m) /(1.5 m/s) / (60 min/s) = 7 min

Total concentration time for Kjelsrud is (Tc) = 3 + 7 = 10 min

The IDF- curve: the representative rainfall event is based on the calculated concentration time. Where also return period is significant to decide which rainfall intensity to consider for the design storm.

The following rainfall intensity calculations were based on a method was described in Lindholm (2008). Where 20 years was considered as a return period and a design storm was lasted for 60 min with a 5 minutes time-interval.

The rainfall intensity data ***Table 6.1*** was extrapolated form IDF-20 years curve from Vestli gauging station. Where rainfall duration varied between 5-60 minutes. The rainfall intensity of both 40 and 50 minutes were not given but rather were interpolated.

Table 6.1: Rainfall intensity extrapolated from IDF-Curve of Vestli gauging station

Duration (min)	Rainfall intensity (l/s. ha)
10	229.8
20	143.5
30	110.3
40	92.0
50	80.1
60	72.1

Rainfall duration 10 minutes with respective intensity

$$(I_{10}) = 229.8 \text{ l/s ha}$$

Rainfall duration 20 minutes with respective intensity

$$(I_{20}) - (I_{10}) = ((143.5 * 20) – (229.8*10))/10 = 57.2 \text{ l/s ha}$$

Rainfall duration 30 minutes with respective intensity

$$(I_{30}) - (I_{20}) = ((110.3 * 30) – (143.5*20))/10 = 43.9 \text{ l/s ha}$$

Rainfall duration 40 minutes with respective intensity

$$(I_{40}) - (I_{30}) = ((92.0 * 40) – (110.3*30))/10 = 37.1 \text{ l/s ha}$$

Rainfall duration 50 minutes with respective intensity

$$(I_{50}) - (I_{40}) = ((80.1 * 50) – (92.0*40))/10 = 32.5 \text{ l/s ha}$$

Rainfall duration 60 minutes with respective intensity

$$(I_{60}) - (I_{50}) = ((72.1*60) - (80.1*50))/10 = 32.1 \text{ l/s ha}$$

This will give the following 5 minutes time -interval calculations ***Table 6.2*** to construct a symmetric hyetograph representing the design storm over the catchment in *Figure 6.2*.

Table 6.2: The calculated rainfall intensity (l/s.ha)

Duration after storm starts (min)	Intensity (l/s.ha)
00:05	32
00:10	33
00:15	37
00:20	44
00:25	57
00:30	230
00:35	230
00:40	57
00:45	44
00:50	37
00:55	33
01:00	32

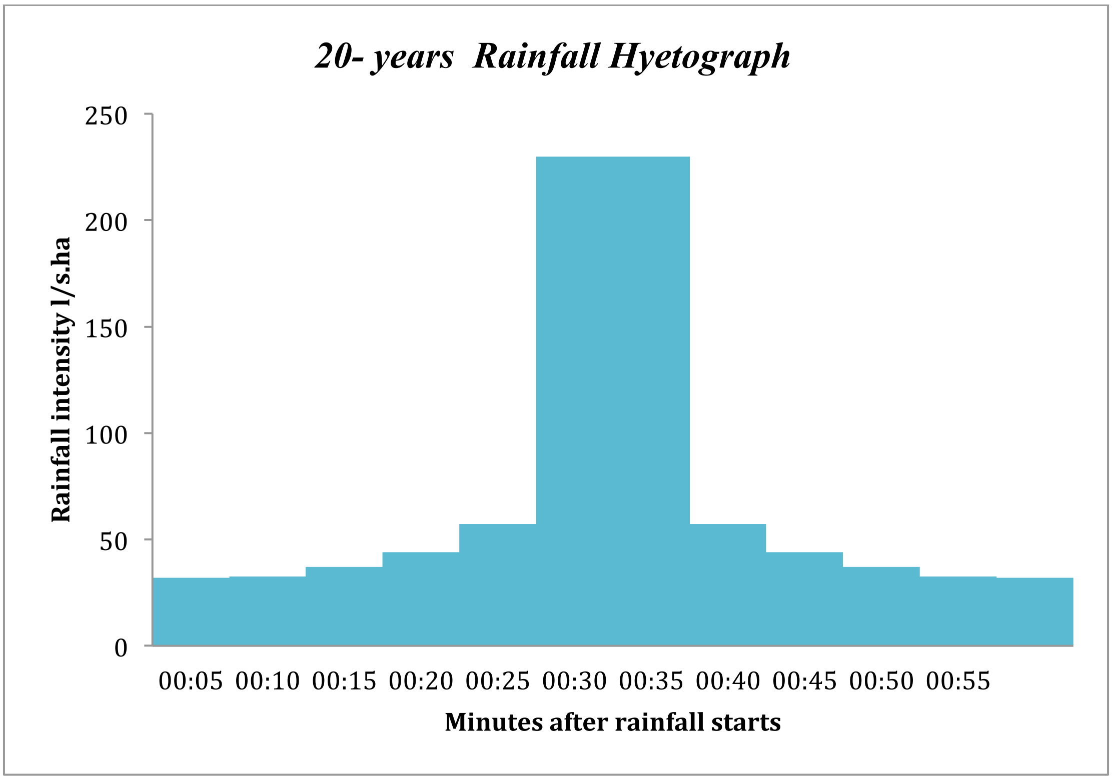


Figure 6.2: Rainfall Hyetograph for Kjelsrud

6.1.3 IMPACT OF CLIMATE CHANGE:

The impact of the climate change is significant when planning/ designing a new drainage system, also the climate change factor (C_f -value) is used here as a reference to explain the expected changes in the future precipitation patterns.

Following the municipality of Oslo guidelines and Oddvar Lindholm recommendations, this study is considering a long-term lifetime of 100 years for the drainage system. Further, considering an increase of about 50 % of today´s IDF-curves and a C_f –value of 1.5.

Table 6.3. Listing the rainfall intensity calculations with future climate considerations and C_f –value of 1.5 as an additional value.

Table 6.3: The calculated rainfall intensity with 1.5 Cf –value

Duration after storm starts (min)	Intensity (l/s. ha)
00:05	48
00:10	49
00:15	56
00:20	66
00:25	86
00:30	345
00:35	345
00:40	86
00:45	66
00:50	56
00:55	49
01:00	48

The time- interval calculations with C_f –value of 1.5 listed in ***Table 6.3*** gives a hyetograph see *Figure 6.3*.

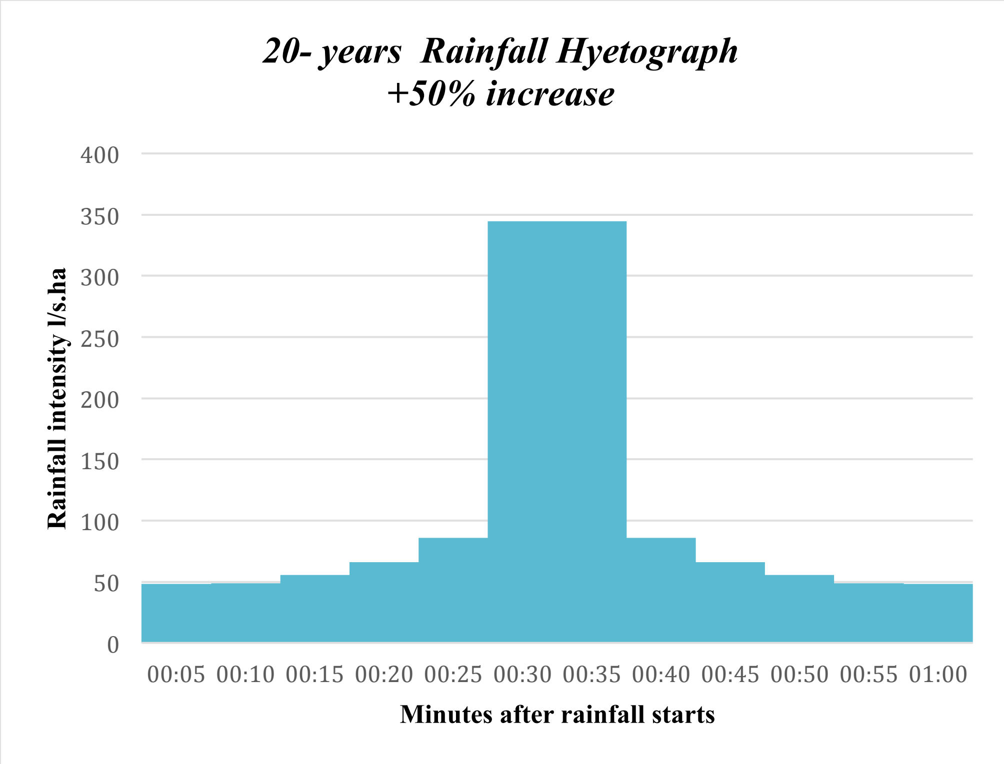


Figure 6.3: Rainfall hyetograph of 20 years return period and 60 min duration with 50% value increase

The comparison between two different hyetograph for the same rainfall event (20 years return period and 60 minutes duration), with and without climate change considerations is shown in *Figure 6.4*. Where the maximum rainfall intensity increased from 230 l/s.ha to 345 l/s.ha due to the climate change. This increase should be countered when the final plan for the drainage system is taking place.

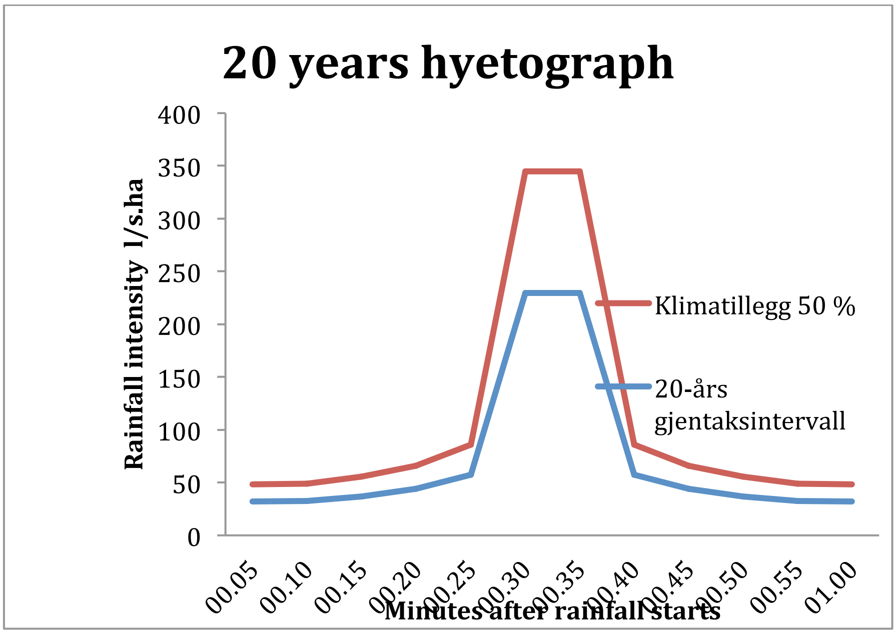


Figure 6.4: Comparison between 20 years rainfall intensity with and without climate change

Further, *Figure 6.5* gives a realistic image about the substantial impact of the climate change with respect to the future rainfall intensity. Where an increase of only 50 % in the future 20 years rainfall is almost 16 % more than todays 100 years return period.

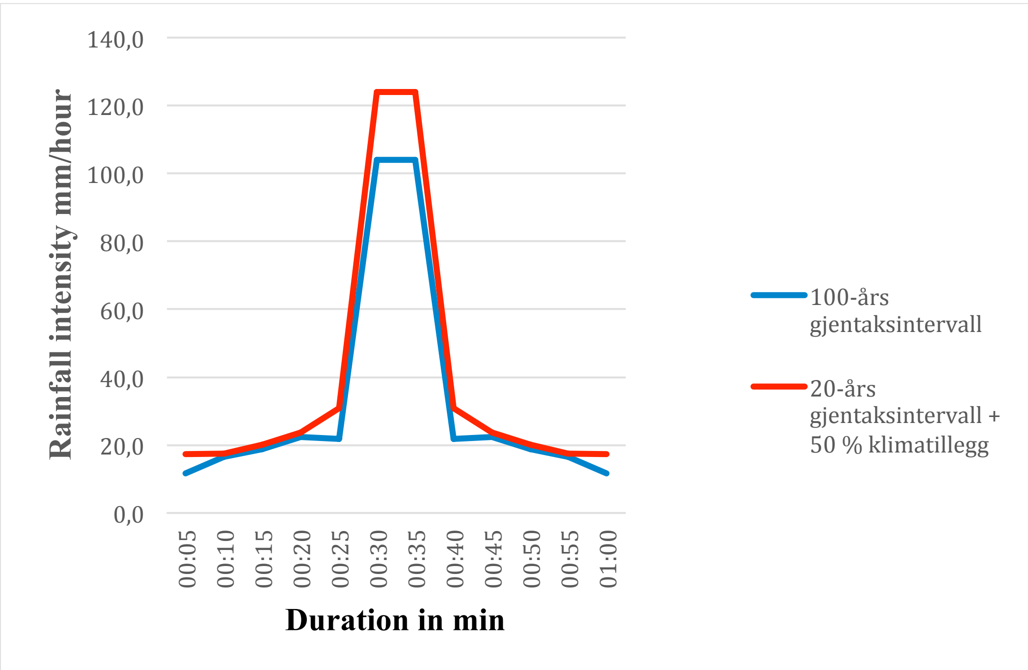


Figure 6.5: Todays 100- years rainfall is 16% less than the future 20-years rainfall with 50% increase

6.2 THE CATCHMENT RESPONSE

In order to estimate the catchment hydraulic response to the design storm, the runoff estimates are achieved based on manual calculations using the rational formula. Thus, the runoff peak flow is estimated for the whole catchment with and without climate change considerations.

Still, there are a number of parameters related to the catchment should be decided first. These parameters including the followings:

- Catchment area
- Time of concentration
- Return period
- Rain intensity based on IDF- curves
- Runoff coefficient
- Climate factor

Catchment area (A): the catchment total area is 38 ha. Where, the impervious surfaces about 22 ha, in which rooftops is 17 ha and roads is 5 ha, however this value is compromising 58 % of the total area comparing to 42% of pervious surfaces (see *Table 10.8* in Appendix A).

