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Testing of Infiltration System for Stormwater

Permeable Pavement

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Civil and Environmental Engineering

Submission date:	June 2016
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Abstract

In this master thesis it is focused on permeable pavement and with an extra focus on permeable interlocking concrete unit blocks as the type of pavement. This is because the test facility in Sandnes is built with this type of pavement, and all the data collected and used in the master thesis is from this test facility. The measurements started in May 2015 and parameters like air temperature, ground temperature, precipitation intensity, air pressure, wind speed, wind direction, water level in the infiltration basins and excess flow from the permeable areas are continuously registered in the system.

The different parts of the test facility from construction of the permeable pavement to the instrumentations used for collecting data are described in detail. Test area 1 and 2 is the access road and are only infiltrating water from the area they cover, which is 2 x 120m^2. Test area 3 is 104m^2 and infiltrates water from an impermeable part of the parking area of 1250m^2.

The test facility have been doing constant registrations of data since the time of construction, but they were not controlled to see if the quality were good and if they were useful and could give some indications of the behavior of the permeable pavement. The potential evaporation from the permeable pavement is calculated and events with inflow to the infiltration basins in connection to the permeable test areas are analyzed.

The saturated hydraulic conductivity was measured for one of the test areas after almost a year without any maintenance since the construction. When the openings in the joint filling material remained untouched the infiltration rate was measured to be 78.9 cm/h and 107.2 cm/h. The upper 5 cm of the joint filling material were removed and the infiltration was measured to be 554.6 – 670.9 cm/h. The results show indication of fine particles in the upper 5 cm of the joint filling material clogging the pavement. The grain size distribution analyses show that 10 % of the weighted joint filling material had a smaller diameter than 2 mm.

The infiltration capacity was calculated for different rainfall events where inflow to SF2 was registered. The calculated infiltration capacities are lower than the measured infiltration capacity, but the time of the year might have affected the permeable pavement. Most of the calculated values for infiltration rate are from December to February, while the measurements were done in April.

Sammendrag

Fokuset for denne masteroppgaven har vært på permeable dekker og med et ekstra fokus mot belegningsstein som type permeabelt dekke. Det er fordi testfeltet i Sandnes er av denne typen og alle data som er samlet inn og benyttet i masteroppgaven er fra dette testfeltet. Målingene startet i mai 2015 og parametere som lufttemperatur, temperatur i bakken, nedbørintensitet, lufttrykk, vindhastighet, vindretning, vanndybden i infiltrasjonskummer og avrenning fra de permeable områdene måles kontinuerlig.

De forskjellige delene av det permeable dekket, fra oppbygging til instrumentering brukt for innsamling av måledata er beskrevet i detalj. Test område 1 og 2 er innkjøringsveien og infiltrerer kun vann fra det området de dekker, som er 2 x 120 m^2. Test område 3 er 104 m^2 og infiltrerer vann fra den 1250 m^2 store impermeable delen av parkeringsplassen.

Testfeltet har gjort kontinuerlige målinger siden det stod klart, men kvaliteten på de innsamlede dataene er ikke kontrollert eller sjekket for å se om de er brukbare. Det har vært ønskelig å se om dataene kan gi noen indikasjoner på oppførselen til et slikt system med permeable dekker og infiltrasjonskummer. Den potensielle fordampningen fra det permeable dekket er regnet ut, og hendelser hvor vannføring er registrert inn i infiltrasjonskummene er analysert.

Den mettede hydrauliske ledningsevnen ble målt for et av testområdene nesten ett år etter ferdigstillelse, uten at noe vedlikehold har vært utført på området. Infiltrasjonsraten ble målt til 78.9 cm/h og 107.2 cm/h for urørt fugemasse mellom belegningssteinene. Når de øverste 5 cm av fugemassen ble fjernet, ble infiltrasjonskapasiteten målt til 554.6 – 670.9 cm/h. Resultatet viser indikasjoner på tetting av fine partikler i de øverste 5 cm av fugemassen. En kornfordelingsanalyse ble gjennomført og resultatet viser at 10 % av det veide materialet fra fugemassen hadde mindre enn 2 mm diameter.

Infiltrasjonskapasiteten ble regnet ut for forskjellige nedbørshendelser hvor vannføring inn i SF2 var blitt registrert. Den beregnede infiltrasjonskapasiteten var lavere enn den målte infiltrasjonskapasiteten, men tid på året for målingene og registreringen av data kan ha påvirket det permeable dekket. De fleste av de beregnede infiltrasjonskapasitetene er fra hendelser i desember til februar, mens målingene ble gjennomført i april.

Preface

This master thesis is conducted at the Department of Hydraulic and Environmental Engineering at the Norwegian University of Science and Technology (NTNU) and delivered in June 2016. The thesis has been supervised by Professor Sveinung Sægrov which also introduced me to the project, and Associate Professor Tone Merete Muthanna as co-supervisor. Per Møller-Pedersen in Storm Aqua has worked as my external supervisor.

I will like to say how grateful I am for the regular meetings with Professor Sveinung Sægrov and his guidance and advices in the process of writing this thesis. I will also like to mention Associate Professor Tone Merete Muthanna for her guidance and help with parts of this project, and Sveinn T. Thorolfsson for sharing his knowledge.

I will also express my gratitude to Per Møller-Pedersen who have provided me with all the information about the test facility in Sandnes and answered all my questions during this semester. When I visited the test facility in April, Per Møller-Pederesen and especially Geir Lillebø helped me out with the infiltration tests and everything I needed, thank you.

Trondheim 10.06.2016

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1 Introduction

1.1 Background

In the recent years, we have seen an increasing runoff from rainfalls in populated areas. This is because of the continuous building of impermeable surfaces in the cities and the increase in rainfall intensity and duration due to climate changes. In Norway the pipelines are getting old and they can't handle this huge amount of runoff from impermeable surfaces like roads, parking areas, roofs and so on. This is because they are designed for other conditions with regards to rainfall intensity and adjacent area of impermeable surface, which has changed in the last years. There are restrictions in many municipalities to regulate the amount of runoff you can add to the pipelines. We need to find sustainable solutions to take care of the stormwater, and to infiltrate and detain water at the source seems to be a plan for the future. The rainwater should be looked at as a resource and be taken care of in such ways that flooding of the system downstream should be prevented.

With this new thinking of how to handle stormwater, there are a lot of interesting options for infiltration, detention and delaying the runoff. One of them is to use permeable pavement instead of regular impermeable pavement. With permeable pavement you can infiltrate the rainwater and at the same time maintain a hard functional surface for vehicles and people.

Skjæveland Gruppen in Sandnes is producing concrete blocks for permeable paving and they are interested in good solutions for infiltration and reduction of rainfall runoff. This is why they built a research field with permeable concrete block pavement. The test field has instrumentations to do measurements and have been doing so since May 2015. The data have just been collected and stored. A big part of the work will be to process the measurements, discuss the functions of the test field, give recommendations of how to document such a test field and help to understand more about the use of permeable pavement.

1.2 Given tasks for the thesis

1. Present the test facility at Skjæveland including the permeable pavement, ground conditions, groundwater conditions, infiltration manholes and attenuation facilities and discuss how each element contribute to reduction and leveling of runoff from the test site.
2. Collect and process the results from the measurements with regards to precipitation, runoff, groundwater level and temperatures in the ground. Come up with suggestions for localizing more wells for ground water observations and process the data from the measurements.
3. Build a water balance model for the test field that includes the main elements in the hydrological cycle, precipitation, evaporation, infiltration, attenuation and drainage. See if the program SHYFT from Statkraft can be used for this purpose.

4. Discuss the functional ability of the system and identify possible weak spots and bottlenecks

1.3 Refinement and changes done to the given tasks

There is some discussion of the groundwater and the impact on the infiltration and the test area. It is planned to drill wells to measure the groundwater level, but they are not completed. Only suggestions for placement are made. A closer look at the groundwater and its impact on the test area will be possible when the data for groundwater level is registered.

The SHYFT program from Statkraft was looked into, but it was complicated and for the purpose of just looking into the water balance of the test area in Sandnes, excel and more simple approaches are more suited.

In addition to the given tasks, a test of the saturated hydraulic conductivity for the permeable pavement in test area 3 where done, together with a particle size distribution analysis of the joint filling material.

1.4 Outline

Chapter 1 – Background for the project.

Chapter 2 – Is about stormwater management, and why implementations of local treatment solutions for stormwater are necessary.

Chapter 3 – In this chapter different types of permeable pavements are presented, together with design methods, the structure from top to bottom and benefits and limitations with the use of permeable pavements.

Chapter 4 – Presentation of the test facility in Sandnes. A description of test area 1-4 and all the sensors used to measure water level, precipitation, water flow and the PLC system.

Chapter 5 – Presentation of the results from the data collected in the period from May 2015 to April 2016. The data are discussed as they are presented in the chapter.

Chapter 6 – The data in Chapter 5 are used together with the Penman formula to calculate the potential evaporation for the permeable pavement in test area 3.

Chapter 7 – The method, results and discussion of the infiltration test are presented in this chapter.

Chapter 8 – The method, results and discussion of the grain size distribution analysis are presented in this chapter.

Chapter 9 – This chapter is about the water balance model. The infiltration capacity of test area 3 is looked into together with rainfall events with registered inflow to SF2.

Chapter 10 – Some suggestion for improvements of the test area and the processing of data that could be helpful for monitoring such a test facility, and some more general experiences for the construction of such a stormwater infiltration system.

Chapter 11 – The conclusion of the thesis, and some suggestions for further work.

Chapter 12 – References

Appendix A – Given tasks for the master thesis.

Appendix B – Matlab code for calculating saturated hydraulic conductivity.

Appendix C – Excel sheet with input data from MPD measurements used by the Matlab code in Appendix B

2 Context of stormwater management

2.1 Stormwater management, basic hydrology

To understand why permeable pavements are interesting and what we try to achieve by implementing them, some basic hydrology is necessary.

Stormwater is runoff from precipitation and snow melting. In urban areas the runoff is high because of impermeable surfaces like roads, pavements, roofs and open areas covered with asphalt or other pavement that prevents the natural infiltration of water. Figure 1 shows what happens with the runoff from an area where we have an increasing urbanization. The runoff volume increases in maximum runoff and total volume, and all this happen in much faster time. (Ødegaard et al., 2012) Chap.14 and Chap.2

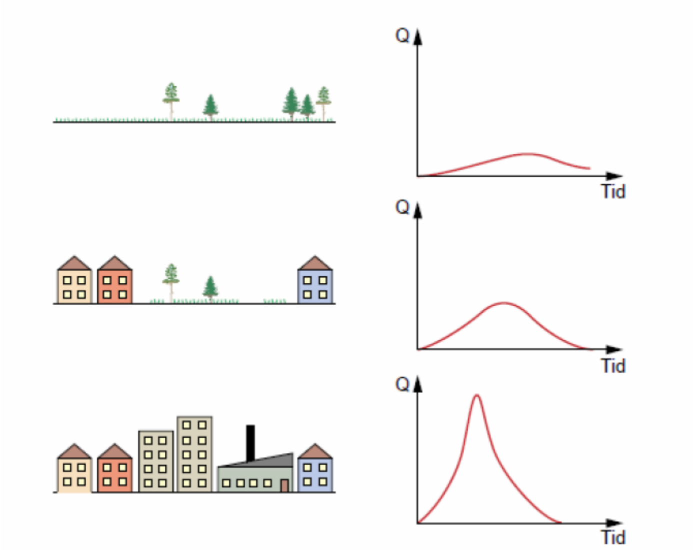

Figure 1 How runoff from an area change with increasing level of urbanization (Ødegaard et al., 2012)

Norsk Vann has come up with a “three step strategy” for how to handle stormwater (See Figure 2). This is how it should be done, to achieve the goals they have set for dealing with stormwater. The goals are as follow (Magnussen et al., 2015):

1. Prevent damage
2. Utilize the water as a resource
3. Strengthen the biological diversity in the city environment

The “three step strategy” means that all small precipitation events on the property will be infiltrated. All medium sized events should be delayed and detained. And for the bigger events, safe flooding paths should be provided. (Magnussen et al., 2015)

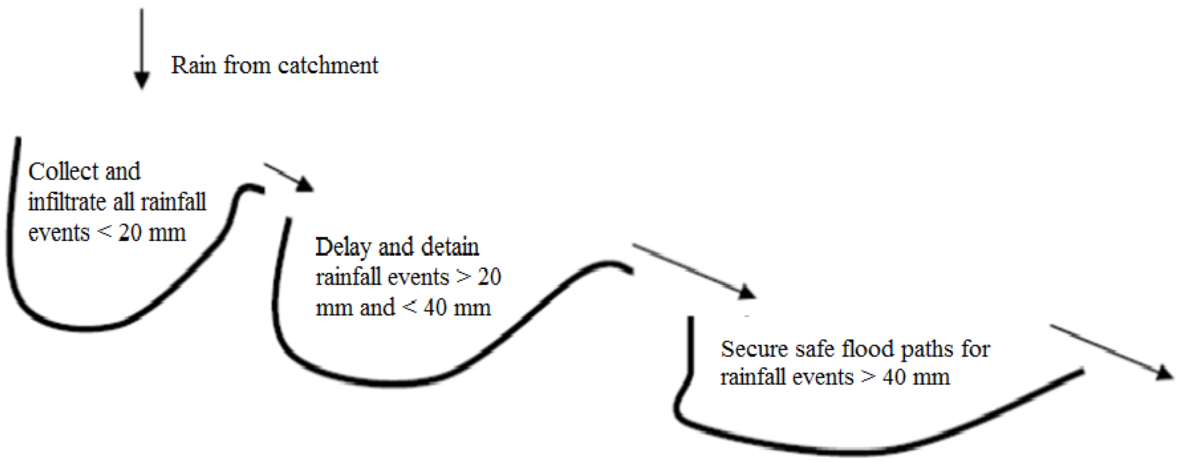

Figure 2 Three step strategy to handle stormwater, adjusted from (Magnussen et al., 2015)

First step in the figure is to collect and infiltrate all rainfall events less than 20 mm. The second step is to delay and detain rainfall events between 20 mm and 40 mm. The last step is to secure safe flood paths for rainfall events larger than 40 mm.