Table 6.4: The pervious and impervious proportions in the catchment

Surfaces type	Land-use	Area (m2)	Area (ha)
Impervious	Residential	167920	17
	Roads	49051	5
Total		216971	22
Pervious	Park and green areas	25021	3
	Ravine Forest	8980	1
Total		34001	3

The concentration time or the travel-time: since the concentration time for the catchment is 10 min is equal to the design storm duration. In another word, the flood event can be expected as the surface flow rate increases and the equilibrium between the storm duration and concentration time is not established.

The runoff coefficient: the φ-value for the catchment is equal to the imperviousness proportion in the catchment. This compromised a value of φ-value = 0.58

The rainfall intensity (I): based on the hydrograph for the catchment design storm, the rainfall intensity without climate change considerations is 230 l/s.ha while with climate change consideration is compromised 340 l/s.ha (see *Figure 6.2 &Figure 6.3*).

By using the rational formula, the runoff flow peak for the catchment as the following:

Without climate change considerations:

$$Q = \varphi \text{ X } A \text{ X } I \;= 0.58 \text{ X}38 \text{ ha X } 230 \text{ l/s.ha} = 5069 \text{ l/s}$$

However, the peak flow with climate change considerations:

$$Q = \varphi \text{ X } A \text{ X } I \text{ X } C_f = 0.58 \text{ X}38 \text{ ha X } 345 \text{ l/s.ha} \quad \text{X } 1.5 = 11406 \text{ l/s}$$

The calculated runoff peak flow compromised an amount of 5069 l/s or an amount of 11406 l/s. Both amounts are significant in the urban area and requiring a stormwater drainage system.

7. THE DESIGN: URBAN RUNOFF DRAINAGE SYSTEM

This thesis is suggesting two independent alternatives to collect and transport the stormwater runoff from the catchment through two independent drainage systems, where both of the alternatives are considering the catchment response to the design storm. Further, the two alternatives are expected to collect the runoff from the impervious surfaces (residential areas and roads) and routed it into pipes or channels for further transport to the closest recipient. There are three water bodies to be considered in the catchment, Alna River, and two other

7.1 THE MODEL ASSUMPTIONS:

In order to secure the quality of the runoff simulation data, there are a number of assumptions were considered:

1. Assuming rainfall data is varies over the time. Where the rainfall is synthetic and was extrapolated from historical rainfall records based on IDF-curves obtained from Vestli gauging station.
2. Assuming that the surfaces were snow and frost free to avoid including snowmelt in the calculations.
3. Assuming the soil characteristics in each subcatchmnet are ignored
4. Assuming time variations in rainfall and also runoff
5. Assuming that runoff started at zero, where groundwater inflow is neglected
6. Assuming period of plus degrees
7. Assuming that runoff quality is not of concern
8. Assuming the natural drainage flow to avoid using pumps
9. Assuming there is no need to calibrate the simulation model until after establishing the actual drainage system.

7.2 MODEL-SETUP

The hydrologic characteristics of the catchment is listed below, also were used as input parameters in SWMM (see ***Table 10.9*** in Appendix A):

<u>Subcatchmnet:</u> using SWMM implies subdividing the catchment into 42subcatchmnets (Sub), where each subcatchmnet was considered as an independent hydrologic unit and enjoyed own features. Still, these features were defined according to the respective land variations, topography, and the drains location. Hence, these features have influenced the generated runoff characteristics.

Both of the topography and drainage location were defined using the online mapping software at Norgeskart, which was also used to localize the catchment on the map.

Further, ***Table 7.1*** includes the different zones in the catchment. Where the catchment was divided into 4 different zones, where each zone included a number of subcatchmnets that connected to an outlet.

Table 7.1: The different zones within the catchment

Zone	Outlet nr.	**Elevation**	Subcatchmnet
A	1	**120**	Sub1-7, sub 16, sub38
B	2	**119**	Sub 33- 39, sub 40-41
C	3	**119**	Sub12-15, sub117-29
D	4	**120**	Sub 30-33

Runoff flow direction (s): *Figure 7.1* illustrates the runoff flow different direction. This figure was used as a starting point to estimate the ideal locations for the outlet. The current location for each of the outlets was based on the elevation profile for the catchment was obtained at Norgeskart.no.

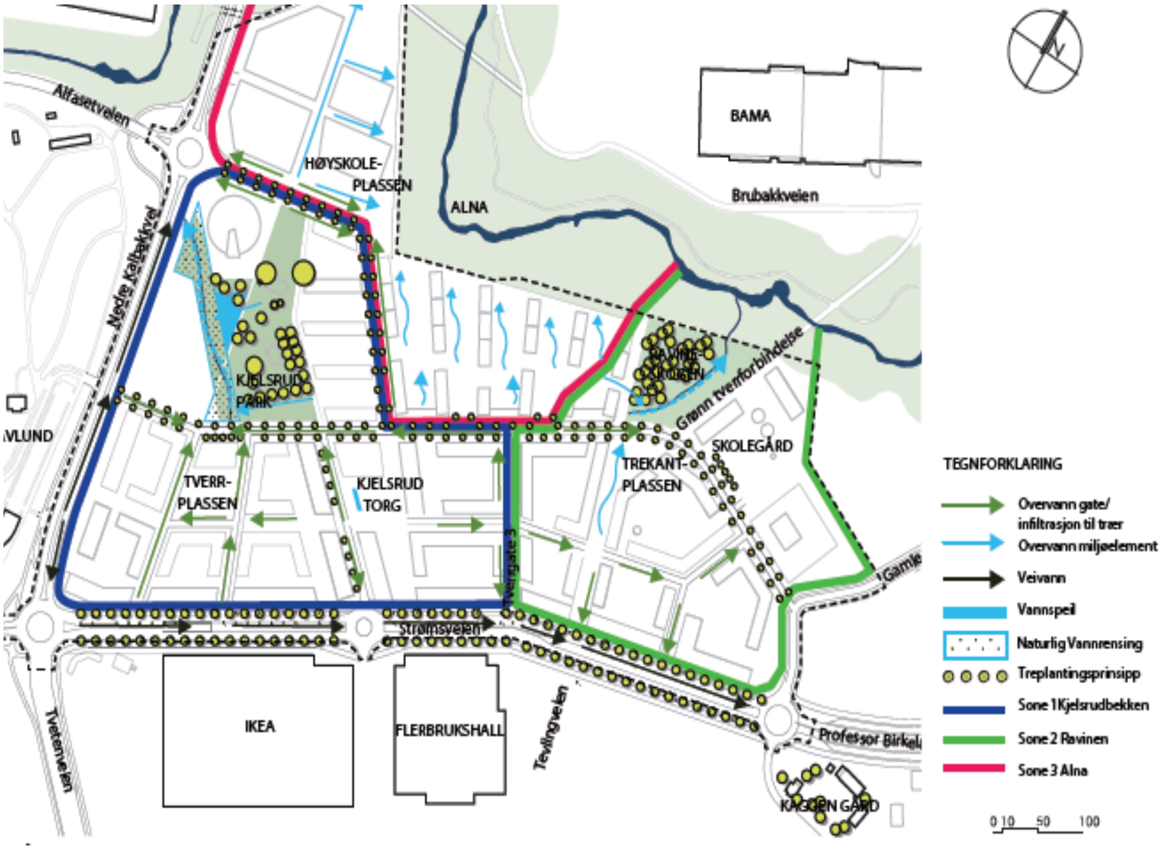


Figure 7.1: The stormwater natural drainage (ARC, 2013)[15]

Area: the area was defined according to the subcatchment boundary, and the area value was determined using CAD-drawings from the architecture company ARC (See *Figure 10.1* in Appendix A)

Design storm: the design storm compromised a synthetic rainfall event that lasted for 60 minutes associated with 20-years return period and applied evenly to the catchment. Each subcatchment is linked to the design storm, where the storm is represented by a time series object (Rain Gage) in the model, that is named as time series 20-years rainfall (TS20) with a 5 minutes time-interval and plotted in *Figure 6.2,* and another one named 20-years Rainfall + 50 % (TS+50%) plotted in *Figure 6.3*. The runoff rate and volume depends on design storm

[15] *For better resolution see Figure 10.3*

magnitude and distribution over the catchment, thereafter the runoff will be routed through the two different drainage systems.

Width: the subcatchment width was determined using the following formula

$$Width = \frac{\text{Subctahcmnt area}}{\text{length of the longest overland flow path}}$$

Slope: the catchment slope was determined using the following equation:

$$\%Slope = \frac{altitude\ 2 - altitude\ 1}{Length} * 100\%$$

The slope value is the same regardless the imperviousness of the surfaces, but it used to define the runoff flow direction and path.

Roughness coefficient: the *Manning's roughness coefficient n* for close conduit/ pipe and open channel was used to reflect the resistance that overland flow encountered when running on the surfaces. Different values were used for pervious and impervious surfaces (See ***Table 10.2*** & ***Table 10.3*** in Appendix A).

The imperviousness of the subcatchment: is a parameter in SWMM refers to the proportion in the subcatchment that drains directly to the stormwater transport system (pipes, channels, Swales…etc.). Hence, it represents the area that is used for rooftops, roads and streets and parking places. Also, the runoff coefficient (φ value) can be used also to represent the impervious proportion within the subcatchment. However, the pervious proportion is always assumed to have Zero for φ value. In this thesis, the imperviousness was visually estimated.

Present of impervious area with no depression storage (%zero-dep.): this parameter represents the immediate occurrence of runoff and before the depression storage is filled up. However, it is a changeable parameter according to roughness of subcatchmnet surface.

Infiltration method: the Horton model is used to compute the infiltration loss using the dynamic simulations.

Dynamic wave: is used as flow routing method to solve the Saint Venant equations for the entire transport system (see under 4.7.1). Hence, this method can simulate the flow under different conditions that can take place in the drainage system.

Table 7.2: The catchment properties

Parameter	Value
Catchment area	30 ha
Nr. of subcatchmnets	42
Width	4.5-76.4
% Slope	0.6
% Imperviousness	
Residential	50
Roads	100
Green areas - Park &forest area	2
Manning´s for overland flow on pervious surfaces	
Green area- Park & forest	0.15- 0.4
Manning´s for overland flow on impervious surfaces	
Residential	0.013
Roads	0.011
Depth of depression in mm	
Pervious surfaces	5- 7.6
Impervious surfaces	4
% Impervious area with no depression storage (% Zero-imperv.)	
Residential	50
Roads	100
Green areas - Park &forest area	2
Infiltration	Horton infiltration model

7.3 ALTERNATIVE 1: PIPE FLOW- BASED SYSTEM

The first alternative is a pipe flow –based system: where the runoff was collected, and then routed into the pipes under certain pressure to fill the whole cross-section area. It is an under-ground system, where also placement of the pipes considered the frost free zone and the natural flow by gravity. Further, SWMM was used to transform the rainfall into runoff and for further runoff simulation.

The proposed development for the catchment not including significant changes in the topography, this implies that roads and flow pathways will follow the natural slope. The catchment will drain through a drainage system that compromised a network of 27 conduits/pipes that are connected with 30 nodes/manholes to collect the runoff from the subcatchments, and then to convey the runoff to the downstream outlets. The principal of Alternative 1 is illustrated in *Figure 7.2 and 7.3*.

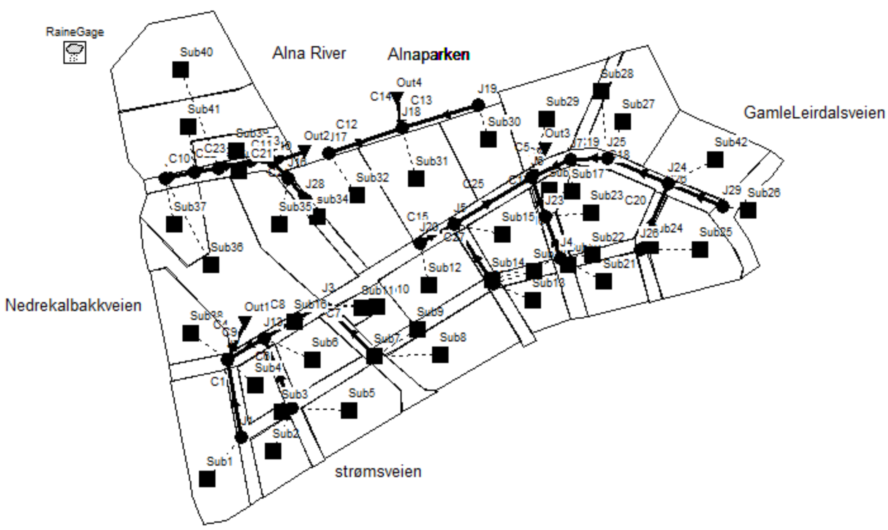


Figure 7.2: Alternative 1 the pipe flow-based drainage system

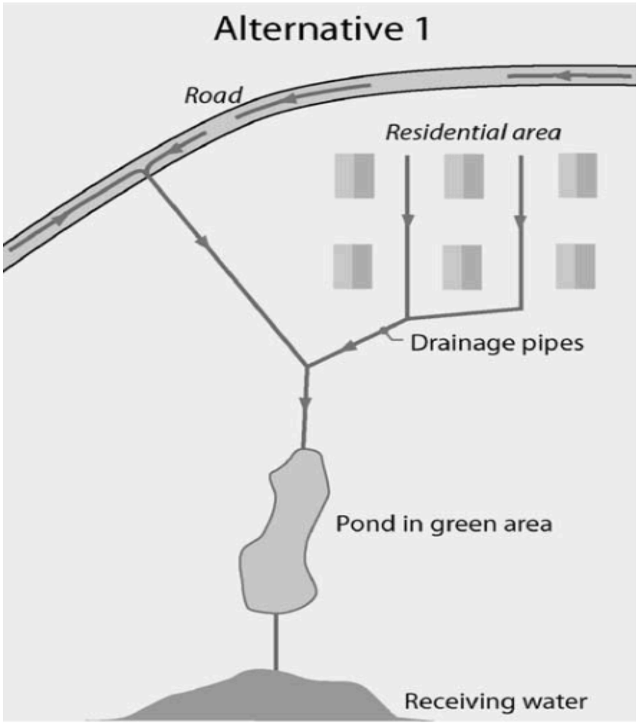
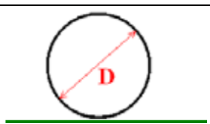


Figure 7.3: Illustration for pipe flow-based drainage system (adopted from (Åstebøl et al., 2004))

Table 7.3 includes the general properties for the drainage pipes/culverts. Where the 23 culverts were assigned to the drainage network and 4 were assigned to convey the runoff to the outlets.