Permeable pavements would be able to contribute to the first two steps of the “three step strategy” and they may also be used as safe flooding paths, but not because of their permeable ability. If the native soil has good infiltration capacity, the permeable pavements will help to infiltrate the water and maintain the ground water level. If the soil gets saturated with water, the water can be stored inside the construction, and if needed an extra tank can be connected to the underdrain and give even more capacity to detain the runoff.

2.2 Polluted stormwater

The difficulties with stormwater are not just related to increased runoff volume. The spreading of pollutants through the stormwater is also an issue. Spilling and wearing from cars on the surface, traffic emissions and building particles. When it rains it gets washed down the drains and contributes to the pollution of the recipient unless it is sent to a waste water treatment plant, which is costly.

Permeable pavements could have a positive effect on particle bound pollutions in stormwater. A lot of the pollution in runoff water from roads is particle bound except for the salt used during the winter. Among the unit process for permeable pavements in Chapter 2.3, there are some processes which provide quality control.

2.3 Unit processes for permeable pavements

Unit processes are mechanisms that reduce runoff volume, reduce flow rates, and reduce pollutant loads or thermal loads. Different Sustainable Urban Design Solutions (SUDS) will provide different unit processes. Examples of unit processes are precipitation, adsorption, peak attenuation and sedimentation. Some unit processes provides water quality control others provide quantity control, and they can often do both. (Water Environment Federation (WEF), 2012)

For permeable pavements the unit processes will be infiltration of water to the soil where this is possible, filtration of the water through the surface layer, peak flow attenuation when water is stored in the base layers, temperature reduction when water goes through the layers, runoff volume reduction also from infiltration of water to the soil, sorption which will happens in the joint filling material when small particles start to attach and clog the pavement, precipitation, coagulation, organic compound degradation, pathogen die-off and disinfection. (Water Environment Federation (WEF), 2012)

When you have unit operations in series of one or more, it is called a system. If you have a green roof connected to the base layer of a permeable pavement it will be a system with two unit operations. In Figure 3 the green roof and the permeable pavement are connected to an infiltration manhole which is connected to a detention basin. This will be a system with several unit operations.

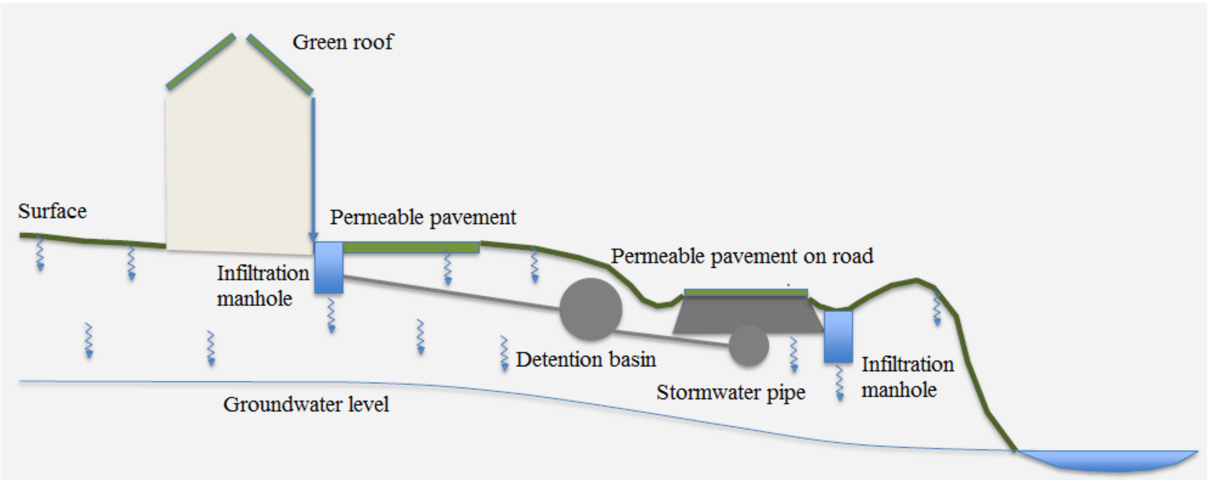

Figure 3 Example of a system with several unit operations. (Møller-Pedersen, 2016)

3 Permeable pavements

3.1 Why permeable pavements

The climate changes we are facing will lead to increased rainfall with regards to both frequency and intensity. For the mainland of Norway it has been registered a statistically increase in annual precipitation of 18 % the last 115 years (Hansen-Bauer et al., 2015). Combined with a growing number of impervious surfaces in the cities, existing stormwater systems are no longer sufficient. The pipelines are running full, and we need to look for alternative methods to handle the water. As far as possible, we want to take care of the stormwater locally at the source. There are different solutions with this goal in mind, and permeable pavements are an interesting option.

In cities and populated areas, it is often necessary with a lot of hard surfaces for cars and people to walk on. You have roads, parking areas, open places for people to walk or just sit down. All those surfaces are usually regular asphalt or other types of pavement which generates a lot of runoff when it starts to rain. This is why permeable pavements are so interesting. They have the possibility to maintain the hard surface you need for practical use, and at the same time infiltrate stormwater and store it locally.

Permeable pavements are used a lot in countries like Australia and USA. Also in Germany there are quite a lot of permeable pavements in use. In Norway there are just a few places with permeable paving. The potential for this type of pavement in Norway is much bigger than we can see today.

3.2 Benefits and limitations of permeable pavements

There are a lot of benefits with permeable pavements if you compare them with impermeable pavements. Most of the benefits and limitations listed below are from (Smith, 2006).

Benefits:

- Reduction in runoff with as much as 100 % for smaller rainfall events.
- There is no need for big detention basins when the water can be stored inside the construction.
- Recharge of groundwater when water infiltrates to the ground.
- Reduce the amount of pollutants in the stormwater which increase the water quality.
- Reduce the peak discharge and time to peak discharge in stormwater sewers.
- If the runoff water is transported to an urban stream, the peak flow reduction will prevent erosion down streams.
- Reduction in construction costs for pipes and stormwater treatment, when the water is taken care of locally by infiltration.

- Less ice on pavements when snow is melting. Prevents the water from freezing on top of the surface by directly infiltrate the water.
- Reduce the temperature of the stormwater when it is infiltrated through the layers of the pavement (Eisenberg et al.).
- Improved surface benefits like: reduce the risk of hydroplaning, reduce heat island effect, lower risk of freeze/thaw conditions if compared with regular pavements (Eisenberg et al.).

Limitations:

- It is expensive compared to regular pavement and also compared to other LID/SUD solutions it could be expensive.
- The site needs more evaluation and more work is needed in the design process and the construction process
- Maintenance of the surface is needed to prevent clogging and maintain good infiltration capacity.
- Permeable pavements are usually not suitable for roads with high traffic because of reduced strength compared with regular pavements.

As listed above, the benefits from the use of permeable pavements are great when they perform as expected. The problem is when surrounding conditions reduce the performance, with clogging and freezing as the two main concerns. Some types of permeable pavement are not necessarily more expensive and work demanding than regular pavement, but they have other limitations like lower temperature of the permeable asphalt when laid out.

3.3 Different types of permeable pavements

Permeable pavements are often divided into four different categories. They are porous asphalt, pervious concrete, permeable interlocking concrete pavement and grid pavement systems. (Eisenberg et al.)

3.3.1 Porous asphalt (PA)

Porous asphalt looks like regular asphalt, but fines are removed from the asphalt which creates greater void space and room for water to infiltrate. Higher graded binder is also typically used to increase the durability and prevent the asphalt binder to drain down through the structure together with the stormwater. (Eisenberg et al.)

It is not recommended for roads with a lot of heavy vehicle traffic. It is better for parking areas, sideways, pathways and less trafficked areas. The better things with PA is the high surface permeability, could range from 430 cm/h to 1250 cm/h and the high level of pollutant removal. Porous asphalt is similar to conventional asphalt but with a courser texture. It is suitable in most places and climates where you can use conventional asphalt. (Eisenberg et al.)

Freezing conditions

Even if porous asphalt is appropriate to use in areas where conventional asphalt is used, some extra design criteria's has to be met to make sure it will operate as normal and not fail due to the cold climate conditions. (Eisenberg et al.)

The factors include:

- frost depth
- frost durability of materials
- frost heave of the subgrade
- frost durability of the saturated system

Prevent damage from freezing:

- Appropriate drainage: Could be a layer of open graded stone in the bottom of the structure to prevent water from going into the structure from the bottom. If the subbase gets saturated and it freezes it could lead to frost heave and this will weaken the structure.

Porous asphalt is often performing well in cold climates, because the structure has a lot of open spaces where water can expand when freezing (Eisenberg et al.). In the article by (Eisenberg et al.) it is referred to a study done by (Roseen and Ballestero, 2008) where the need for winter maintenance salt is reduced by 50 % to 75 % for PA compared to regular asphalt. The reduction can be explained because when snow and ice melts on the PA pavement the melting water will infiltrate and not refreeze on top of the pavement as it will do for regular pavement. In Figure 4 the difference between the two types of pavement is pictured.

Figure 4 Dry porous asphalt close to the camera and regular asphalt further away from the camera (Eisenberg et al.) p. 55

3.3.2 Pervious concrete (PC)

Cementitious binding combined with open graded aggregate forms a rigid pavement with interconnected void spaces. The pervious concrete pavement is most used in parking areas and the aggregate used for mixing will decide the surface texture of the pavement. 5.75 mm is the most commonly used grain size. (Eisenberg et al.)

3.3.3 Permeable interlocking concrete pavement (PICP)

The permeable interlocking concrete pavement is also tested with success on parking areas, pathways and low volume roadways. It is the opening between the concrete units that provides the permeability of the pavement. This is usually 5 % - 15 % of the total area, and consists of course graded material which allows the water to infiltrate. (Eisenberg et al.)

Freezing conditions

(Eisenberg et al.) claims that it has been demonstrated that PICP does not heave when frozen, as a conclusion of years with experience and monitoring. They remain stable when exposed to freezing and thawing climates. This can be explained by the factors listed below (Eisenberg et al.):

- The pavement drains prior to freezing.
- The air in the void spaces in the structure have an isolating effect slowing the freezing of soil.
- The earth provides some heat so the structure and soil have time to drain.
- There is space in the subgrade for water to expand if freezing should occur.
- Water will infiltrate during warm winter days, so the water will not freeze again and again.

3.3.4 Grid pavement systems

Grid pavements are made of open celled plastic or concrete units that can be filled with soil and grass or permeable aggregates. The surface of the grid pavements will be cooler than for the other types of permeable pavement because of the different surface material. Infiltration rates range from 1 – 19 cm/h and with 6.9 cm /h as an average of all the sites tested (Eisenberg et al.).