Table 7.3 includes the general properties for drainage pipes/culverts

Type of shape	Cross section	Representing	Diameter (mm)	Manning´s roughness coefficient (n)
Circular		Pips/culvert	0.3-0.6	0.013

7.3.1 SIMULATION RESULTS: PIPE FLOW- BASED ALTERNATIVE

Figure 7.4 shows the hydrographs for each outlet for the design storm (20-years rainfall), where each hydrograph represent the total inflow at each of the outlets. However, *Figure 7.4* shows also four hydrographs for the same outlets for the design storm but with climate change considerations.

Based on *Figure 7.4 & Figure 7.5*, the runoff peak flow was sensitive to the changes in the rainfall pattern, where it compromised 900 l/s and 2050 l/s with and without climate change consideration respectively. Note the considerable increase in the peak discharge as the today's hydrographs increases only by1.5 (C_f-vlaue), where the more rainfall received the more runoff was generated.

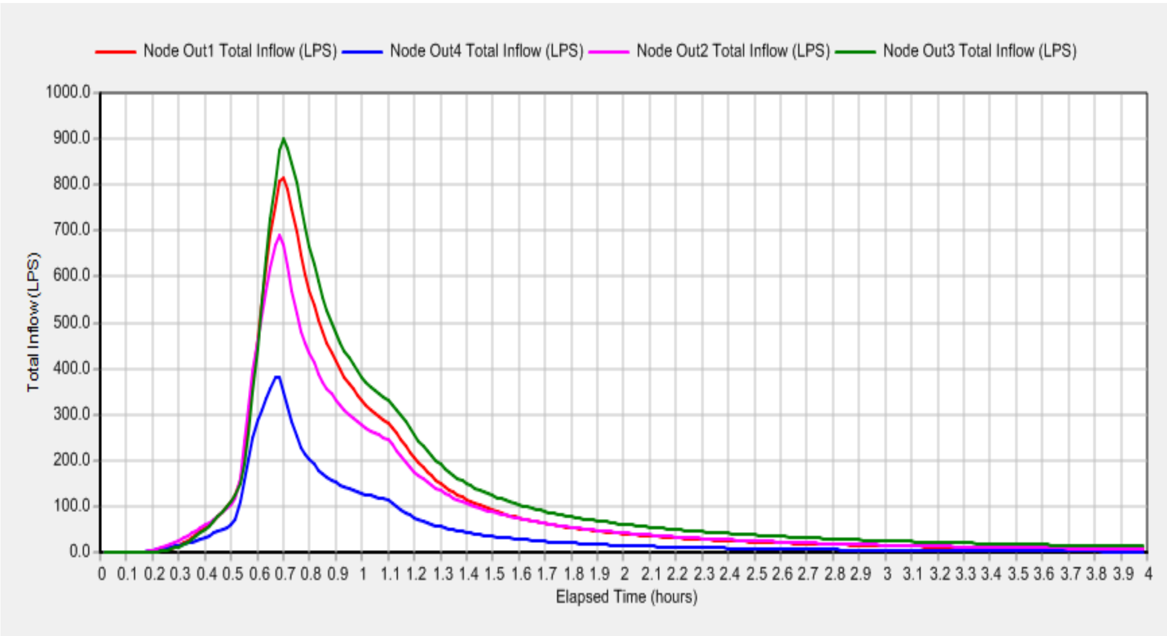


Figure 7.4: Runoff hydrographs for drainage system outlets

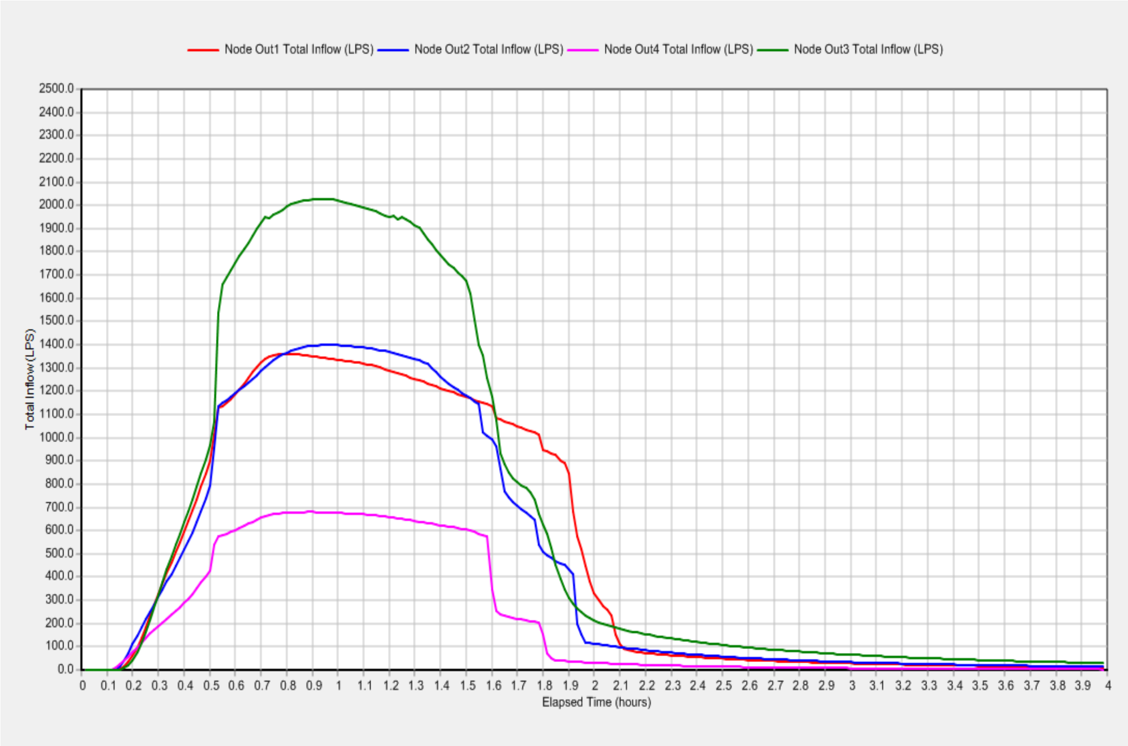


Figure 7.5: Runoff hydrographs for drainage system outlets with climate change considerations

Figure 7.6 illustrates the hydraulic response of nodes in zone 1 at 00:42 minutes. In case of design storm of current rainfall data, the designed capacity was able to meet the runoff inflow without significant risk for surcharge or flooding. *Figure 7.7* illustrates a map of study area where there was no flooding or surcharges atop the nodes in the entire drainage system.

However, *Figure 7.8* illustrates the hydraulic response of nodes in zone 1 at 00:42. The nodes experienced considerable surcharge as a response to the increase in the flow peak due to increase in rainfall intensity with 1.5 only. All nodes in the entire drainage system experienced the same as illustrated in *Figure 7.9*. This flooding occurred as the flow exceeded the designed capacity and the manholes surcharge was likely to occur. But, this drainage system was designed with a 100 m^2 ponded area around each junction/manhole, this will allow the ponding-on mechanism as the overflow can still routed back to the pipes when the capacity permits. Further, the impact of the climate change was also obvious in zone 2,3 and 4, where the simulations results show a considerable flooding a tope all the nodes in each of the zones (see *Figure 10.4 &10.5*/ Appendix B).

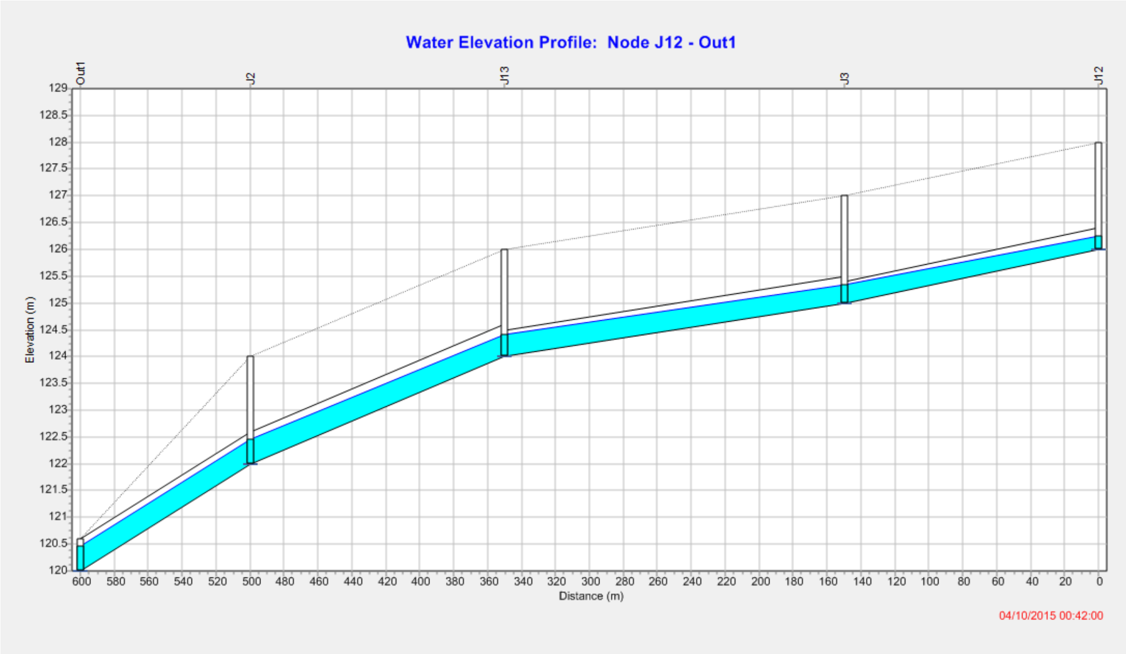


Figure 7.6: Profile plot for zone 1response to the design storm

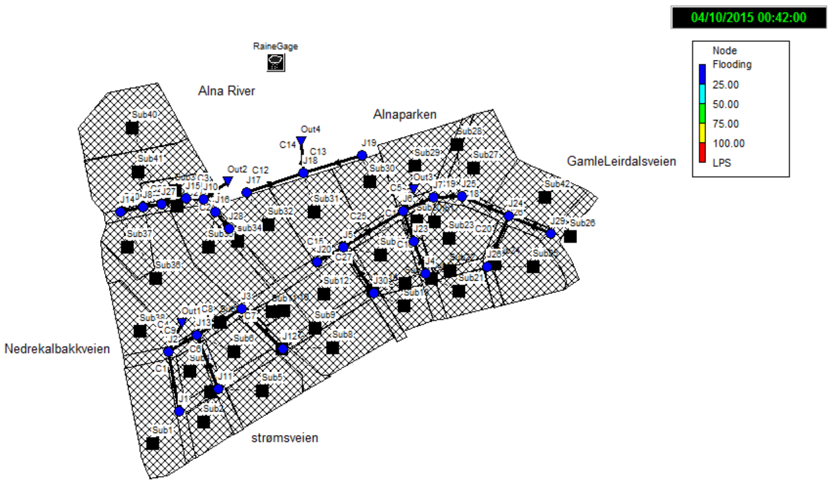


Figure 7.7: Study map with No flooding atop the nodes under the design storm

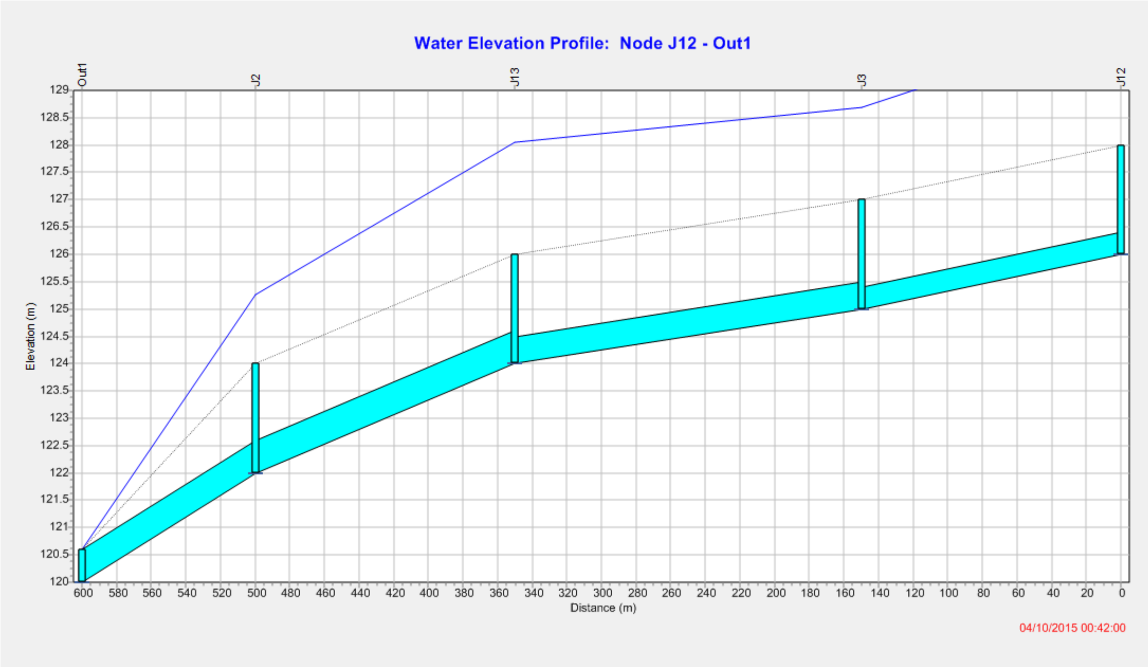


Figure 7. 8: Profile plot for zone 1 response to the design storm+ 50% climate change

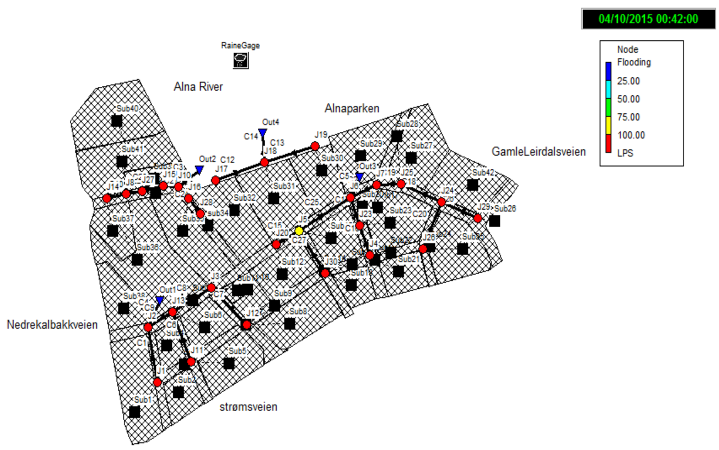


Figure 7.9: Study map with flooding atop all nodes the design storm + 50 % climate change

7.4 ALTERNATIVE 2: SURFACE DRAINAGE OPEN CHANNELS- BASED SYSTEM

Alternative 2- is an open channel -based system: where the runoff was collected from impervious surfaces (residential areas, parking and roads) to fill up the different conveyance links such as gutters& Swales. Hence, these links were able to control the flow between the different nodes in the system by natural gravity, also with no external pressure rather than the atmospheric pressure. However, the nodes were defining the drainage system elevation and also the hydraulic variations at each link end. The flow from each zone was conveyed through the respective outlet.

SWMM used the dynamic wave routing method also in this alternative, where the hydraulic routing expresses all inflows at each conduit and the conveyance of these flows to the respective outfall. The model of the drainage system consisted of same number of links and nodes as in Alternative1; it was 27 links in form of Gutter ad Swales and 30 nodes. While the links are to some extend oversized to avoid flooding within the site and to ensure that all the generated runoff is conveyed to the downstream outlets. The principal of Alternative 2 is illustrated in *Figure 7.10*.

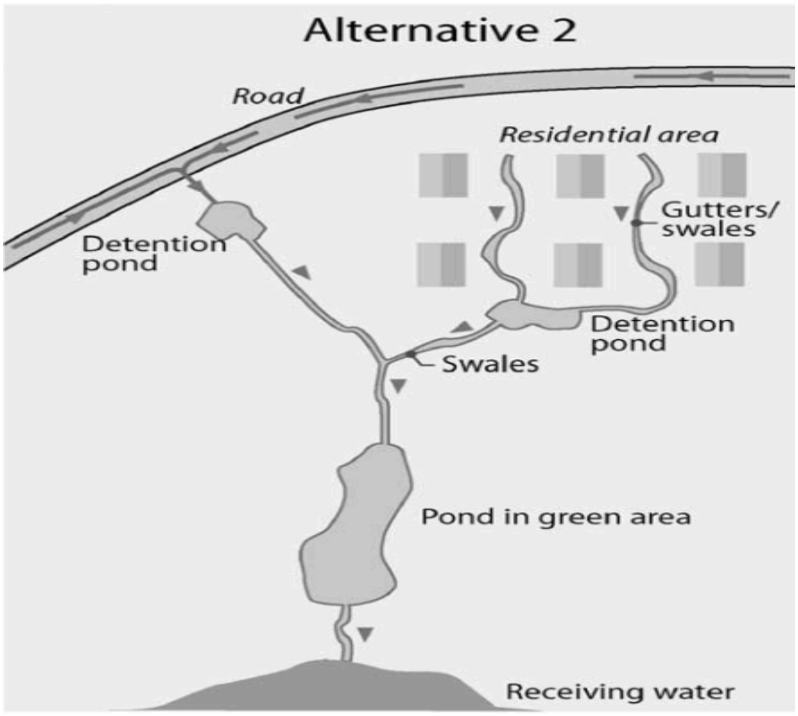


Figure 7.10: Illustration for open-channel flow drainage system (adopted from (Åstebøl et al., 2004)

7.4.1 STORMWATER MITIGATION STRUCURES

The selection of the stormwater mitigation structures was based on the catchment characteristics and planed land use, as well as the intended performance form these structures. Hence, the performance of these structures is of high concern for a short and long term, therefore the selection aimed also to meet Oslo Municipality criteria in selecting multifunctional structures in terms of meeting the climate change, utilizing the stormwater as a resource and improving the runoff quality.

The selection was limited to few stormwater mitigation structures, where the Green-roof was selected for runoff inflow control and to improve the runoff quality. In addition, a number of detention ponds were also considered for their flood control function also for runoff storage and treatment purposes also for their recreational and environmental values. In addition, Swales and open channels were also considered as runoff conveyances and for infiltration purposes (for more details see under 2.6).

Swales and Cutters: Swales and Cutters replaced the pipes and culverts in Alternative 1, where these multifunctional structures used as conveyances and for detention and infiltration purposes. *Figure 7.11* illustrates the main features of these structures with small slope and wide shallow channels to ensure slow and continuous flow and to ensure a natural appearance.

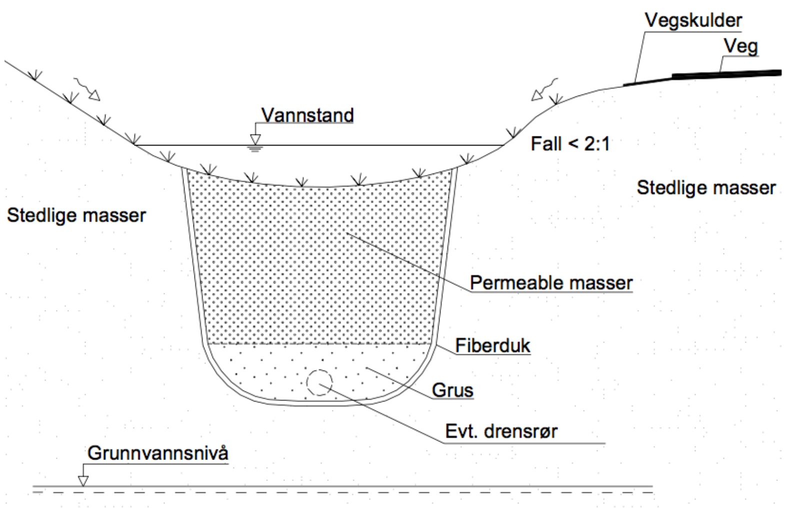


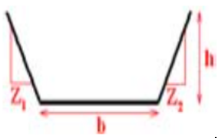
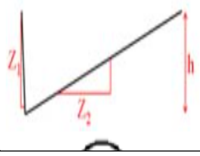
Figure 7.11: Illustration of general design of Swales /channels (BergenMunicipality, 2005)

Table 7.5 listed the cross -sectional shapes of the different conveyances were used in this alternative. The different values of both Swales and gutter were based on the general design practice at Bergen Municipality.

The Swale properties: side slopes (Z1&Z2 (2:1) i.e. horizontal: vertical), roughness coefficient (n), bottom width (b) and height (h).

The gutter/channel properties: the cross- slopes (Z1 & Z2), roughness coefficient (n), bottom width (b) and height (h).

Table 7.4: General properties for Swales and Gutter (BergenMunicipality, 2005)

Type of shape	Cross section	Representing	Z1	Z2	b(m)	h(m)	n**
Trapezoidal		Swales	2	2	1	1	0.05
Trapezoidal		Gutter	2	2	0.0001*	1	0.016

* SWMM do not accept a slope zero

The Manning´s *n* for open channels listed in ***Table 10.4*. Where *n* varies according to the used material (Rossman, 2010)

Detention pond: are used in this alternative to encourage runoff infiltration and sedimentation, also for flood control, storage and recreational purposes. *Figure 7.12* illustrates the main features of detention pond design, where the pond depth should be between 1-1.5 m in draught period and 2-2.5 m in periods with more rainfall. The required detention volume is decided by the highest difference between the inlet and outlet volume, where rainfall-envelop method is used to determine this volume (COWI, 2007).

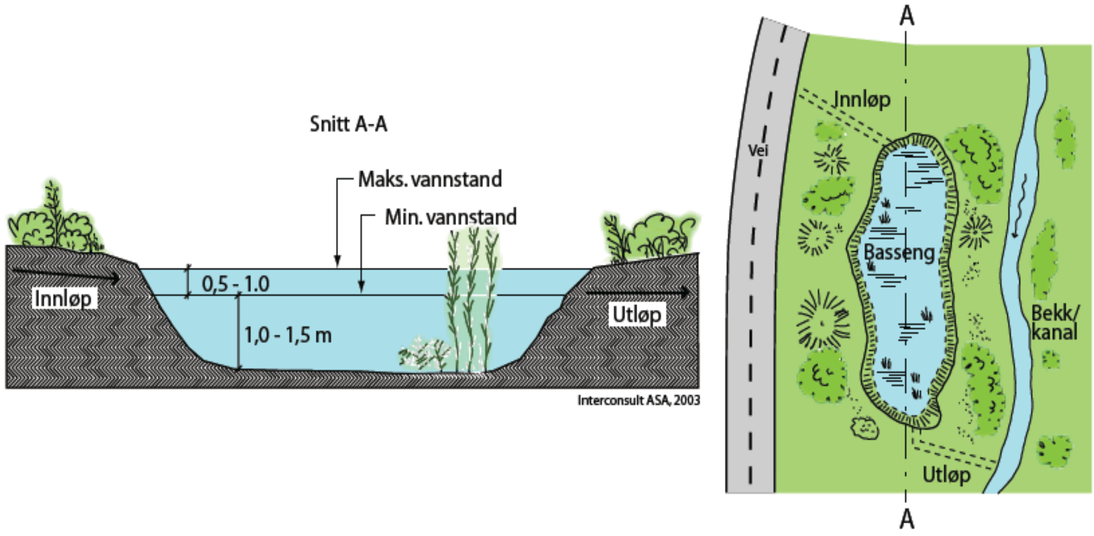


Figure 7.12: Illustration of general design of detention basin (COWI, 2007)

Green roofs: are multi-functional structures, as the structure able to control the inflow by retaining the runoff and consequently reducing the flood peak. Also the roofs functioning as filter medium and able to improve the water quality. *Figure 7.13* illustrates a typical design of a green-roof.

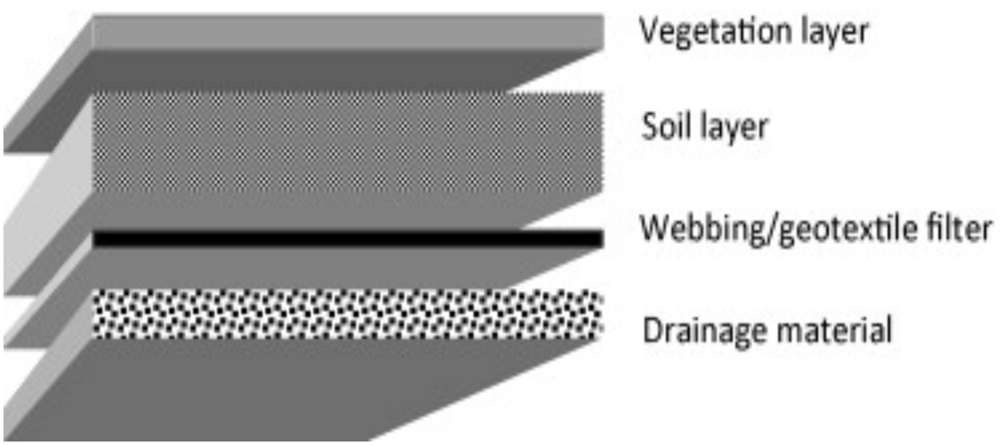


Figure 7.13: Typical cross-section of a green-roof (Berndtsson, 2009)

7.4.2 RESULTS FROM ALTERNATIVE 2

The results from Alternative 2 are limited to results from detention volume and green-roof calculations, also from the simulation results of Swales and green-roofs using SWMM.

7.4.2.1 SIZING OF DETENTION VOLUME

The potential detention volume was determined by using the rainfall-envelop method, where the balance between the inlet and the outlet volume is also of concern to avoid failure in the structure. The rainfall -envelope method depends on the quantity of the generated runoff. This method was used estimate the potential detention volume by the calculating the balance between the inlet and outlet flow in the planned detention pond (s).

The NWA (2012) recommended three different alternatives to determine the detention volume, of which alternative A was considered in this thesis (*see under 3.4.1*). Further, each municipality decided the limits for runoff discharge to avoid loading the drainage system with excessive amounts of runoff. Following Rambøll (2015) recommendations for urban expansion where the runoff discharge from detention structures must not exceed 10l/s.ha (Ramboll, 2015).