Figure 5 Grid pavement with grass to the left from (Dye, 2014) and aggregate to the right from (Ltd, 2015)

3.4 Structure of permeable pavement

There are many kinds of permeable pavement, and there are great variations in the designs. The visible part of the permeable pavement is what we see of the pavement design. Roughly speaking we can say that there are two principles for the top layer of permeable pavement.

1) The water is infiltrating directly through the paving material. This is the case for porous asphalt, porous concrete, grass and gravel.
2) The water is infiltrating through open spaces in the pavement designed for water to infiltrate: Permeable interlocking concrete pavement.

One of the biggest differences between regular pavement and permeable pavement is that for permeable pavement water is allowed in the structure. The water cannot be stored indefinitely it needs to be infiltrated to the ground if possible, or taken away by some kind of drainage. (Beeldens and Vijverman, 2015)

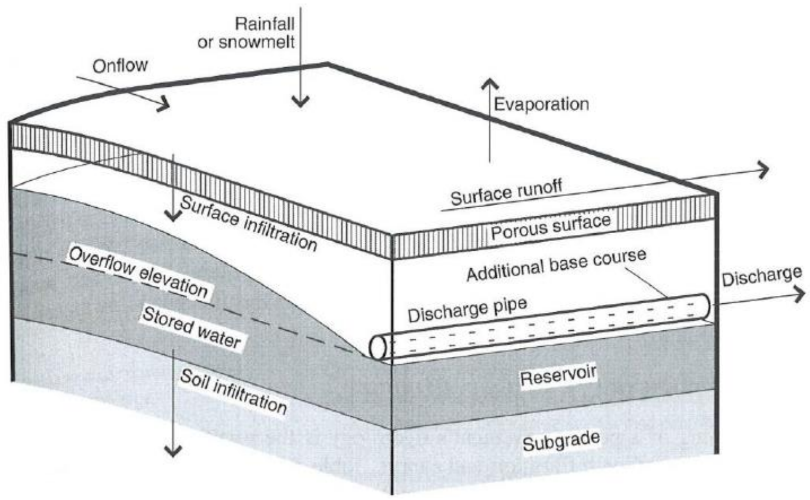

Figure 6 Hydrologic processes in porous pavement from (Korkealaakso et al., 2014)

The porous pavement or the openings between the concrete blocks infiltrates the water to the structure. If any rainfall or water falls on top of the pavement, it will go through if the infiltration capacity of the permeable pavement is sufficient. The water can't stay this high in the structure it has to infiltrate further down. The paving bed provides a smooth surface for the concrete pavement units or porous pavement to be laid. The base layer gives the bearing capacity and it is designed for the traffic load expected on the road or parking area. The subbase layer is where the water is stored in the structure. It also gives protection against frost heave. If infiltration to the ground is not sufficient, some kind of drainage will be required. (Beeldens and Vijverman, 2015)

It is important that the water permeability increases with the depth in the structure. Because particles that come with the water will then clog the top layer and not the layers further down in the structure. If the surface layer is clogged, there is different ways to clean it by water jetting, vacuum cleaning or refilling the joints. The clogging can then be fixed by replacing only the top layer. This makes it important for permeable interlocking concrete pavements to refill the joints if they are missing. If not the chance of clogging will increase. And if it happens in some other part of the structure, like the bedding layer because of the missing joint materials, the costs of regaining the pavements infiltration capacity is more expensive. Then structural layers have to be removed in order to clean the pavement. (Beeldens and Vijverman, 2015)

Another important thing is to check the thickness of the base layer and see if it can take the design load from cars and vehicles passing on top. It is not only the water storage capacity that needs to be calculated. Also the thickness in connection to the traffic load needs to be controlled. (Beeldens and Vijverman, 2015)

3.4.1 Infiltration systems

Depending on what type of native soil, groundwater table, pollutions in the ground and topography, there are three types of systems for permeable pavements to choose between. (Myhr, 2013)

System A: Total infiltration

If the native soil has good infiltration capacity and the groundwater table is low, there is no need to take into account for storing water in the structure. When building this kind of system it is important that the ground is not water sensitive or exposed to freezing conditions. The limiting factor of this system is the permeability of the drainage cell or open joints between the concrete block units.(Myhr, 2013)

System B: Partial infiltration

If the capacity in the native soil is not sufficient to infiltrate all the stormwater or the groundwater table is high, some kind of drainage will be required. This will typically be a perforated pipe in the bottom of the structure leading the water away. The peak flow will be detained in the base layer and subbase layer. Some kind of basin beside the permeable pavement is required to temporary store the water before it can enter the municipal stormwater network under controlled conditions.(Myhr, 2013)

Detention to increase the infiltration over time is also a possibility. Either in the structure or in a sandtrap/infiltration basin connected to the permeable pavement.

System C: No infiltration

It is not possible to infiltrate water to the ground. The boundary between the native soil and the subbase layer needs to be secured with a tightly closed membrane if the ground is polluted or need protection from water. A drainage pipe is there to transport the water out of the structure. (Myhr, 2013)

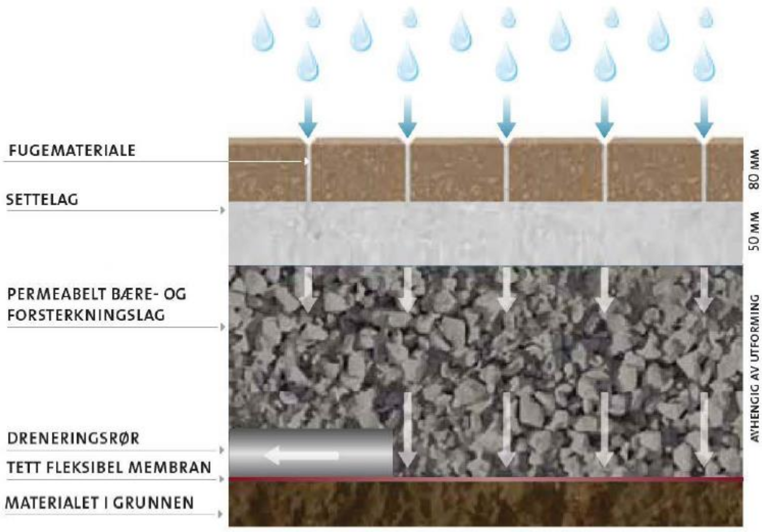

Figure 7 Design of system C: No infiltration (Myhr, 2013)

Figure 7 shows a possible design of the structure if there is no infiltration to the ground. The only difference between this structure and the one for system A and B, is that for system A there is no drainage pipe and the tight flexible membrane in the bottom is changed with a geotextile. For system B the structure looks quite the same, but some water is going through and infiltrates to the ground.

To make sure the water will have time to infiltrate, baffles, check dams or berms can be built. When permeable pavements are constructed on areas with a slope, typically greater the 3 %, the water tends to follow the soil and not infiltrate. To fully utilize the infiltration capacity of the ground and not only the most downstream part, the design in Figure 8 can be followed (Eisenberg et al.). From (Korkealaakso et al., 2014) it is referred to the article by (Hansen, 2008a) where berms, baffles and check dams are recommended for a ground steeper than 5%.

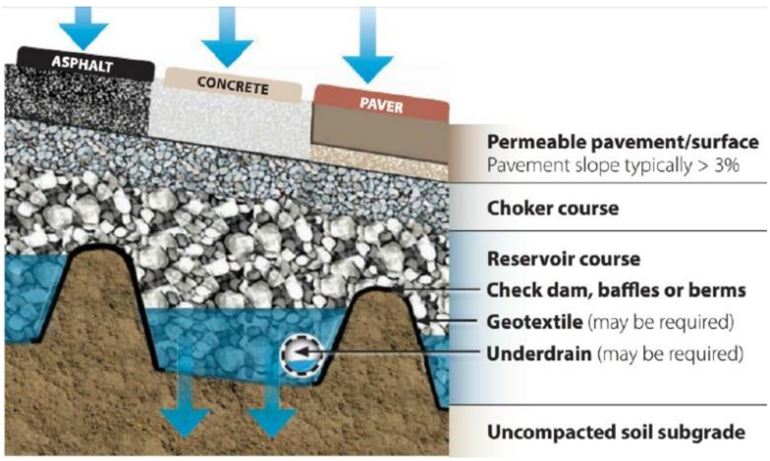

Figure 8 Sloped permeable pavement, built to secure infiltration to the ground (Eisenberg et al.)

In the document by (Eisenberg et al.) it is made a decision tree for how to approach the hydrological and structural design of a permeable pavement (see Figure 9).

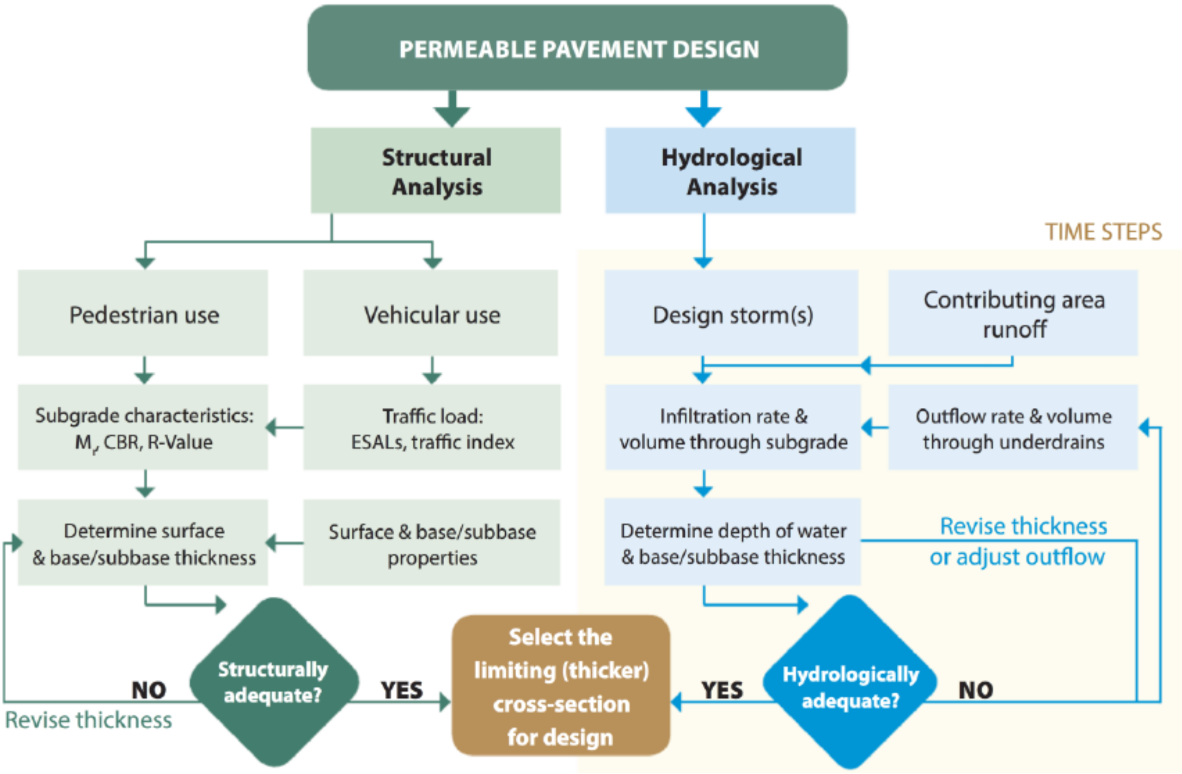

Figure 9 Structural and hydrological decision tree for permeable pavements (Eisenberg et al.)

Explenation to the decicision tree Figure 9.

- CBR California Bearing Ratio
- ESALs Equivalent and single axle load
- K Modulus of subgrade reaction
- M_r Resilient modulus
- R-value Resistance value of soil

For the hydrological design two conditions must be considered:

1) The permeability of the pavement.
2) The storage capacity.

The permeability is the ability of water to move through the pavement. The storage capacity is the combined capacity of the pervious pavement, the capacity of the subbase, and the water leaving the system through infiltration to the ground. In (Korkealaakso et al., 2014) the volume space in the previous concrete layer is included in the storage capacity.

It is recommended that impervious areas close to the permeable pavement contributing to the runoff should not be larger than two times the pervious area(Minnesota Stormwater Manual, 2013). Another article is suggesting a ratio of impervious over pervious of 5:1(Hansen, 2008a). They are both referred to in the article by (Korkealaakso et al., 2014)

3.4.2 Design suggestions from different manuals and studies

Statens Vegvesen Håndbok - N200

Permeable pavements are suggested to be constructed with basis in Statens Vegvesen Håndbok – N200 (Statens Vegvesen, 2014). The thickness and grain size is given for each layer. The design in Figure 10 is made specific for paving stones. It is important that there is no zero substance in any of the mass used for the structure.