The maximum allowed discharge is summarised with the following formula:

$$Q_{oulet} = Q_{discharge} \times total\ area$$

Where:

Q_{outlet}: Constant outlet runoff flow (l/s)

$Q_{discharge}$: Allowed runoff flow discharge per unit area, which is equal to 10 l/s.ha

Total area: Catchment area (ha)

Then the maximum allowed runoff inflow rate from the catchment is the following:

$$Q_{outletv\ ol} = \frac{10\text{l}}{\text{s. ha}} \times 30\ ha \ = 300\ l/s$$

a) <u>The detention volume calculations without climate change considerations</u>

The allowed runoff discharge volume is express as a function of time with the following formula:

$$V_{outlet} = \frac{Q_{\ oulet\ \mathrm{vol}}\ X\ t\ X\ 60}{1000}$$

Where:

V_{outlet}: Calculated outlet volume (m^3) from the detention structure (s) after rainfall event

$Q_{oulet\ vol}$: Maximum allowed discharge from the catchment (l/s) to the closest recipient

t: Rainfall duration (min)

Table 7.5: The outlet volume calculations for 20-years rainfall without climate change considerations

Rainfall duration (t)(min)	Runoff discharge ($Q_{out.vol}$) (l/s)	Rainfall intensity (I) l/s	Outlet discharge volume (V_{outlet}) (m3)
10	300	230	180
20	300	145	360
30	300	110	540
40	300	92	720
50	300	79	900
60	300	72	1080
70	300	67	1260
80	300	63	1440
90	300	52	1620

The inlet volume of the detention structure is calculated using the following formula:

$$V_{inlet} = \frac{Q_{inlet\ vol}.X\ t\ X\ 60}{1000}$$

V_{inlet} : Calculated inlet volume (m^3) from the catchment to the detention structure

t: Rainfall duration time (min)

$$Q_{inlet} = A\varphi I$$

$Q_{inlet\ vol.}$: Runoff flow from the catchment to detention structure

A: Area of the catchment (ha)= 30 ha

φ: Runoff Coefficient = 0.7

I: Rainfall intensity (l/s.ha)

Following the NWA (2012) recommendations, the runoff coefficient (ϕ- value) for urban areas can ranged between 0.7-0.9. In this thesis ϕ- value= 0.7

Table 7.6: Calculated inlet volume (m^3) from the catchment to the detention structure

Rainfall duration (t) min	Rainfall intensity (I) l/s.ha	Runoff from the catchment $Q_{inlet.vol}$ l/s	Inlet from the catchment (V_{inlet}) m3
10	230	4830	2898
20	145	3045	3654
30	110	2310	4158
40	92	1932	4637
50	79	1659	4977
60	72	1512	5443
70	67	1407	5909
80	63	1323	6350
90	52	1092	5897

The calculations for potential detention volume is estimated using the following formula and summarised in ***Table 7.7***:

$$V_{reten.vol} = V_{inlet} - V_{outlet}$$

Where:

$V_{deten.vol}$: detention volume (m^3)

V_{inlet}: Inlet volume into the pond (m^3)

V_{oulet}: Outlet (discharge) volume from the pond (m^3)

Table 7.7: The calculated expected detention volume in m^3

Rainfall duration (t) min	Inlet from the catchment (V_{inlet}) m3	Outlet discharge volume (V_{outlet}) (m3)	Detention volume (m3)
10	2898	180	2718
20	3654	360	3294
30	4158	540	3618
40	4637	720	3917
50	4977	900	4077
60	5443	1080	4363
70	5909	1260	4649
80	6350	1440	4910

Figure 7.14 explains the calculated Rainfall -Envelop for the catchment, where after 80 minutes since the rainfall event started, the maximum detention volume was estimated of (Vmax.) = 4910 m^3.

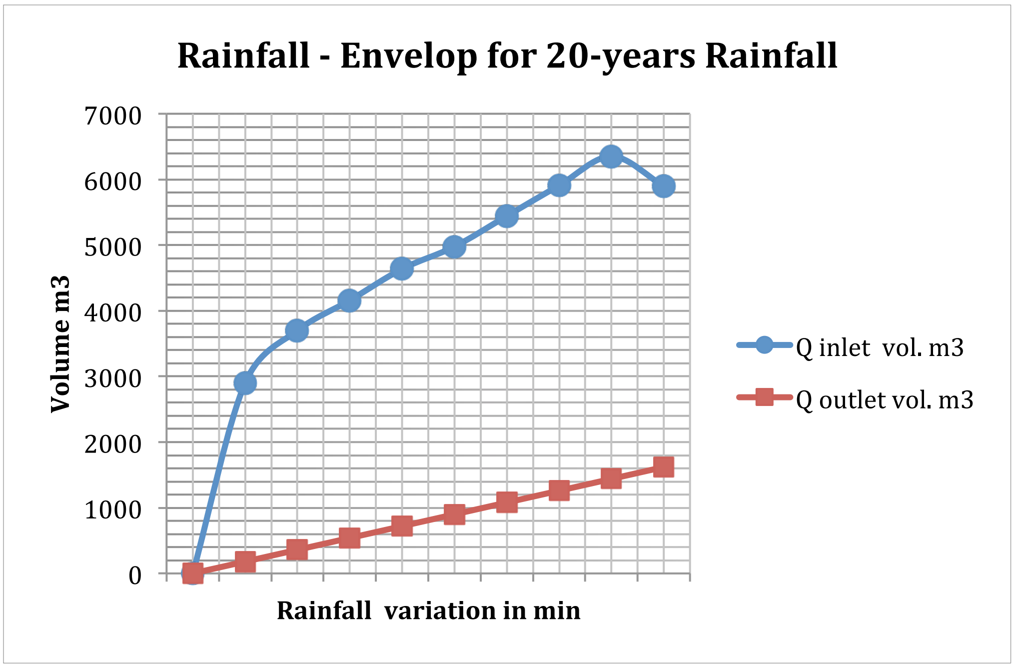


Figure 7.14: Calculated Rainfall –Envelop for the catchment without climate change

b) <u>The detention volume calculations with climate change considerations</u>

The expected increase of the precipitation pattern with only 50% of todays IDF-curves, in another word (C_f -value) of about 1.5, will be significant in Rainfall-Envelop calculations.

Table 7.8 indicates the significant increase in the inlet volume when considering the C_f –value in the calculations. Still, the outlet volume should be adjusted also to ensure the flow and volume balance in the detention structure.

Table 7.8: The calculated expected detention volume (m^3) taking the climate change into consideration

Rainfall duration (t) min	Outlet Discharge volume (V_{outlet}) m^3	Inlet Volume + 50% (V_{inlet}) m^3	Retention volume ($V_{deten.vol}$) m^3
10	180	4347	4167
20	360	5481	5121
30	540	237	5697
40	720	6955	6235
50	900	7466	6566
60	1080	8165	7085
70	1260	8864	7604
80	1440	9526	8086
90	1620	8845	7225

Figure 7.15 explains the calculated Rainfall -Envelop for the catchment with 50% climate change increase, where after 80 minutes since the rainfall event started, the maximum detention volume (Vmax) = 8086 m^3. This implies to adjust the capacity of the detention structure also with 50% of current capacity.

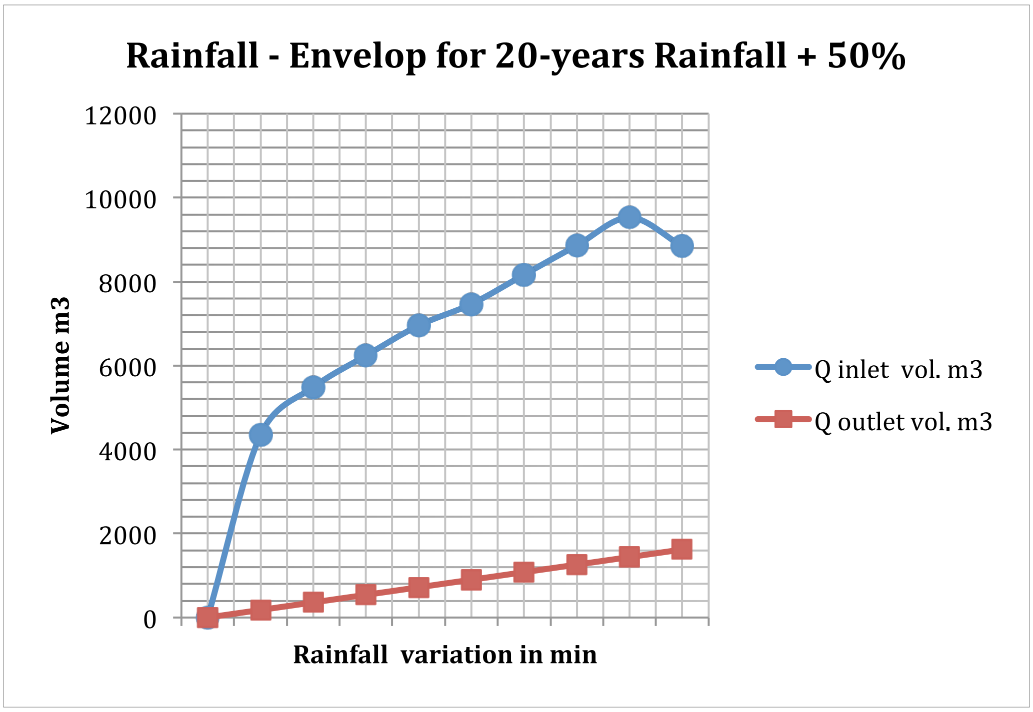


Figure 7.15: Calculated Rainfall –Envelop for the catchment with 50 % climate change

7.4.2.2 GREEN ROOF CALCULATIONS:

Following the green-roof research, this practice is able to reduce the runoff intensity of about 40-50% and thereafter reduce the flood peak with the same amount (Braskerud, 2014).

Following to Oddvar Lindholm recommendations, since the design storm is assumed as a single event with frost -free surface and warmer temperature, thereafter the green-roof can retain only 6mm of this rainfall event.

Based on documents were obtained from Arc, the total area of pervious and impervious surfaces is compromised 38 ha about 380000 m^2. Further, the total area of the impervious surfaces only is compromised 216971 m^2; this included roads and rooftops.

Following to Oddvar Lindholm recommendations, green-roof is able to retain up to 6mm of a single rainfall event, this makes 0.006 m

Total area of rooftops = 167 920 m^2

The total retained runoff from green-rooftops = 167920 m^2 X 0.006 m = 1007.52 m^3

7.4.2.3 DETENTION POND(S) AND GREEN ROOF – SUMMARY

Both of the Green-roof and the detention structure are aimed for runoff mitigation also to meet requirements of changes in the rainfall pattern. However, these measures are considered in this thesis to reduce the flood peak in both the drainage system and the recipient. *Table 7.9* summarising the runoff mitigations performance in detaining the generated runoff.

However, the calculated potential detention volume is assumed by three different detention ponds, each has a detention capacity of 1637 m^3 and a minimum depth of 1m.

Table 7.9:Summary of runoff mitigation

Structure	Detaining volume in m^3
Green- roof	1007.52
Retention structure (s)	4910
Retention structure (s) + 50% climate change	80086

7.5 ECONOMIC COST FOR THE DRAINAGE SYSTEM ALTERNATIVES

The prices for Alternative 1 & 2 were based on Roar Finsrud estimations in 2012 for wastewater conduits. However, with contact with Finsrud he recommended to increase the 2012 prices by 15% to meet the price change in 2015.

Alternative 2 consisted of several stormwater mitigations of which green-roof, detention pond and Swales, but due to time limitations only prices for green-roof and detention ponds were obtained. The detention pond prices are based on Finserud prices for 2012 for moulded basins, while the green-roof prices were obtained through communication with Jostein Sundby at Vital Vekst AS.

7.5.1 THE ECONOMIC COST FOR ALTERNATIVE 1

The pipe-based drainage system consisted of 27 pipes of circular type and diameter varied between 0.3-0.8 m, also the length varied from 25-200 m. *Table 7.10* includes prices in NOK per m. Note in 2012 the total cost of the conduits was 3 744 855 NOK while in 2015 the price increased to 4 306 583 NOKwith about 561 728 NOK. However, prices included the foundations and refilling ballasted bu limited for rural and easy to dig -into areas, also were (see *Table 10.10* / Appendix A)

Table 7.10: The economic cost of wastewater links (adapted from (Finsrud, 2015))

Conduit	Max depth Diameter (m)	Length (m)	Price NOK (2012)	Price increase 15%	Total price NOK (2015)
1	0,30	150	176 550	26 483	203 033
2	0,60	100	151 400	22 710	174 110
3	0,60	100	151 400	22 710	174 110
4	0,60	100	151 400	22 710	174 110
5	0,80	150	279 000	41 850	320 850
6	0,40	150	176 550	26 483	203 033
7	0,40	150	127 500	19 125	146 625
8	0,50	200	235 400	35 310	270 710
9	0,60	150	227 100	34 065	261 165
10	0,40	50	42 500	6 375	48 875
11	0,60	70	105 980	15 897	121 877
12	0,40	100	117 700	17 655	135 355
13	0,30	110	93 500	14 025	107 525

14	0,40	100	117 700	17 655	135 355
15	0,30	50	93 000	13 950	106 950
16	0,40	100	85 000	12 750	97 750
17	0,40	100	85 000	12 750	97 750
18	0,50	150	279 000	41 850	320 850
19	0,60	100	151 400	22 710	174 110
20	0,50	150	176 550	26 483	203 033
21	0,40	50	93 000	13 950	106 950
22	0,40	25	29 425	4 414	33 839
23	0,60	50	75 700	11 355	87 055
24	0,30	50	42 500	6 375	48 875
25	0,50	150	176 550	26 483	203 033
26	0,50	150	176 550	26 483	203 033
27	0,30	150	127 500	19 125	146 625
Total		2955,00	3 744 855	561 728	4 306 583

7.5.2 THE ECONOMIC COST FOR ALTERNATIVE 1

Price for Detention pond (s): *Table 7.11* listed the prices of three detention ponds proposed in Alternative 2. Following Finsrud recommendations, the prices were also subjected to increase by 15% comparing to prices in 2012. The price unit is NOK per m^3. Each of the three suggested ponds has a detention capacity of about 1637 m^3, in case the prices refers to the pond detention capacity volume in m^3, then this compromise a cost of 2562346 NOK for the three ponds.

Table 7.11: Price of detention pond (adapted from (Finsrud, 2015))

Pond no.	Volume m3	Total price NOK in 2012	Price increase by 15%	Total Price NOK in 2015	Price /pond
Pond 1	1637	4194560	629184	4823744	4823744
Pond 2	1637	4194560	629184	4823744	4823744
Pond 3	1637	4194560	629184	4823744	4823744
Total price for 3 ponds					14 471 232

Price for Green-roof:

The green-roof prices depend on the type of green-roof. *Table 7.12* listing the green-roof prices inclusive VAT and lifting up to the rooftop.

Table 7.12 Green-roof prices (AS. and Sundby, 2015)

Green-roof type	Thickness (mm)	Price NOK /m2	Comment
Extensive		300	Incl. growth medium (fleece 800-1200 gr)
		350	Incl. substrate as growth medium
Semi intensive	60-100	500-1000	
Intensive		Not available	

Considering the prices listed in *Table 7.12*, the catchment rooftop area of 167920 m^2 will compromise a considerable economic cost as listed in *Table 7.13*.

Total rooftop area m^2	Total pice NOK Extensive green-roof	Total price NOK Semi-intensive green-roof
167920	50 376 000	83 960 000

Summary cost for Alternative 2:

The cost for Alternative 2 is limited to the three detention ponds and the green-roof.

Table 7.13: Total cost for Alternative 2

Total cost for three detention bonds	14 471 232 NOK for 1637 m^3 detention capacity for each

Total cost for green-roof	50 376 000 NOK assumed the extensive type
Total cost for Alt.2	64 847 232 NOK

7.6 DISCUSSION

The representative design storm of 20-years rainfall event that was lasted for 60 minutes was applied to the contributing area then was converted into runoff. The runoff quantities were estimated using the rational formula under both current rainfall conditions and with considerations to changes in these conditions due to the climate change that was represented with 1.5 C_f –value.

Based on the rational formula calculations, the catchment was able to generate considerable quantities of runoff due to urbanization and the increase of the impervious surfaces. This was estimated by 5069 l/s under the current rainfall conditions and with 11406 l/s when the rainfall conditions changed. However, these quantities are of concern in the urban areas due to the high risk of flooding and manholes surcharge. To avoid these problems two drainage systems were suggested, both of the systems show different hydrologic performance in handling the generated runoff under the different rainfall conditions.

The first alternative was a pipe flow –based alternative (Alternative 1). This alternative was able to drain the runoff successfully under the current rainfall conditions through a network of 27 pipes and 30 manholes. The runoff was routed into the pipes without significant risk for flooding. However, considering the increase in the rainfall due to the climate change the same system was not able to adapt with this increase. The system shows high sensitivity towards the change in the runoff quantities and the risk for flooding was high. All manholes in the system responded with a significant surcharge at 00:42 minutes after the storm has started.

The second alternative was the surface drainage open channel- based alternative (Alternative2). This system considered runoff mitigation structures includes Swales/ channels, green-roofs and detention ponds. The selection of these structures was due to their multifunctionality and compatibility one to another. The hydrological performance of these structures was estimated based on manual calculations.

The green-roof structure used to control runoff inflow also to improve the water quality. This structure was able to retain up to 6 mm of the total received rainfall under the design storm. In return, this compromised 1007.52 m^3 from the rainfall regardless the rainfall conditions. In return the green-roof will be able to reduce considerably the amounts of the generated runoff based on the rational formula.

The three detention ponds were used for flood control purposes in addition to other functions. Using the rainfall-envelop method, the three ponds in total are able to detain a max volume (Vmax) 4910 m^3 at 80 minutes after the rainfall has started. However this amount was almost doubled to 8086 m^3 as the rainfall conditions changed also at 80 minutes since the rainfall has started. The different stormwater mitigations in the system are expected to adjust to the change in the rainfall pattern and the increase of the impervious surfaces.

The total cost of the first alternative was estimated using Finsrud tables for wastewater conduits. The cost compromised 4 306 583 NOK. The total cost for the second alternative was estimated for both the total green-roof and three detention ponds. The green-roof economic cost was estimated using data from Vital Vekst As. The green-roof is expected to cover the entire rooftops (167920 m^2) in the catchment at a cost of 50 376 000 NOK for the extensive green-roof and 83 960 000 NOK for the semi-intensive green-roof. However the prices for the intensive green-roof were not available. The only available prices for the detention ponds were based on prices provided by Finsrud for moulded basins. The prices were based on each pond maximum detention capacity (Vmax) of 1637 m^3, which compromised a cost of 4 823744 NOK/ pond. The total cost for the three ponds was 14 471 232. The total minimum cost for alternative 2 was estimated of about 64 847 232 NOK.

8. CONCLUSION

The expected change in the future rainfall patterns and the need for more urban areas are challenging the VAV at Oslo Municipality. In this thesis these challenges were addressed with respect to the urbanisation plan in Kjelsrud in Oslo. Two drainage systems were proposed to drain the generated runoff from the impervious surfaces in the catchment. The hydrologic performance of the pipe-based systems was simulated using SWMM. Under the current rainfall conditions the system responded with no significant problems and was able to drain the generated runoff. However, the system was vulnerable and was not able to adjust to the increase in the rainfall due to the climate change, flooding and surcharges atop the nodes were considerable in the entire system.

A number of stormwater mitigations were introduced in the second alternative the surface drainage open channel- based system. The hydrological performance of the suggested mitigations was evaluated based on manual calculations. The results from the manual calculations gave an approximate image about the hydrologic performance for the green-roof and the three detention ponds under the design storm conditions. These stormwater mitigations show tolerance towards the increase in the rainfall. The detention ponds were able to adapt with the change in the rainfall by increasing the detention volume accordingly. The green-roof also was able to retain considerable amounts of rainfall regardless the rainfall conditions.

However the economic cost of the first alternative was much less than the second alternative considering prices in 2015.

SWMM simulations results for alternative 1, considered valuable and provided with an idea about the drainage system response to the current rainfall patterns and the projected changes. Further, one can consider SWMM to simulate the different stormwater mitigations also comparing the results from the two alternatives based on SWMM simulations for both. Hence evaluating the two alternatives on equal bases.

Due to estimated hydrological performance and the economic cost for the two alternatives, one can investigate further a combination of the two alternatives to achieve an optimal drainage system in terms of hydrologic performance and cost effectiveness.

9. REFERENCES

AKAN, A. O. 2002. Modified rational method for sizing infiltration structures. *Canadian Journal of Civil Engineering* 539-542.

ARC 2013. Veilende Plan for det Offentlige Rom (VPOR-Kjelsrud). Trondheim.

AS., V. V. & SUNDBY, J. 2015. *RE: Economic prices for Green-roof* Type to AMIN, F.

BARTON, M. 2010. *Technical memorandum* [Online]. Available: http://your.kingcounty.gov/dnrp/library/wastewater/wtd/construction/CSOBeach/TechDocs/FacilityPlans/NBeach/Draft/AppB_NB.pdf[Accessed 25.04 2015].

BERGENMUNICIPALITY. 2005. *Retningslinjer for overvannshåndtering I Bergen Kommune* [Online]. Bergen Bergen Kommune [Accessed 23.08 2014].

BERNDTSSON, J. 2009. Green roof performance towards management of runoff water quantity and quality: A review. *Ecological engineering* 36**,** 251-360.

BJERKHOLT, J. 2012. Avrenning. *Lecture notes THT261.* Ås: Institutt of matmatiske Realfag og Teknologi (IMT) ved Norges miljø-og biovitenskapelige universitet (NMBU).

BRASKERUD, B., ET. AL 2014. *Grønne vegetasjondekkede tak for flomdemping* [Online]. Available: http://www.bioforsk.no/ikbViewer/Content/108236/Gr%C3%B8nneTak.pdf [Accessed 3.04 2015].

BRENNHOVD, B. 2014. *Målt pg modellert avrenning : Analyse av ledningsnett og urbastasjon på Vestli I Oslo.* Norges miljø-og biovitenskapelige universitet (NMBU).

BUHLER, L. 2013. *Analyse av klimaendrings påvirking på Rustadfeltet med kalibrert modell.* MSc., Norges miljø-og biovitenskapelige universitet (NMBU).

BULTER, D. & DAVIES, J. 2011. *Urban Drainage,* London, Spon Press.

CITYOFCHIAGO. 2014. *City of Chicago- Green Stormwater Infrastructure Strategy* [Online]. Chicago: City of Chigao. Available: http://www.cityofchicago.org/content/dam/city/progs/env/ChicagoGreenStormwaterInfrastructureStrategy.pdf [Accessed 29.07 2015].

COWI. 2007. Utforming av overvannsda. *Miljø Blad* [Online]. Available: http://www.va-blad.no/wp-content/uploads/2014/11/Blad-75_09.11.08.pdf[Accessed 05.08.2015].

COWI 2010. Valg av rørmateriell. *VA/Miljø-Blad* 30.

COWI 2013. På lag med regnet –veieleder for local overvannshåndtering. *In:* COWI (ed.) *Rogaland fylkeskommune/Jæren vannområde.* COWI.

COWI, C. F. M., DRONNINGA LANDSKAP AS 2014. Blågrønn factor- veileder om BGF i byggesaksbehandling, beregning, juridisk forankring og eksempelsamling. Oslo: Bærum og Oslo kommuner/Fremtidens byer/MD.

EKLIMA. *Intensity Duration Frequency curves (IDF)* [Online]. Norwegian Meteorological Institute. Available: http://sharki.oslo.dnmi.no/portal/page?_pageid=73,39035,73_39049&_dad=portal&_schema=PORTAL [Accessed 20.01 2014].

ENDRESEN, S. 2009. Overflateinfiltrasjon. *VA/ miljø-blad* 92.

ENGAN, J. A. 2011. Modelbygging , kalibrering og praktisk bruk av modeller av vannforsyningsnett. *Lecture Notes THT300.* Institutt of matmatiske Realfag og Teknologi (IMT) ved Norges miljø-og biovitenskapelige universitet (NMBU).

ENGAN, J. A. 2014. Håndtering av overvann i et våtere og villere klima. Oslo: Union of Norwegian landscape engineers

EPA. 2009. *Storm water managmnet model applications manual* [Online]. CINCINNATI, OH The United States Environmental Protection Agency (EPA). Available: http://nepis.epa.gov/Adobe/PDF/P100EA7H.pdf [Accessed 20.8 2014].

EPA. 2015. *Storm Water Managmnet Model (SWMM) with Low Impact Development (LID) Controls* [Online]. United States Environmental Protection Agency,

. Available: http://www2.epa.gov/water-research/storm-water-management-model-swmm [Accessed 20.05 2015].

FINSRUD, R. 18.05.2015 2015. *RE: Prices for wastewater conduits and casted pond.* Type to AMIN, F.

FLOODING, F. *Flash Floods* [Online]. National Oceanic and atmospheric adminstration Available: http://www.nws.noaa.gov [Accessed 10.04 2015].

GRIMALDI, S. & SERINALDI, F. 2010. Design hyetograph analysis with 3-copula function. *Hydrological Sciences Journal,* 2.

GRORUDDAL, O. *Historisk utvikling* [Online]. Available: http://www.oppdaggroruddalen.no/(kart) [Accessed 2.01 2014].

IPCC. 2013b. *Fifith Assesment Report (AR5)* [Online]. Working group I- Climate Change 2013: The Physical Science Basis. Available: http://www.climatechange2013.org/images/uploads/WGIAR5_WGI-12Doc2b_FinalDraft_Chapter07.pdf [Accessed 02.12 2014].

IPCC. 2015. *Regional climate change: Findings of IPCC AR5 WGI* [Online]. Intergovernmnetal Panel on Climate Change (IPCC). Available: https://http://www.ipcc.ch/pdf/unfccc/cop19/4_krishna13sbsta.pdf [Accessed 20.07 2015].

KITTERØD, N. O. 2013. Hydrologi. *Lecture notes VANN200, Ås.* Institutt of matmatiske Realfag og Teknologi (IMT). ÅS: Institutt of matmatiske Realfag og Teknologi (IMT) -Norges miljø-og biovitenskapelige universitet (NMBU).

KLEIDORFER, M. 2009. *Uncertain calibration of urban drainage models. A scientific approach to solve practical problems* [Online]. Welcome to the University of Innsbruck. Available: http://www.uibk.ac.at/umwelttechnik/teaching/phd/phd_kleidorfer_online.pdf [Accessed 20.03 2015].

KRISTIANSANDMUNICIPALITY. 2014. *Dimensjonering av overvannsmengder* [Online]. NorskVann, VA-Norm. Available: http://va-norm.no/id_78885 [Accessed 20.02 2015].

LANGELAND, E. 2011. *Implementation of local stormwater management.* Norges miljø-og biovitenskapelige universitet (NMBU).

LAURENSON, E. M. 1964. A catchment storage model for runoff routing. *Journal of Hydrology* 2**,** 141-163.

LINDHOLM, O. 2008. Omdaning av I-V-F kurven til modellregn som f.eks. symmetrisk hyetogram *Lecture notes THT300.* Ås: Institutt of matmatiske Realfag og Teknologi (IMT) -Norges miljø-og biovitenskapelige universitet (NMBU)

LINDHOLM, O. 2012a. Planlegging av framtidens overvannssytemer. *In:* TEKNA (ed.) *Tekna innlandet*

LINDHOLM, O. 2012b. Avløpssystem. *Lecture notes THT 261.* Ås: Institutt of matmatiske Realfag og Teknologi (IMT) ved Norges miljø-og biovitenskapelige universitet (NMBU).

LINDHOLM, O. 2013. Videregående emner om nedbør til bruk I urbanhydrologien. *Lecture notes THT300.* Institutt of matmatiske Realfag og Teknologi (IMT) -Norges miljø-og biovitenskapelige universitet (NMBU).

LINDHOLM, O. 2014. Overvann. *Lecture Notes THT 261.* Ås: Institutt of matmatiske Realfag og Teknologi (IMT) ved Norges miljø-og biovitenskapelige universitet (NMBU).

MUZIK, I. 1996. Hydrological Processes- Flood Modelling with GIS-drived distributed unit hydrographs. 10.

NORGESKART.NO. *Kjelsrud aerial photo* [Online]. Statens kartverk. Available: http://gammel.norgeskart.no/adaptive2/default.aspx?gui=1&lang=2 [Accessed 20.01 2013].

NORSKVANN. *Norsk VA-norm* [Online]. Nors Vann. Available: http://www.va-norm.no/VA-norm/Generelle-bestemmelser/(list)/all [Accessed 20.12 2014].

NORSKVANN 2012. Veiledning i dimensjonering og utforming av VA-transportsystem.