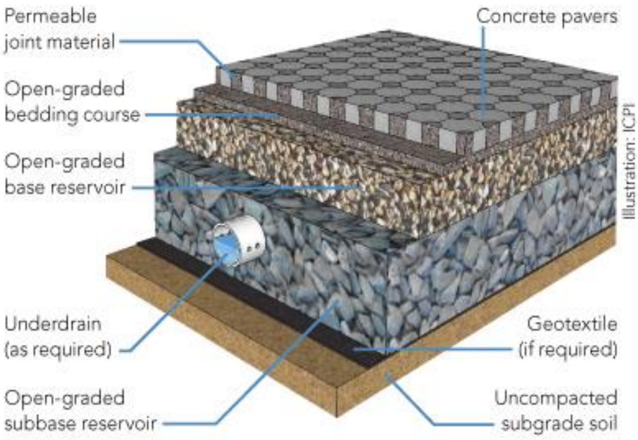

Figure 10 Design of permeable pavement with concrete pavement units (FARLEY, 2016)

In the manual by (Myhr, 2013), it is made a proposal based on Statens Vegvesen Håndbok 018 on how to build the permeable pavement. This is now Statens Vegvesen Håndbok – N200.

- Concrete pavement units, thickness 8-10 cm
- Bedding layer, thickness 3 cm, crushed rock 2/8 or nature sand
- Base layer, thickness 10-15 cm, crushed rock 4/32 mm
- Subbase layer, crushed rock 20/120

The thickness given is just some indicative values. The thickness will be dependent on soil conditions, traffic loading, the desired water storage capacity in the structure and freezing conditions. If coarser material is used when you go downwards in the construction, the permeable joint material will be the design value for the drainage capacity. Further it is recommended to use the same type of material in the bedding layer as for the permeable joint material. Geotextile is only used as separation between finer and coarser material in the bottom of the trough. (Myhr, 2013)

PICP manual (Smith, 2006)

The base and the subbase material is recommended to be a hard rock with:

- 90 % fractured faces
- Los Angeles Abrasion of < 40
- Minimum effective porosity of 0.32
- Design CBR 80 % or more

Fractured face is defined as “angular, rough, or broken surface of an aggregate particle created by crushing, by other artificial means, or by nature” (Interactive, 2007)

Los Angeles Abrasion test is a test to measure the degradation of a course aggregate sample. The sample is placed in a drum together with steel spheres. When the drum is rotating the sample is prone to abrasion by contact with the rest of the samples and the steel spheres. The aggregate that is broken to smaller parts is expressed as a percentage of the total mass. So high L.A. Abrasion loss indicates weaker aggregate, low L.A. Abrasion loss indicates tougher aggregate. Some rock types to compare with. Granite has typical L.A. Abrasion of 27-49 and Limestone has typical L.A. Abrasion of 19-30. (Interactive, 2011)

Porosity = volume of voids/total volume of the base

California Bearing Ratio (CBR) is a measure of strength. It compares the bearing capacity of a material with a well graded crushed stone. A high quality crushed stone should have a CBR of 100 %. (Interactive, 2009)

Stones that are commonly used for bedding material are ASTM No.8, for base layer ASTM No.57 and for the subbase layer ASTM No.2. (Smith, 2006)

Recommendations (Beeldens and Vijverman, 2015)

Base and subbase from unbound aggregates (0/32 – 2/32 – 2/40 – 2/20)
Requirements:
- fines (< 0.063 mm) < 3 %
- fraction < 2 mm < 25 %
- if recycled concrete aggregates, no aggregates < 2 mm

Base layer from bound material: porous lean concrete
- permeability: $4*10^{-4}$ m/s
- compressive strength on cores Ø 113 mm: average > 13 MPa, individual > 10 MPa

Bedding course from unbound aggregates (0/6.3 mm – 2/6.3 mm)
- fines (< 0.063 mm) < 3 %
- maximum grain size: 6.3 or 8 mm
- LA < 20 – MDV < 15

Recommendations for aggregate size PICP (Eisenberg et al.)

Aggregate subbase

Table 1 Recommendation for aggregate size, subbase layer

Sieve Size, mm (in.)	Percent Passing (%)
75 (3)	100
63 (2.5)	90–100
50 (2)	35–70
37.5 (1.5)	0–15
19 (0.75)	0–5

Aggregate base

Table 2 Recommendation for aggregate size, base layer

Sieve Size, mm (in.)	Percent Passing (%)
37.5 (1.5)	100
25(1)	95–100
12.5 (0.5)	25–60
4.75 (No. 4)	0–10
2.36 (No. 8)	0–5

Aggregate bedding layer

Table 3 Recommendation for aggregate size, bedding layer

Sieve Size, mm (in.)	Percent Passing (%)
12.5 (0.5)	100
9.5 (0.4)	85–100
4.75 (No. 4)	10–30
2.36 (No. 8)	0–10
1.16 (No. 16)	0–5

Summary of the recommended grain sizes

In Table 4 it is a comparison of the different grain size recommended for each layer.

Table 4 Comparison of different recommendations for grain size in the different layers of the structure

Layers/source of information	From (Myhr, 2013) with design values taken from Statens Vegvesen Håndbok 018	From (Smith, 2006) PICP Manual 3rd edition	From (Beeldens and Vijverman, 2015)	From (Eisenberg et al.)
Joint filling material	Recommended to be the same as bedding material	Recommended to be the same as bedding material	No data	No data
Bedding layer	Smallest/biggest grain size passing the sieve 2/8 mm	Range of 1.16 – 12.5 mm in the grain distribution curve	Range of 0/6.3 mm or 2/6.3 mm	Range of 1.16/12.5 mm in the grain distribution curve
Base layer	Smallest/biggest grain size passing the sieve 4/32 mm	Range of 2.36 – 37.5 mm in the grain distribution curve	Range of 0/32 mm, 2/32 mm, 2/40mm or 2/20mm	Range of 2.36/37.5 mm in the grain distribution curve
Subbase layer	Smallest/biggest grain size passing the sieve 20/120 mm	Range of 19 – 75 mm in the grain distribution curve	Range of 0/32 mm, 2/32 mm, 2/40 mm or 2/20 mm	Range of 19/75 mm in the grain distribution curve

The recommendations for grain size are very similar to each other. Smallest grain size in the joint filing material and the bedding layer, bigger grain size for the base and subbase layer. The most important is to have an increasing grain size from the joint filling material to the subbase layer. This ensure that the pavement clog in the first infiltrating layer and make the maintenance work much easier and less expensive than if you had to change the whole pavement structure.

With bigger grain size at almost the same size, you achieve more porosity and volume to store water in the deeper layers.

3.5 Infiltration rates of permeable pavement systems

3.5.1 Infiltration rates of the pavement

The infiltration rate of permeable interlocking concrete pavement is dependent on the infiltration rate of the joint filling material. This is usually the limiting factor, because the material in the base layer and subbase layer are often coarser with higher infiltration rates (Smith, 2006). A common mistake is to assume that the percent of the open surface area is the same as the amount of perviousness. The perviousness of the pavement is decided by the infiltration rate of the joint filling material. And the other materials in the structure, but usually the limiting infiltration rate would be found in the joint filling material. As an example 30 % open surface area is not the same as 30 % perviousness and 70 % imperviousness. (Eisenberg et al.)

If open-graded crushed aggregate is used as joint filling material, an infiltration rate of 10^{-3} m/s could be expected. Permeable pavements are prone to clogging so an infiltration rate of 2.1×10^{-5} m/s or 210 l/s/ha could be used as design surface infiltration rate for a lifetime of 20 year. (Smith, 2006)

The Belgian Road Research Centre (BRRC) has been looking at surface permeability over the whole service life of permeable pavements since 2003. The values seem so far to be a permeability of $5.4 \cdot 10^{-5}$ m/s for newly constructed areas, and $2.7 \cdot 10^{-5}$ m/s for areas with longer existence. This is without maintenance on the surface area. (Beeldens and Vijverman, 2015)

The permeability is not usually the limiting design factor for the permeable pavements. They will most likely have an infiltration rate of 2.4 mm/s, which is more than 100 times the permeability of most types of sand.(Korkealaakso et al., 2014)

In a study by (Kumar et al., 2016) permeable concrete, permeable asphalt and permeable pavers where tested over a four year period at a parking lot. When they were installed the infiltration rate was 38.2 mm/s for concrete, 31.1 mm/s for asphalt, 25.4 mm/s for pavers. After the fourth year the infiltration rates were as follows 6.0 mm/s, 9.1 mm/s and 3.8 mm/s.

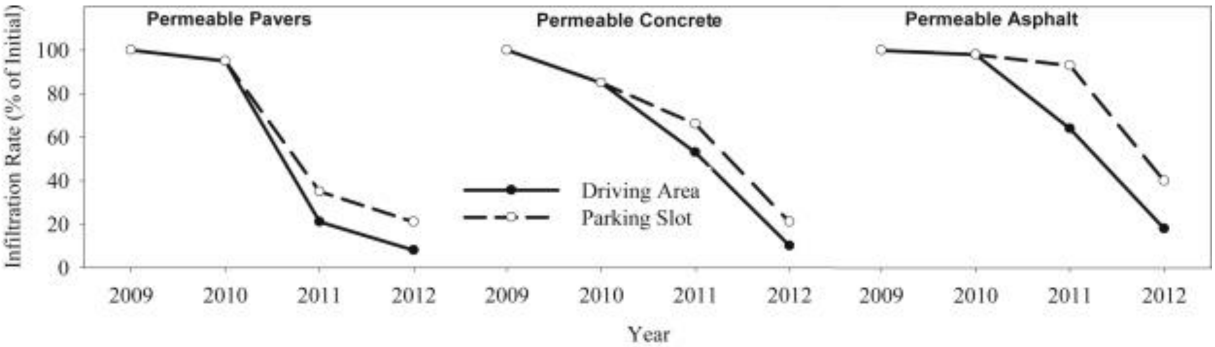

Figure 11 Percent of initial infiltration rate for three permeable pavements over a four year period (Kumar et al., 2016)

Figure 11 is showing the infiltration rate for the three areas as percentage of the initial infiltration rate for each year since the construction. It decreases drastically over the four years, but even if it is down to less than 20 % of initial infiltration rate, it has not been detected any runoff from the area. The capacity of the pavements are still higher than most of the rainstorm events that occur in the area. (Kumar et al., 2016)

From another study by (Bean et al., 2004) sixteen sites with concrete grid pavement were tested and showed a mean infiltration rate of 7.1 cm/h. Fourteen of the sites were maintained and the mean infiltration rate increased to 11.8 cm/h. Eleven permeable interlocking concrete pavement sites were tested and six out of them had infiltration rates higher than 150 cm/h. The average infiltration rates for the sites tested free of loose fines were 2000 cm/h. There was registered a significantly decrease in the infiltration rate for pavements near disturbed soils with fines. The infiltration rate was then only to 60 cm/h.

Table 5 Infiltration rates from different articles for both newly constructed pavements and pavements which have been used for some years.

	Infiltration rate (new) [mm/s]	**Infiltration rate (old) [mm/s]**
(Smith, 2006)	1	2.1×10^{-2}
(Beeldens and Vijverman, 2015)	5.4×10^{-2}	2.7×10^{-2}
(Korkealaakso et al., 2014)	2.4 (moderate estimate)	
(Kumar et al., 2016)	38.2 – 25.4	9.1 – 3.8
(Bean et al., 2004)		2.0×10^{-2} (CGP) and 5.56 (PICP)

In the articles the measured infiltration rates varies from the different test sites. Table 5 show the great variation among the pavements tested. The infiltration rate of the pavements decreases rapidly with time. The surroundings and the use of the pavements are factors with great influence on how fast they clog. Even if the infiltration rate after a few years is reduced from the time of construction, the capacity is in many cases more than sufficient to handle storms and infiltrate rainwater.

To give some perspective to the numbers a 5 minutes rainfall with 50 year return period have a predicted precipitation of 221.7 l/s*ha for the area around Voll, Moholt and Tyholt in Trondheim (Trondheim kommune, 2015). This is equal to $2.217 * 10^{-2}$ mm/s. This means that most of the pavements which start to clog in Table 5 have an infiltration rate that is still good enough for this amount of rainfall.