NOU. 2009. Klima I Norge 2100- Bakgrunnsmateriale til NOU klimatilpassning. Available: http://www.nve.no/Global/Vann og vassdrag/Effekter av klimaendringer/klima-norge-2100.pdf [Accessed 10.06.2015].

O´BRIEN, K., S., E., L., S. & LO., N. 2006. Questioning Complacency: climate changes impacts , vulnerability , and adaptation in Norway. *AMBIO: a journal of the Human Environment* [Online], 35. [Accessed 23.06.2015].

OECD. 2013. *The Climate change impacts on the water systems in Norway- country profil. Complied by the OECD secretariat and reflects information avilabele as of june 2013* [Online]. The organisation for Economic Co-operative and Develoment Available: http://www.oecd.org/env/resources/Norway.pdf [Accessed 20.06 2015].

OSLOMUNICIPALITY. 2008. *Oslo mot 2025: Kommunalplan 2008* [Online]. OsloMunicipality Available: https://http://www.oslo.kommune.no/getfile.php/Innhold/Politikk og administrasjon/Kommuneplan/Kommuneplan 2008%2C Oslo mot 2025.pdf [Accessed 20.04 2015].

OSLOMUNICIPALITY. 2013. *Strategi for overvannhåndtering I Oslo (2013-2030)* [Online]. Available: http://www.vann-og-avlopsetaten.oslo.kommune.no/getfile.php/vann- og avl%C3%B8psetaten (VAV)/Internett (VAV)/BILDER NY/Vannet v%C3%A5rt/Overvann/2014-02-06-Strategi for overvannsh%C3%A5ndtering_endelig.pdf [Accessed 02.02 2013].

OSLOMUNICIPALITY. 2015a. *Befolkining etter bydel, delbydel, landbakgrunn og alder* [Online]. Available: http://statistikkbanken.oslo.kommune.no/webview/index.jsp?catalog=http%3A%2F%2Fstatistikkbanken.oslo.kommune.no%3A80%2Fobj%2FfCatalog%2FCatalog52&submode=catalog&mode=documentation&top=yes [Accessed 20.04 2015].

OSLOMUNICIPALITY. 2015b. *Boligbygging etter grunnkrets, delbydel,bydel, bygningsstatus og hustype* [Online]. Available: http://statistikkbanken.oslo.kommune.no/webview/index.jsp?catalog=http%3A%2F%2Fstatistikkbanken.oslo.kommune.no%3A80%2Fobj%2FfCatalog%2FCatalog52&submode=catalog&mode=documentation&top=yes [Accessed 25.04 2015].

OSLOSPEILET. 2013. *Boligbygging i Oslo mot 2034: Hvor bygges det og hvor mange boliger?* [Online]. Oslo Municipality Utviklings- og kompetanseetaten. [Accessed 10.11 2013].

RAMBOLL. 2015. *Risikoakseptkriterier for overvannsflom og dimensjonerende nedbør* [Online]. miljodirektoratet. Available: http://www.miljodirektoratet.no/Documents/publikasjoner/M318/M318.pdf [Accessed 30.04 2015].

ROSSMAN, L. 2010. *Storm Water Management Model (SWMM) user´s guide* [Online]. Environmental protection agency (EPA). Available: http://www2.epa.gov/water-research/storm-water-management-model-swmm [Accessed 20.11 2014].

SSB. 2012. *Befolkningsframskrivinger, 2012-2100* [Online]. Statistisk sentralbyrå. Available: http://www.ssb.no/folkfram/ [Accessed 10.01 2014].

STATENVEGVESEN 2011. 018 håndbook- Vegbygging. OSlo: Staten Vegvesen

SÆLTHUN, N. 2013. Hydroteknikk Hydrologiske modeller. *Lecture notes THT 300.* Ås: Institutt of matmatiske Realfag og Teknologi (IMT) ved Norges miljø-og biovitenskapelige universitet (NMBU).

WILLEMS, P., ARNBJERG-NIELSENC, K., OLSSOND, J. & NGUYENE, V. T. V. 2012. Climate change impact assessment on urban rainfall extremes and urban drainage: methods and shortcomings. *Atmospheric Research,* 103**,** 106-118.

ØDEGAARD, H. 2012. *Vann- og avløpsteknikk*, Norsk Vann

ÅSTEBØL, S., HVITVED-JACOBSENB, T. & SIMONSEN, Ø. 2004. Sustainable stormwater management at Fornebu-from an airport to an industrial and residential area of the city of Oslo. *Science of the Total Environment journal* [Online], 334-335.

10. APPENDICES

APPENDIX A: INPUT PARAMETERS IN SWMM

Table 10.1: Depression Storage (Rossman, 2010)

Impervious surfaces	1.27- 2.54 mm
Lawns	2.54 – 5.08
Pasture	5.08 mm
Forest litter	7.62

Table 10.2: Manning's roughness coefficient (n) - Close Conduits (Rossman, 2010)

Conduit Material	Manning n
Asbestos-cement pipe	0.011 - 0.015
Brick	0.013 - 0.017
Cast iron pipe - Cement-lined & seal coated	 0.011 - 0.015
Concrete (monolithic) - Smooth forms - Rough forms	 0.012 - 0.014 0.015 - 0.017
Concrete pipe	0.011 - 0.015
Corrugated-metal pipe (1/2-in. x 2-2/3-in. corrugations) - Plain - Paved invert - Spun asphalt lined	 0.022 - 0.026 0.018 - 0.022 0.011 - 0.015
Plastic pipe (smooth)	0.011 - 0.015
Vitrified clay - Pipes - Liner plates	 0.011 - 0.015 0.013 - 0.017

Table 10.3: Manning's roughness coefficient (n) - overland flow (Rossman, 2010)

Surface	**n**
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Good wood	0.014
Brick with cement mortar	0.014
Vitrified clay	0.015
Cast iron	0.015
Corrugated metal pipes	0.024
Cement rubble surface	0.024
Fallow soils (no residue)	0.05
Cultivated soils	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Range (natural)	0.13
Grass	
Short, prarie	0.15
Dense	0.24
Bermuda grass	0.41
Woods	
Light underbrush	0.40
Dense underbrush	0.80

Table 10.4: Manning's roughness coefficient (n)-Open channels (Rossman, 2010)

Channel Type	Manning n
Lined Channels	
- Asphalt	0.013 - 0.017
- Brick	0.012 - 0.018
- Concrete	0.011 - 0.020
- Rubble or riprap	0.020 - 0.035
- Vegetal	0.030 - 0.40
Excavated or dredged	
- Earth, straight and uniform	0.020 - 0.030
- Earth, winding, fairly uniform	0.025 - 0.040
- Rock	0.030 - 0.045
- Unmaintained	0.050 - 0.140
Natural channels (minor streams, top width at flood stage < 100 ft)	
- Fairly regular section	0.030 - 0.070
- Irregular section with pools	0.040 - 0.100

Table 10.5: Avilable cross-section shapes for conduits

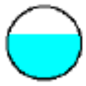
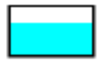
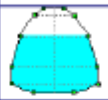
Name	Parameters	Shape	Name	Parameters	Shape
Circular	Full Height		Circular Force Main	Full Height, Roughness	
Filled Circular	Full Height, Filled Depth		Rectangular - Closed	Full Height, Width	
Rectangular – Open	Full Height, Width		Trapezoidal	Full Height, Base Width, Side Slopes	
Triangular	Full Height, Top Width		Horizontal Ellipse	Full Height, Max. Width	
Vertical Ellipse	Full Height, Max. Width		Arch	Full Height, Max. Width	
Parabolic	Full Height, Top Width		Power	Full Height, Top Width, Exponent	
Rectangular-Triangular	Full Height, Top Width, Triangle Height		Rectangular-Round	Full Height, Top Width, Bottom Radius	
Modified Baskethandle	Full Height, Top Width		Egg	Full Height	
Horseshoe	Full Height		Gothic	Full Height	
Catenary	Full Height		Semi-Elliptical	Full Height	
Baskethandle	Full Height		Semi-Circular	Full Height	
Irregular Natural Channel	Transect Coordinates		Custom Closed Shape	Full Height, Shape Curve Coordinates	

Table 10.6: The infiltration parameters for Horton infiltration (Rossman, 2010)

Soil type	**Maximum infiltration rate (mm/h)**		
	Sandy	Loam	Clay
	Dry soil		
with little or no vegetation	127	76.2	25.4
with dens vegetation	254	152.4	50.8
	Moist soil		
soils which have drained but not dried out(i.e. field capacity)	Dry soil/3	Dry soil/3	Dry soil/3
soils close to saturation	Min. Infiltration applied	Min. Infiltration applied	Min. Infiltration applied
Soils which have partially dried out	Dry soil / value (1.5-2.5)	Dry soil / value (1.5-2.5)	Dry soil / value (1.5-2.5)
Soil texture class	Minimum infiltration rate (mm/h) is equal to soil´s saturated hydraulic conductivity (K)(mm/h)		
Sand	120.4		
Loamy Sand	30.0		
Sandy Loam	10.9		
Loam	3.3		
Silt Loam	6.6		
Sandy Clay Loam	1.5		
Clay Loam	1.0		
Silty Clay Loam	1.0		
Sandy Clay	0.5		
Silty Clay	0.5		
Clay	0.25		

Table 10.7: Decay constant values and soil drying time (Rossman, 2010)

Decay constant, K (h^{-1})	Soil drying time (days)	Max. Infiltration volume (mm)[16]
2-7	2-14	0 if not applicable

Table 10.8 Land-use within the catchment

Total area in ha	**38**		
Land use	Area (m2)	Area (ha)	%
Commercial	60020	6,00	16
Residential	176960	17,70	47
Offices	29700	2,97	8
Existing industrial	12280	1,23	3
Hotel& conference	17440	1,74	5
School	16500	1,65	4
Training centre	5790	0,58	2
The Norwegian Police University College	50000	5,00	13
Cultural center	5220	0,52	1
Parking	4790	0,48	1
divsers	4220	0,42	1
Total	382920	38,292	100

---

[16] *It s defined as the differance between the soil´s porosity and it wilting point times the depth pf the infiltration zone (Rossman, 2010)*

Table 10.9 Subcatchments geometric properties

Subcatchment	Area (ha)	Width (m)	Slope (%)	Present of impervious
1	1.1	38.7	0.6	50
2	0.4	14.5	0.6	50
3	0.7	25.0	0.6	100
4	0.3	9.7	0.6	50
5	0.7	24.9	0.6	50
6	0.5	16.3	0.6	50
7	0.3	10.0	0.6	100
8	0.9	32.0	0.6	50
9	0.2	8.3	0.6	100
10	0.2	6.6	0.6	100
11	0.1	4.5	0.6	100
12	0.7	29.1	0.6	50
13	0.4	15.9	0.6	50
14	0.2	10.5	0.6	100
15	0.6	27.0	0.6	50
16	0.7	24.1	0.6	100
17	0.7	31.2	0.6	100
18	0.1	4.8	0.6	100
19	0.3	12.2	0.6	2
20	0.2	7.9	0.6	2
21	0.5	20.9	0.6	50
22	0.1	5.6	0.6	100
23	0.9	38.3	0.6	50
24	0.2	9.4	0.6	100
25	1.0	42.1	0.6	50
26	0.3	11.3	0.6	100
27	0.9	40.3	0.6	30
28	0.2	7.2	0.6	100

29	0.9	39.7	0.6	2
30	0.7	50.8	0.6	50
31	1.1	76.0	0.6	50
32	1.1	76.0	0.6	50
33	0.3	20.2	0.6	100
34	0.3	20.2	0.6	100
35	1.0	64.3	0.6	50
36	1.5	95.4	0.6	2
37	0.5	32.6	0.6	2
38	0.6	39.8	0.6	50
39	0.2	6.9	0.6	100
40	1.3	55.2	0.6	70
41	1.3	55.2	0.6	70
42	1.0	42.1	0.6	30

***Table 10.10 Price of b 2012 for wastewater conduit for rural** and easy-to dig area included foundations and refilling ballasted (adopted from (Finsrud, 2015))*

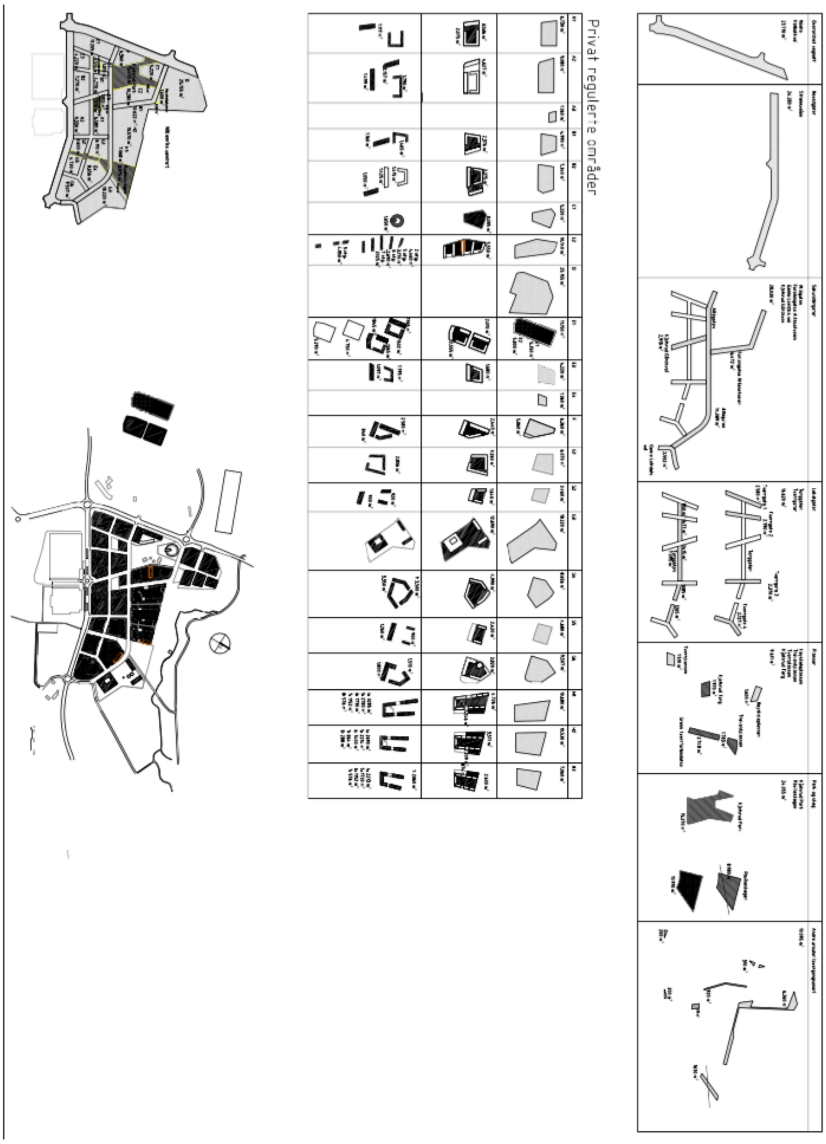
Rørdim	Område	Grunnforhold	Kostnad	Anmerkning
mm			kr/m	
300	Landlig	Gravbart	850	
400	Landlig	Gravbart	1177	Fundament og omfylling
500	Landlig	Gravbart	1514	med pukk
600	Landlig	Gravbart	1860	
700	Landlig	Gravbart	2254	
800	Landlig	Gravbart	2944	
900	Landlig	Gravbart	3726	
1000	Landlig	Gravbart	4600	

***Table 10.11 Price of 2012 for moulded basin** (adopted from (Finsrud, 2015))*

Basseng-volum	Område	Grunnforhold	Sum kostnad	kr/m3	Anmerkning
m3			kr.		
100	Landlig	Gravbart	498816	4988	
200	Landlig	Gravbart	782726	3914	Ved fjell øker
300	Landlig	Gravbart	1018753	3396	kostnaden med
400	Landlig	Gravbart	1228228	3071	ca. 10%
500	Landlig	Gravbart	1419942	2840	
600	Landlig	Gravbart	1598595	2664	
700	Landlig	Gravbart	1767070	2524	
800	Landlig	Gravbart	1927296	2409	
900	Landlig	Gravbart	2080643	2312	
1000	Landlig	Gravbart	2228127	2228	

APPENDIX B: MAPS AND FIGURES

Figure 10.1: Areal calculations (ARC, 2013)



igure 10.2: Illustration of main alternatives for VPOR –Kjelsrud (ARC, 2013)

Figure 10.3: Runoff flow direction in Kjelsrud (ARC, 2013)

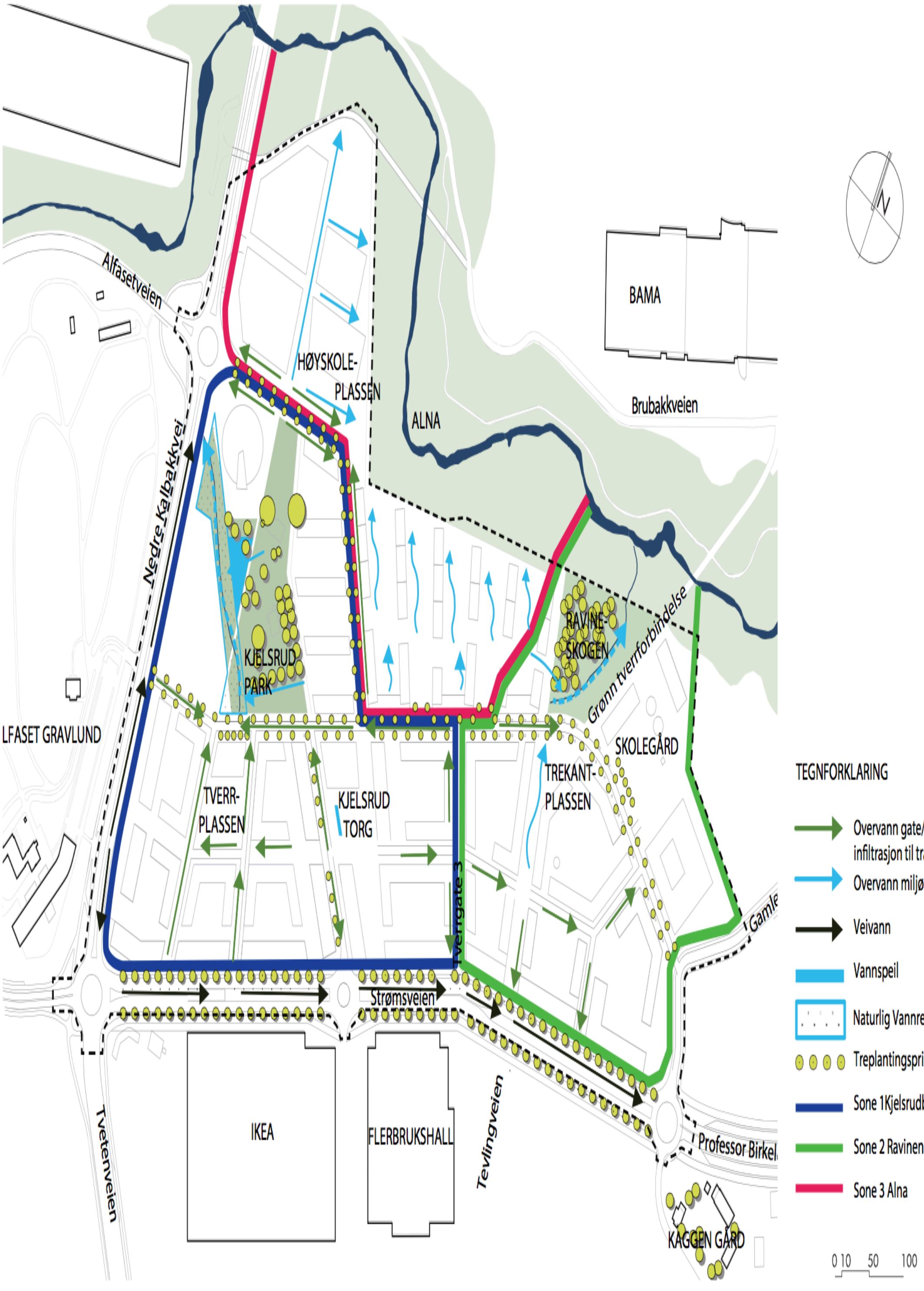
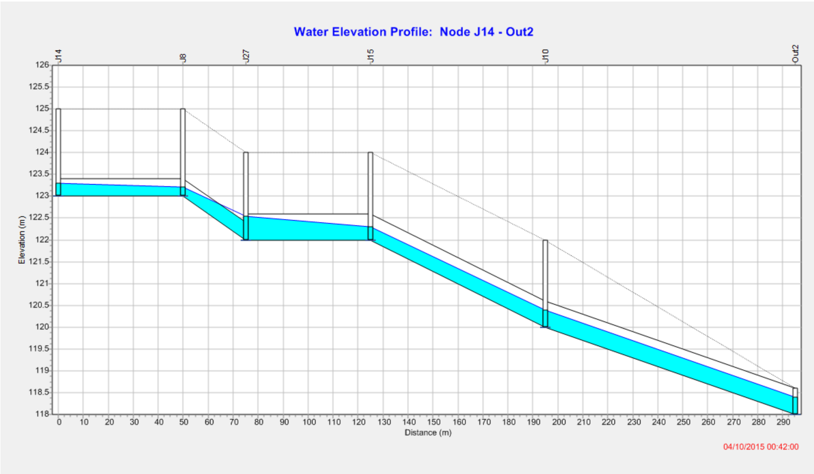
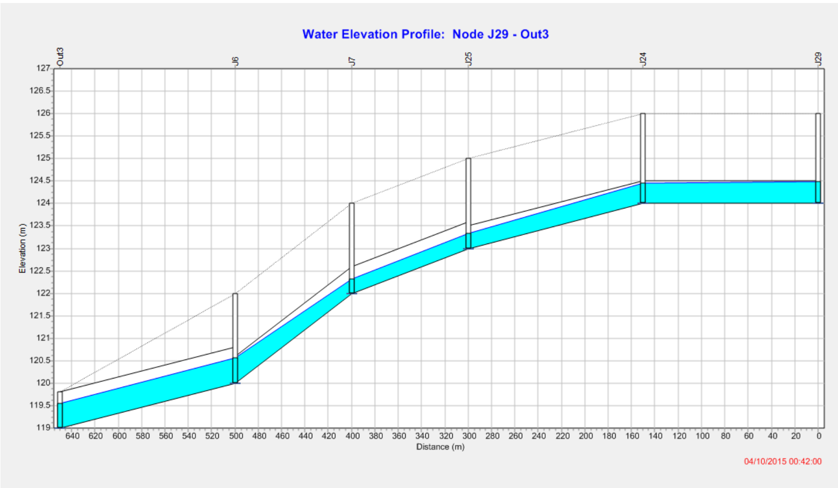


Figure 10.4: Profile plot for outlet 2 -4 response to the design storm without climate change





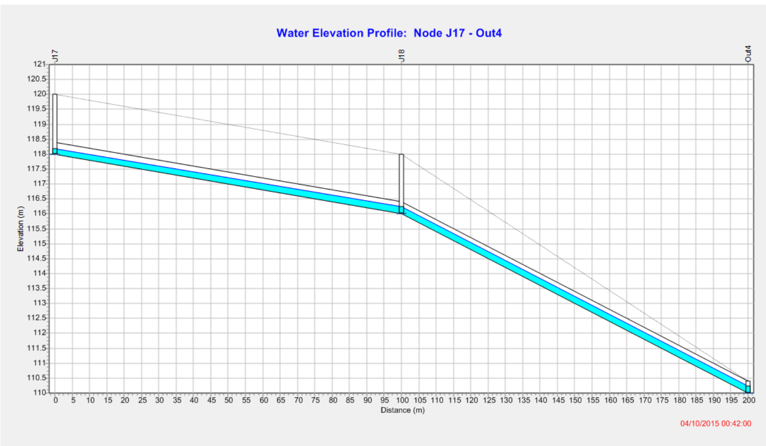
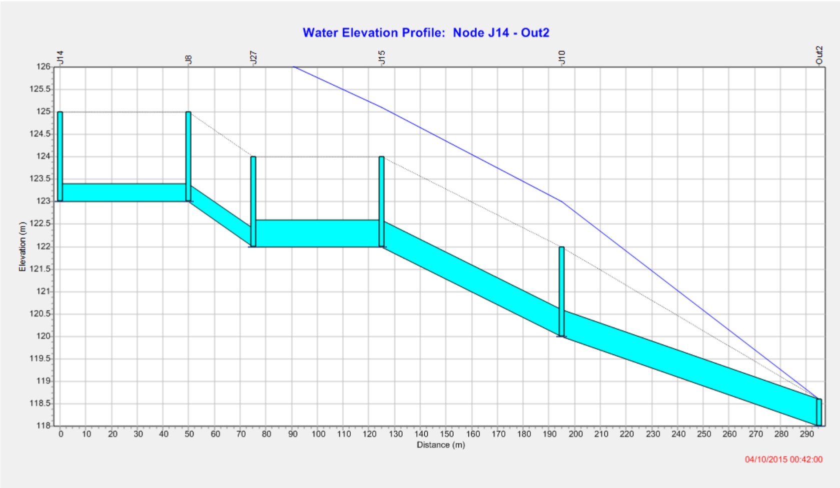
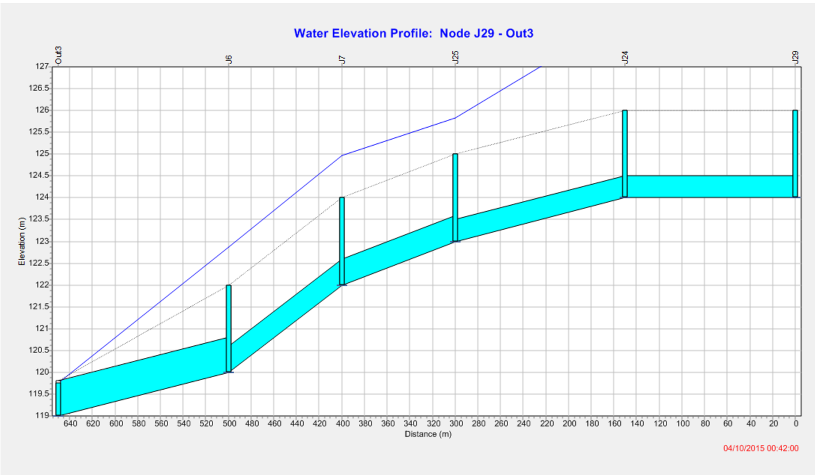
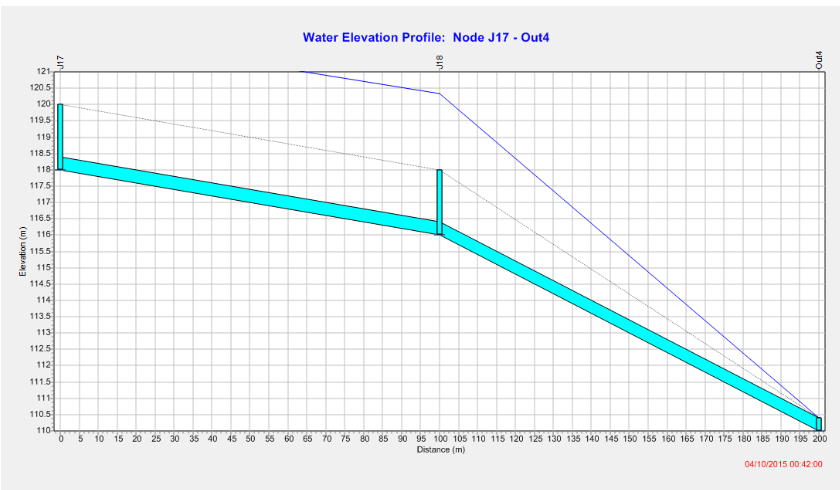


Figure 10.5: Profile plot for zone 2 -4 response to the design storm +50% climate change







Norwegian University
of Life Sciences

Postboks 5003
NO-1432 Ås, Norway
+47 67 23 00 00
www.nmbu.no