3.5.2 Ground conditions for infiltration:

When the permeability of the native soil is found before constructing the permeable pavement, the value is recommended to be considered as an approximation and not as an absolute value. During construction there is likely to be a loss of the soils conductivity and porosity. A safety factor of 2 is recommended for the measured soil infiltration rate (Smith, 2006). In the Minnesota Stormwater Manual referred to by (Korkealaakso et al., 2014) it is recommended with a safety factor of 2.5 for the soil infiltration rate, unless the soils infiltration capacity is measured after compaction.

The permeable pavement will in most of the occasions have higher infiltration capacity compared to the soil. Clay soil (CL) will typically have an infiltration rate of 10^{-9} m/s and silty sand (SM) could have an infiltration rate of 10^{-7} m/s. The open graded aggregate used for the joint filling material could have as much as 10^{-3} m/s when constructed. (Smith, 2006)

The permeability in soils is depending on what type of soil it is, the separation grade and compaction grade (NGU, 2016). Figure 12 shows the permeability (hydraulic conductivity) for different types of soils.

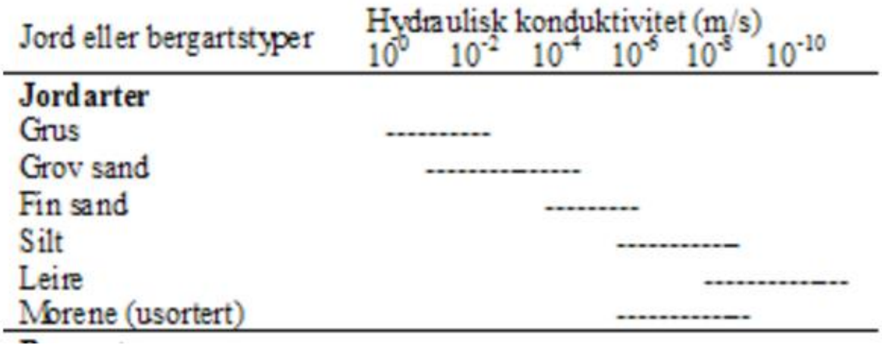

Figure 12 Hydraulic conductivity of different types of soils (NGU, 2016)

From (Becker, 2015) it is referred to (Paus et al., 2015) that the hydraulic conductivity of a soil should be more than 10 cm/h to be referred to as good infiltration capacity. From Figure 12 everything from fine sand to gravel will be in that range. An article referred to in (Korkealaakso et al., 2014) recommend the soils infiltration rate to be in the range of 0.25 – 25.4 cm/h, dependent on how much water that are discharged through another mechanism.

In (Korkealaakso et al., 2014) it is referred to (Ferguson, 2005) which says that to make sure the capacity to infiltrate water to the ground is maintained, it is recommended with a minimum distance of 60 cm from the bottom of the pavement structure to the highest ground water level or bedrock bottom.

3.6 Design methods

PICP manual (Smith, 2006)

You can design the base storage area in two ways.

1) Compute the minimum depth of the base layer given the area of the permeable pavement. (Minimum depth method)
2) Compute the minimum area of the permeable pavement given the design depth of the base layer. (Minimum area method)

The most common will be to use the minimum depth method. Very often you have a fixed area to use for the permeable pavement. Then it is more convenient to find the depth to store your infiltrated runoff.

Minimum depth method (Smith, 2006)

1) You chose the rainfall you want to design for (P) [m]. Then calculate the runoff from the contributing area connected to the permeable pavement. (ΔQ_c) [m]

2) Find the depth of the aggregates base. (d_p) [m]

$$d_p = \frac{\Delta Q_c * R + P - fT}{V_r}$$

Equation 1

d_p = Depth of the crushed stone base [m]
ΔQ_c = runoff from contributing area for the chosen design storm [m]
R = A_c/A_p
A_c = contributing area for runoff
A_p = permeable pavement area
P = design rainfall [m]
f = the infiltration rate of the soil underneath the pavement [m/h] adjusted for compaction before and after construction, clogging from sediments and so on.

T = effective filling time of the base (typical 2 hours)
V_r = The void ratio (porosity, volume of voids/total volume of the base)
d_{max} = maximum allowable depth of aggregate base and subbase

3) Check if d_p is less or equal to d_{max} and if d_p > 2m above the seasonal ground water table.

$$d_{max} = f * \frac{T_s}{V_r}$$

Equation 2

T_s = maximum allowable storage time (72 hr)

The article by (Korkealaakso et al., 2014)

The depth required to hold the needed amount of stormwater is calculated with the formula from (Korkealaakso et al., 2014). The water storage depth is the amount of rain to be stored in the structure.

$$\text{Thickness of reservoir material} = \frac{\text{water storage depth}}{\frac{void\ volume}{total\ volume}}$$

Equation 3

If you want the water to stay in the structure for a certain time (ponding time), the formula can be written as

$$\text{Thickness of reservoir material} = \frac{\frac{Discharge}{ponding\ time}}{\frac{void\ volume}{total\ volume}}$$

Equation 4

Discharge is all the infiltration and pipe discharge per hour. Ponding time is chosen as the percentage of time the water is allowed to stay in the structure. It could be 10 % of the time and with storm events every 5 days, the allowed ponding time will be 0.5 days. Some assumptions with this approach are that infiltration during a storm is not accounted for and also the reservoir is assumed to be full after every rainfall event which is also conservative. (Korkealaakso et al., 2014)

Acceptable detention rate for reservoir in the pavement structure is from 12 – 72 hours according to (Hansen, 2008b) referred to by (Korkealaakso et al., 2014)

3.7 Clogging of permeable pavement

3.7.1 General facts about clogging

Clogging of permeable pavements might be the biggest concern with this type of pavements. If you want to use permeable pavement as an option to standard piped system with conventional drains, they have to perform good in many years and you have to be able to rely on that they don’t fail to perform their task. In countries with colder climates and freezing conditions, winter maintenance as plowing together with sand and salt applied to the pavement cause extra stress. The other issue which makes developers to hesitate before applying permeable pavement instead or in combination with regular storm water measures is the behavior of the pavement through the winter with snow and frozen ground.

From (Scholz and Grabowlecki, 2007) the main course of clogging is:

- Sediments from surroundings being pressed into the porous pavement by traffic.
- Sediments coming with the water will infiltrate and eventually clog the pavement.
- Shear stress from vehicles making the pores collapse. More of a problem for porous asphalt and concrete than concrete pavement units with joint fillings.

Permeable pavements are good in housing estate roads with low traffic and vehicle load. The problem is clogging. When building new housing areas it is common to build the road before constructing the houses. Then the chance for clogging the pavement is very high. It is suggested to either build the permeable pavement as it is supposed to be and clean it when everything is finished, or build a thin asphalt layer on top of the permeable subbase layer which you later remove and replace with the permeable layer.

Since clogging is having a significant role for how the permeable pavement preform, it is interesting to have a closer look at what conditions and type of material that will cause clogging. (Pratt et al., 1995) is referred to in the article by (Yong et al., 2013) and in this study it is argued that the mass of the particles entering the system is the most important factor of physical clogging. Another study by (Balades et al., 1995) also referred to in the article by (Yong et al., 2013) suggest that sediment size plays a role, since smaller particles will trap larger particles and then the rate of clogging will increase.

In the article by (Yong et al., 2013) it is found that the porous asphalt clogged in the surface layer, and the Hydrapave pavement clogged in the geotextile underneath the permeable concrete bricks. The failure of the porous asphalt will show immediately, but for the failure in the geotextile to be observed, the water needs to rise above the porous concrete bricks. Another interesting discovery by (Yong et al., 2013) was that it showed significant difference in infiltration of total amount of water before clogging when different methods for applying the water where used. By simulating the drying/wetting sequences occurring for rainfall in natural climate, instead of constant continuous flow the amount of extra water infiltrated through the porous pavement was 33 % and through the porous concrete block pavement it was as much as 48 %.

(Borgwardt, 2006) referred to in the article by (Lucke and Beecham, 2011) predicted that the PICP pavements infiltration rates will be reduced to 18 % of the infiltration of a new pavement after approximately 10 years of service.

After looking at the sediment accumulation in the different layers of a permeable interlocking concrete pavement at an parking area in Australia, (Lucke and Beecham, 2011) found that most sediments in mass (kg) are accumulated in the aggregates, then the pavement and then the geotextile. This finding was the same regardless of the pavement blocking. Because of the great storage capacity in the aggregates the sediments in this layer will not cause any problem. The sediments in the geotextile are also considered to not be the cause of clogging. The clogging in the top layer surface is the one that is critical for the infiltration capacity of the pavement.

3.7.2 Lab and field test results

There is done some research on clogging and how it affects the infiltration rates of permeable pavements. In the article by (Illgen et al., 2007) there is done several lab scale tests. In Figure 13 the infiltration rate and the runoff rate is measured for a concrete block pavement with joints of 4 mm and a slope of 2.5 % over a 20 min with simulated rainfall. It is interesting to see that the infiltration rate increase even when runoff occur.

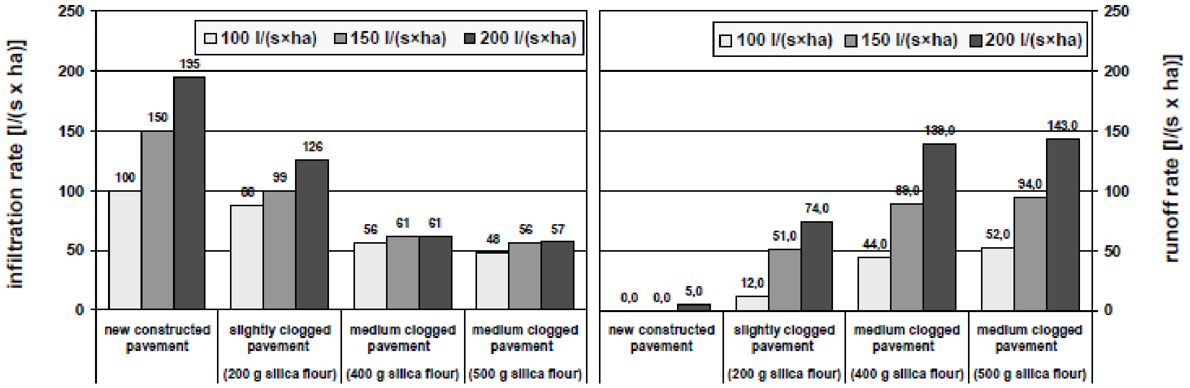

Figure 13 Infiltration rate and runoff rate with varying conditions of clogging and rainfall intensity (Illgen et al., 2007)

The pavements were tested with different slopes of 2.5 %, 5.0 % and 7.5 %. The results showed a slightly difference in infiltration rates when the rainfall intensity and the level of clogging increased (Illgen et al., 2007). The slope is not that relevant when it comes to the infiltration rate of the permeable pavement.

3.8 Water quality improvements

There have been studies showing great water quality improvements from permeable pavements (Eisenberg et al.). This is usually not the reason itself for why permeable pavement is chosen, but it comes as a great advantage compared to regular pavements. Some of the water quality improvements from (Eisenberg et al.) is listed below.

- Pollutant concentrations
  Decreases the pollutants in the water like heavy metals, sediments, oil, TSS and some nutrients.
- Pollutant loads
  Because of the infiltration the pollutant load from runoff compared to conventional pavement is much lower.
- Thermal
  Reduce the temperature of the water when it infiltrates through the pavement structure.
- Buffering
  Can buffer the pH of acidic rainfall due to presence of calcium carbonate and magnesium carbonate in the base layers.
- Phosphorous
  Some studies show sign of phosphorous removal, while others have not. If engineered materials designed to remove phosphorous is used, greater phosphorous removal can be achieved.
- Nitrogen
  Studies have showed that nitrification of ammonia to nitrate can occur. Denitrification processes can also be encouraged.

- Total suspended solids
  Particles will not travel through the pavement unless they are very small.
- Salt
  The salt is not removed or threated in any kind of the permeable pavement, but less salt is usually needed on permeable pavements.

In a study done by (Brattebo and Booth, 2003) there is a big difference between the infiltrated water and the surface runoff. Toxic concentrations where measured in 97% of the samples from the runoff water and in the infiltrating water 31 out of 36 samples the concentration fell below toxic levels.

3.9 Winter climate conditions

In cold climates where minus degrees are expected in longer periods of the year, the damage caused by water freezing in the structure would be a problem. When water freeze it expands in volume by 10 percent (Bruce, 2005). If the pressure from the ice exceeds the pressure of the structure above, the pavement is prone to heaving movement and the particles get pushed away from each other. When the ice melts, the particles are not supported by each other or the ice, and deformation is very likely.(Bruce, 2005)

Thicker pavement will protect the structure from frost damage by insulating the layers underneath and more weight will better resist the pressure from ice freezing below. Thicker pavement is more expensive and has to be evaluated together with the risk of failure.(Bruce, 2005)

Materials that are non-frost-susceptible can be used to protect the subgrade from freezing. This is costly because you need thicker layers. Another solution is to allow the subgrade to freeze.(Bruce, 2005)

Fine grained soil with moisture is especially prone to clogging. The water freezes in the small pores and is supplied by more water from the surrounding pores which leads to a growing ice crystal. The same thing is not likely to happen for courser material with higher porosity. The water will then freeze in isolated crystals and not grow together and cause frost heaving.(Bruce, 2005)

Reservoirs where the water is draining fast because of good infiltration capacity in the subgrade or drainage pipes, will not hold the water long enough for it to freeze. If you are building a permeable pavement in an area where freezing will occur and the infiltration through the subgrade is not sufficient to drain fast enough to prevent the water from freezing. A possibility could be to remove the water by drainage pipes even if the storage capacity is good enough, just to prevent frost heaving. You might change the balance of the unit processes from mostly infiltration to more peak flow attenuation and detention.(Bruce, 2005)

According to (Leming et al., 2007) referred to in (Korkealaakso et al., 2014), to prevent frost heave in areas where cold temperatures and freezing will occur, the rule of thumb for how deep the structure of the pavement should be is half or two-thirds of the frozen depth. This might be more than what is required for the storage capacity.

In (Korkealaakso et al., 2014) it is referred to (Ferguson, 2005) and (Hansen, 2008b) when it is recommended that frost penetration in some parts of the pavement is allowed. The design thickness of the pavement and subbase layers with regards to frost depth should be about 65 % of the frost depth for a ten year freezing event.

In the study done by (Rohne and Lebens, 2009) temperature profiles measured from permeable pavements where compared with temperature profiles from conventional concrete pavements. For the permeable pavements 60 % fewer freeze-thaw cycles where observed over a three year period compared to conventional pavements with the same thickness. One freezing cycle is defined in the study as fall and rise above 0 °C. They believe the difference in freeze-thaw cycles could come from the insulating effect of the air inside the voids in the permeable pavement. Further they discovered that the permeable pavements had 4 °C higher temperature in the subgrade during winter conditions compared to the impervious pavement. This resulted in a maximum frost depth of 457 mm shallower than for the impermeable pavement.

The article by (Guthrie et al., 2013) from the book (2013) is hoping to see if permeable pavement are more resisting to freezing than conventional impermeable pavement. In the conclusion it is said that the surface freezing temperature of the two types of pavement are similar. When it comes to subsurface freezing it is expected that the pervious pavement provide greater resistance against freezing than conventional impermeable pavement. The air temperature for which the temperature in the subsurface reach 0 °C, is colder for the pervious pavement than for the impermeable pavement. This means that permeable pavement have greater resistance against freezing.

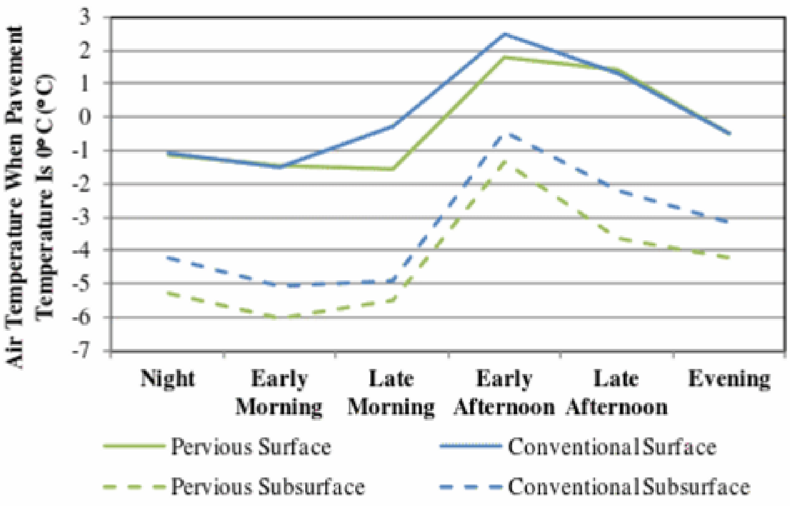

Figure 14 The air temperature when pavement temperature is 0 °C (Guthrie et al., 2013)

Also the studies by (Fischer and Zubeck, 2013) indicates that permeable pavements can have a shallower freezing depth than impermeable pavements. The difference between the two types of pavement is seen when the permeable pavement structure is drained. When water is within the structure the results are almost the same. More research is needed on this topic.

Permeable pavement is often used at parking areas and low traffic roads. Compacted snow will cover the pavement during the cold periods of winter. During freezing and melting cycles ice will form on top of the pavement and prevent water from reaching the pavement and infiltrate. For situations like this, sandtrap basins and drain grates to lead the water into the reservoir in the permeable pavement could be a solution to remove the melted water.

During winter (December – February) it is registered a much lower intensity for the rainfall. For the data registered at Gardermoen it is used a correction factor of 0.6 to get the daily rainfall intensity during winter if the IVF curves are based on yearly values or from the summer months (Dyrrdal and Førland, 2016). So even if the capacity of the pavement could be reduced in the winter, the rainfall intensity could also be reduced compared to summer.

4 Presentation of the test facility at Skjæveland

4.1 Brief introduction of Skjæveland Group

Skjæveland Group consists of the three companies Skjæveland Cementstøperi, Multiblokk and Storm Aqua. Together they can deliver complete solutions for stormwater management. They want to be a leading company when it comes to solutions for taking care of storm water. As a part of that, a test field was built in Multiblokk's drive way in Sandnes, at the corresponding parking area and the access road.

Skjæveland Cementstøperi manufactures concrete products like pipes, pre-constructed manholes, weights for loading and other specialized products. Multiblokk delivers concrete pavingstones, slabs, blocks, kerbstones and those kinds of products to both private and public costumers. Storm Aqua is a center of excellence for stormwater solutions. Multiblokk and Skjæveland Cementstøperi is the one investing in the test field and Storm Aqua is the one operating it. Together they want to use the results from the test area and find new innovative solutions to handle surface water. (Skjæveland Gruppen, 2015/2016)

The test field serve four purposes (Skjæveland, 2015):

1) Measure, gather documentation and present infiltration parameters for permeable pavements, infiltration chamber and pipes.

2) Show that an area can be built without any runoff of storm water to downstream constructions.

3) Improve the foundation for future regulations on how to handle stormwater.

4) Build a test area for new products and services for local stormwater handling.

4.2 Presentation of the test area

The construction of the area started in October 2014 and was finished in December 2014. In 2015 from January until the measurments started in May, instrumentations was installed. The overall size of the area is 1600m^2 and it costed approximately € 600 000.

To the left in Figure 15 the test area is pictured on top of an image of Multiblokk's drive way in Vagleskogveien 10. To the right in Figure 15 the test field is pictured without the surrounding area and it is easier to differentiate the four test areas.

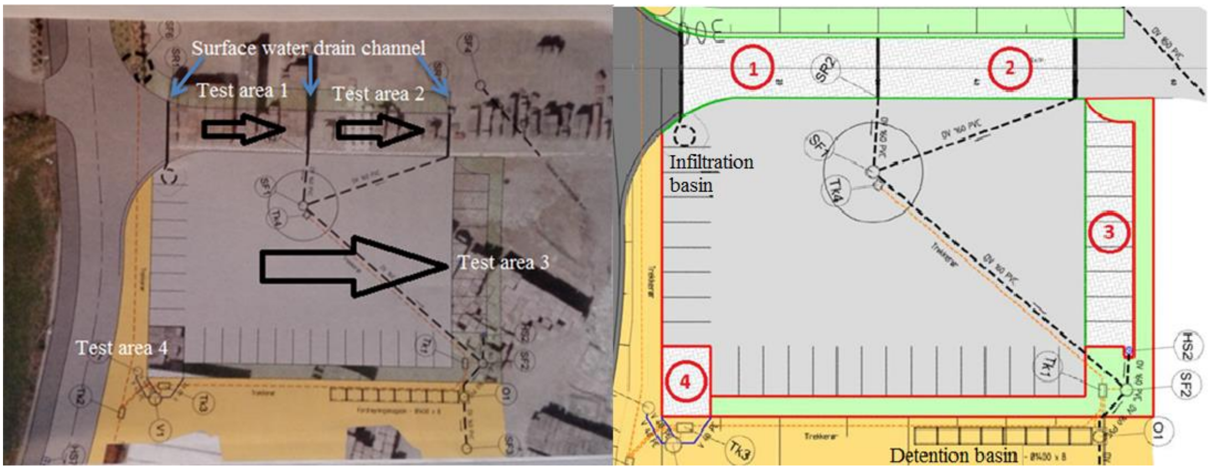

Figure 15 Left: Test area in Vagleskogveien 10. The big arrows point out the direction of water flow, adjusted from (Skjæveland Gruppen, 2015/2016). Right: Test area with the separate parts in more detail, adjusted from (Møller-Pedersen, 2015)

In the next chapter the test area will be presented in more detail. To help with the understanding a flowchart is made to give a more figuratively presentation (Figure 16).

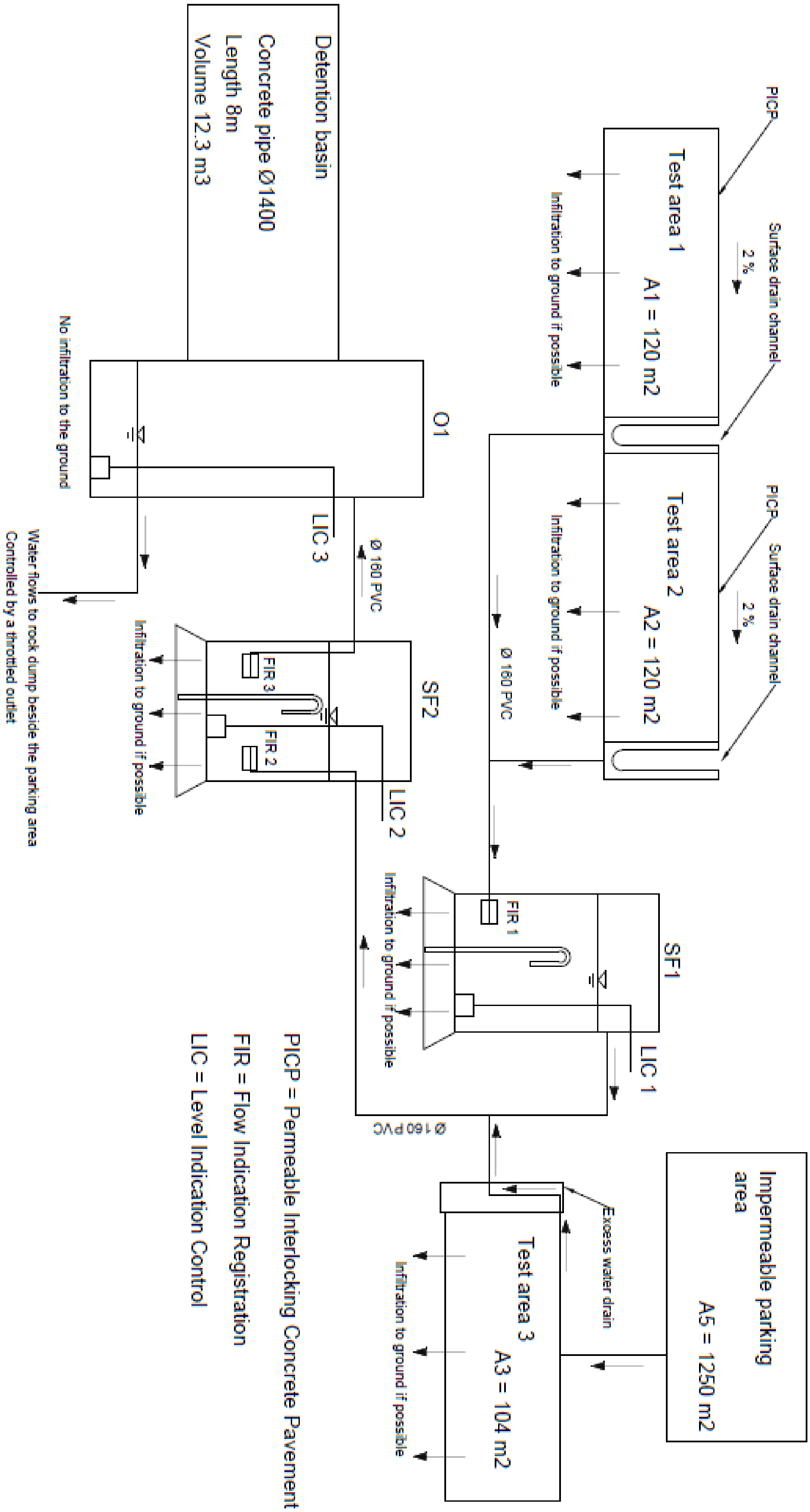

Figure 16 Flowchart of the test facility in Sandnes

4.3 Test area 1 and 2

This is the access road to Multiblokk's industrial area. All traffic in and out is driving on this road, so it is prone to heavy traffic from trucks and other vehicles. The two areas are each 6m wide and 20m long which gives an area of 120m^2 and in total 240m^2. The two areas are separated with a surface water drain channel in the middle and one at the lower side of Test area 2 to prevent water from leaving. They are there to collect water that does not infiltrate through the permeable concrete blocks used for paving, and transport it to a sandtrap/infiltration basin. At the sandtrap/infiltration basin, the water inflow is measured to register how much water that is not infiltrating through the concrete paving units. The longitudinal slope of the road sections is two degrees. (Møller-Pedersen, 2015)

On the upper side of Test area 1 there is also a drain channel for surface water to prevent water from entering the test area. This is led to a separate infiltration basin with no instrumentations.

Figure 17 is from test area 1 and 2 and how they looked like in January 2016 when the picture was taken. The picture at the left is test area 2 closest to the camera. The picture in the middle is looking down towards test area 2 from the drainage pipe that separates test area 1 and 2. The picture to the right shows one of the surface drain channels that collects the excess water.

Figure 17 Test area 1 and 2, Skjæveland (Photo: Jens H. Trandem)

Test area 1 and 2 are designed to resist heavy loads from trucks. On average a daily payload of 1000 ton is passing the road section. A certain design is chosen to achieve maximum stability. The thickness of the road section is 780 mm. The structure of the road design is pictured in Figure 18. On top there is Multiloc Dren (100 mm) which is the permeable interlocking concrete pavement from Multiblokk. Underneath there is a bedding layer (30 mm) to place the concrete stones. It is used crushed rock with grainsize 2-12mm where all finer material is removed. The base layer (150 mm) is made of crushed rock with grainsize 4-32 mm and the subbase layer (500 mm) is made of crushed rock with grainsize 20-120 mm. At 280 mm and 480 mm from the surface it is placed a geonet to help distribute the heavy load. Further to increase stability, there are concrete walls founded in the subbase on each side of the road to lock the permeable concrete units.(Møller-Pedersen, 2015)

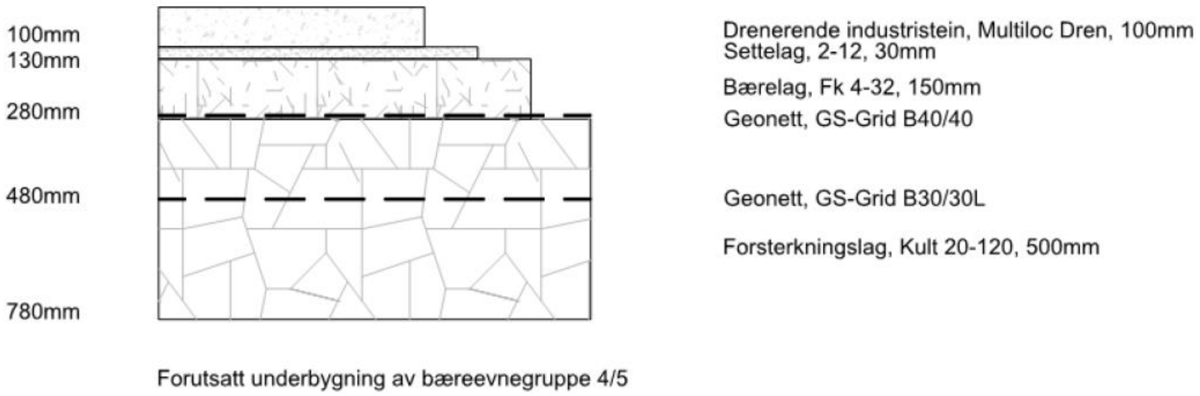

Figure 18 Design of test area 1 and 2 (Møller-Pedersen, 2015)

4.4 Test area 3

Test area 3 is 4.6 m wide and 22.6 m long and covered with permeable concrete block pavements. This gives a permeable area of 104 m^2. It is connected to a larger parking area covered with normal concrete block pavements without any infiltration. This larger parking area is 29.5 m wide and 43.2 m long which makes the area close to 1250 m^2. So the total area that generates runoff to test area 3 is approximately 12-13 times bigger than the permeable area.(Møller-Pedersen, 2015)

Figure 19 Parking area with normal concrete block pavement furthest away from camera. Test area 3 closest to the camera. (Photo: Jens H. Trandem)

In the corner of the test area there is a sandtrap/infiltration basin where inflow of water is measured. The drains position is shown in Figure 15, it is marked as HS2. The design can be seen in Figure 20. This is where the water will go if it don't infiltrate through the permeable pavement in test area 3.

Figure 20 Design of drainage for excess water from test area 3 (Photo: Jens H. Trandem)

In Figure 20, the drain is a bit elevated from the permeable pavement. If the capacity is reached, the water will pond on top of the pavement in the curbstone end of the test area. There is some green algae growth between the concrete pavement units which indicates that there has been water ponding on top.

The impermeable area of the parking lot is laid with a 2 percent slope to the area with permeable concrete blocks (Møller-Pedersen, 2015). The transition can be seen in Figure 21. The permeable concrete blocks continue with a slope of 2 percent before it is finished with curbstones. There is no slope in the direction of the drain at the corner of the permeable concrete pavement.

Figure 21 Transition between the impermeable parking area and permeable parking area. (Photo: Jens H. Trandem)

Test area 3 is constructed a bit different from test area 1 and 2. The parking area is not exposed to such high loads as the road, so the thickness of the subbase layer is 200mm thinner. The bedding layer is made out of 2-8 mm crushed rock and not 2-12 mm as for the road section. Test area 1 and 2 where originally made with 2-8 mm crushed rock in the bedding layer, but because of stability problems the first months it was changed to 2-12 mm grain size. Since the subbase layer is significantly thinner for test area 3 and the traffic load is not that high, the only geotextile in the construction is in the very bottom to separate the subbase layer from the native soil. The parking area is built in the same way, but instead of permeable interlocking concrete pavement, an impermeable concrete block pavement is used. (Møller-Pedersen, 2015)

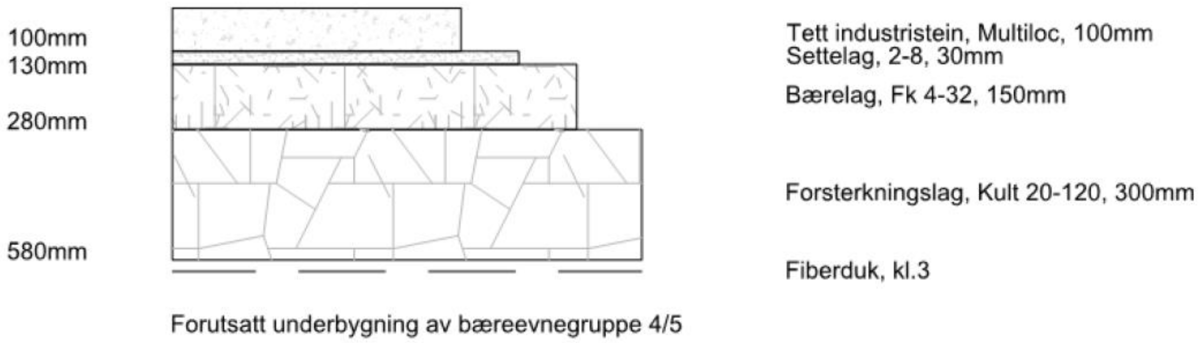

Figure 22 Design of test area 3 (Møller-Pedersen, 2015)

4.5 Test area 4

This area is for demonstration of the infiltration capacity. Rainfall with different intensity can be simulated by a shower system connected to the water supply. The test area is monitored with a camera. This makes it possible to watch either on video or on site. It has been tested for a rainfall of 1000 l/s/ha and the permeable concrete pavement had capacity to absorb all the water. (Møller-Pedersen, 2015)

Figure 23 Test area 4 (Photo: Jens H. Trandem)

4.6 Instrumentation of the test area

4.6.1 Flow meters

Flowmeters are installed to measure the runoff water from test area 1, test area 2, test area 3, and also in the sandtrap/infiltration basins in Vagleskogveien. They are capable of measuring the inflow to the sandtrap/infiltration basin even if no water is infiltrated through the permeable pavement. The capacity of the flow meters to measure this high inflow is compromising the ability of the flow meters to measure low inflows. The expected inflow was estimated before the installation of the flow meters from predicted values of precipitation and performance of the permeable pavement. This resulted in the slightly oversized capacity for the flowmeters which makes them more uncertain when measuring smaller flows and some water might slip through unnoticed. In SF1 there is only one flowmeter installed to measure the inflow, because the infiltration capacity is good and water will never leave SF1. If it should happen, the amount can be estimated from inflow, outflow and infiltrated water in SF2. The flowmeters installed in SF2 is designed for larger flows than the one in SF1. (Personal communication with Per Møller-Pedersen, 2016)

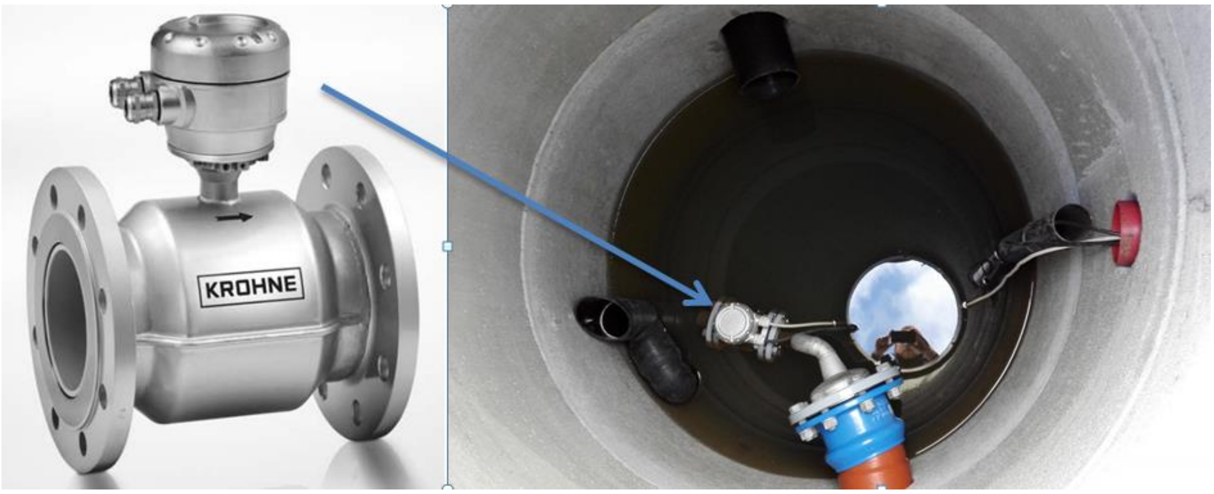

Figure 24 Left: Flow meter installed in the manholes (KROHNE, 2013). Right: The flow meter installed in SF1 (Photo: Jens H. Trandem)

An electromagnetic flow sensor Waterflux3000 from Krohne is used to measure the inflow and outflow from the infiltration manholes. When water flows through the sensor, a voltage is generated and the velocity of the water is found from Equation 5, where all the other factors are known parameters (KROHNE, 2013).

$$U = v * k * B * D$$

Equation 5

U = voltage generated from flow
v = mean flow velocity
k = factor correcting for geometry
B = magnetic field strength
D = inner diameter of flow meter

From the velocity, the flow in volume is calculated. The measuring accuracy is dependent of the flow velocity, see Table 6.

Table 6 Measuring accuracy flow meters (KROHNE, 2013)

	Maximum error
Measured value above 0.5 m/s	0.5 % of the measured value
Measured value below 0.5 m/s	± 2.5 mm/s

It is important that full flow is maintained in the pipe where flow meters are installed. This is why the inlet and the outlet are installed below the water level in the sandtrap/infiltration basins. To the right in Figure 24 and Figure 25 the installation of the flowmeters in SF1 and SF2 is pictured.

4.6.2 Water level sensor

In the sandtrap/infiltration basins there are pressure sensors calculating the water level from the resulting hydrostatic water pressure. The instrumentation used is Signalix SGE – 25. In the datasheet from (aplisens) the accuracy of the pressure sensor is listed. In Table 7 they are reprinted.

Table 7 Accuracy of the pressure sensor for water level measurements. IEC = International Electrotechnical Commission, BFSL = Best Fit Straight Line (aplisens)

Accuracy % FSO acc. to IEC 60770	**0.6%**
Accuracy % FSO acc. to BFSL	0.3 %

The pressure sensor is installed in a smaller pipe inside the manhole. This makes it less affected by turbulence in the sandtrap/infiltration basin which can occur during larger flows of water into the manhole.

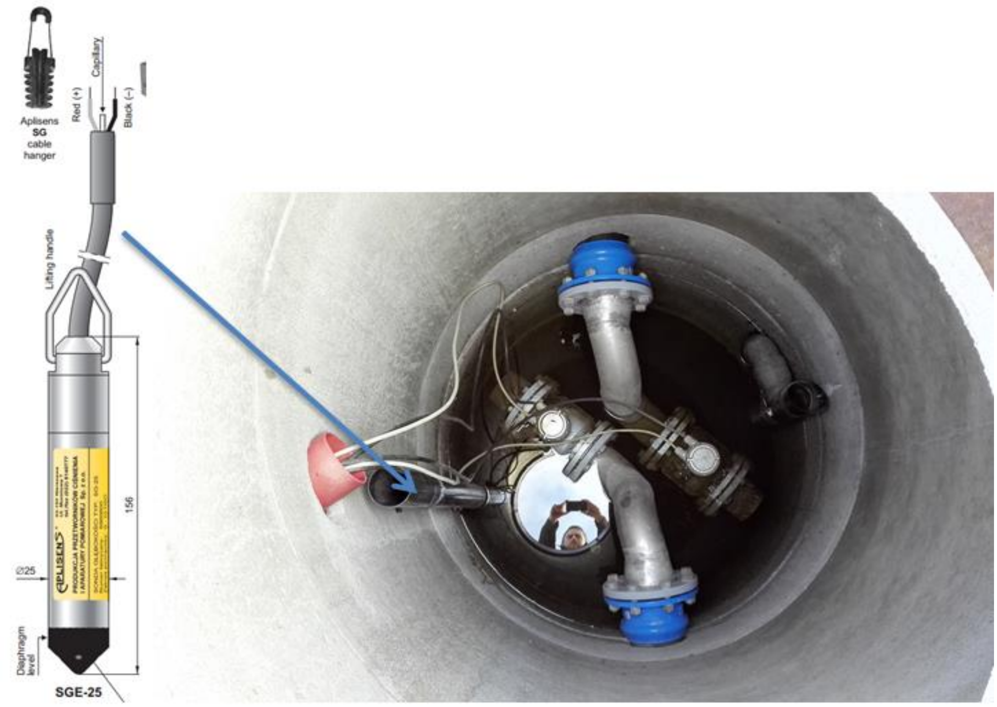

Figure 25 Left: Pressure sensor for water level measurement (aplisens). Right: Pressure sensor installed in SF2 (Photo: Jens H. Trandem).

4.6.3 Weather station

The weather station is built in the end of the detention basin. It is marked with O1 in Figure 15. It measures six parameters: rainfall, relative humidity, temperature, air pressure, wind speed and wind direction. Six temperature sensors are also placed in the ground at different locations. One is placed under the permeable concrete block pavement in test area 3, another one is under the regular concrete pavement and the last one is in the ground close by. Figure 35 is showing the approximately position. There are three more temperature sensors in the ground at 100 mm, 300 mm and 500 mm from top of the ground. They are just placed with present soil on top. (Møller-Pedersen, 2015)

To measure precipitation, relative humidity, temperature, air pressure, wind speed and wind direction, the Vaisala Weather Transmitter WXT520 is used.

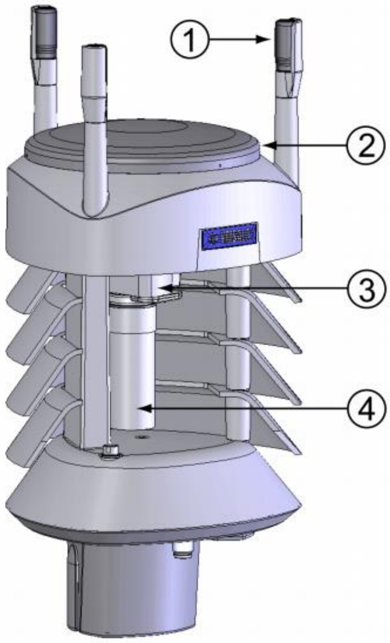

Figure 26 Vaisala Weather Transmitter WTX520, Cut Away View (Vaisala, 2012)

The instrumentations are as follows:

1) Wind transducers
2) Precipitation sensor
3) Pressure sensor inside the PTU module
4) Humidity and temperature sensors inside the PTU module

How the wind speed and direction is measured and found can be checked in the User's Guide by (Vaisala, 2012) together with the description of how data for air pressure, humidity and temperature is collected.

The precipitation is maybe the most important data to measure. How it is done by the Vaisala WTX520 is also in the User's Guide by (Vaisala, 2012), but because it is such an important part of the observations a short description of how it is registered will follow.

The sensor is a steel cover with piezoelectrical sensor on the bottom surface. Individual raindrops are detected upon impact and the volume of the drops can be found. The signal of each drop is then converted to accumulated rainfall. The raindrops will hit the sensor with terminal velocity and the diameter of the drops can be decided from the acoustic signal created by each drop. The acoustic signal is converted to voltages and together with the known surface area of the sensor, the rain is calculated.

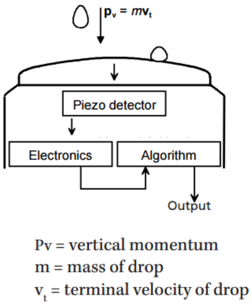

Figure 27 Sensor for measuring rainfall (Vaisala)

The first minute it starts to rain, the intensity is calculated over a period of ten seconds, before it start with fixed one minute steps.

The precipitation sensor has four different modes: Precipitation Start/End mode, Tipping bucket mode, Time mode and Polled mode (Vaisala, 2012). In the PLC system the rainfall intensity [mm/h] is registered every 12 seconds as a mean value of the 60 previous seconds during rainfall.

To know the accuracy of the measurements is important to say something about the quality of the data. If it should be recognized as a research field, it should not be more than 10 % uncertainty with data registered. (Personal conversation with Sveinn T. Thorolfsson, 2016)

Table 8 Accuracy for the parameters measured by the Vaisala Weather Transmitter WXT520 (Vaisala, 2012)

Parameter	Accuracy
Barometric pressure	±0.5 hPa at 0….+30°C ±1 hPa at -52….+60°C
Air temperature	Accuracy at 20°C is ±0.3°C (see User Guide (Vaisala, 2012) for more information)
Wind speed	±3 % at 10 m/s
Wind direction	±3°
Relative humidity	±3 % RH at 0…90 % RH ±5 % RH at 90…100 % RH
Precipitation rainfall	For daily accumulation, better than 5 % (weather dependent) Does not included wind induced error and also spatial variation in the precipitation readings

4.6.4 Programmable Logic Controller

A Programmable Logic Controller (PLC) is used to record the results. It is possible to log on to the system from distances by using a program called TeamViewer. The recorded measurements are made available for everyone through Storm Aqua's website (http://mobile.datafarm.world/stormaqua/main.html).

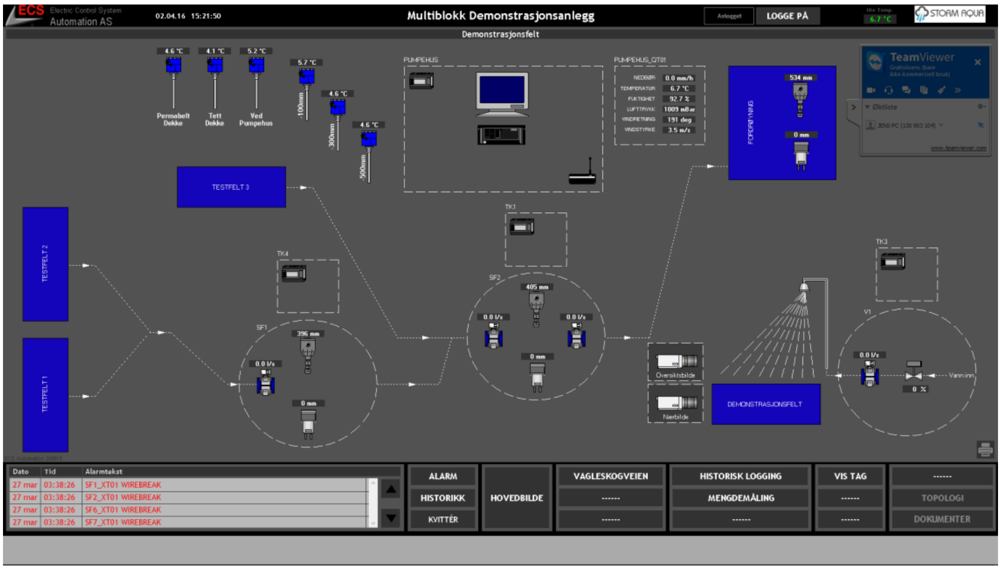

Figure 28 Print screen showing the PLC system

In Figure 28 the setup for the PLC system is pictured. The connection between the test areas and infiltration basins together with the equipment installed in each infiltration basin can be seen from this figure.

4.7 Infiltration chamber

In Figure 15 the position of the two combined infiltration chambers and sandtrap basins are shown as SF1 and SF2. When the infiltration capacities of the pavements are insufficient, the water from test area 1 and 2 will drain to SF1 and the water from test area 3 will drain to SF2. They will work as sandtrap basins with the possibility to infiltrate more water to the ground. The design is pictured in Figure 29, but the measures are not the same as for SF1 and SF2.

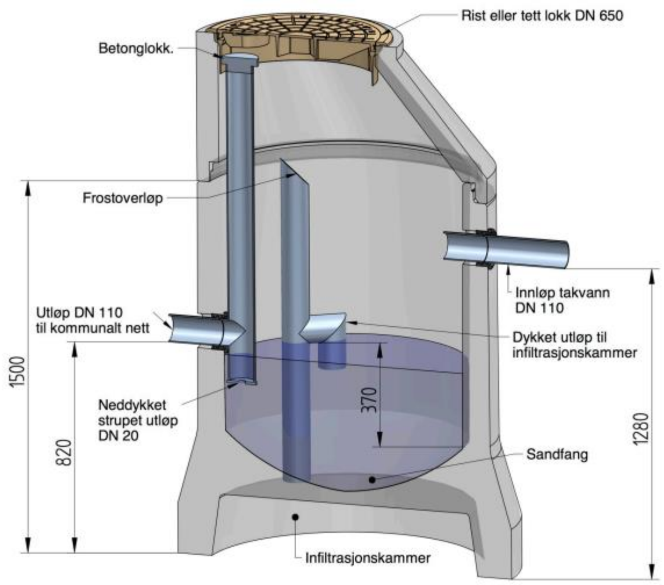

Figure 29 Sandtrap/infiltration basin (Skjæveland, 2015)

The basin is used when the ground conditions allow for infiltration and most of the rainwater are supposed to be infiltrated to the ground. The basin will provide some detention to the system, before the water that is not possible to infiltrate is released, either to the municipal network or in this case the detention basin O1 in Figure 15. The water is coming through the inlet marked as “Innløp takvann DN110” in Figure 29. From there the water level will raise inside the basin and sand particles will fall to the bottom and stay there. When sand has accumulated over time, it is possible to access the basin and empty the sand. The water will go through the dived outlet and to the infiltration chamber underneath the basin. It is used crushed rock for the foundation to provide stability and make infiltration possible. If the infiltration capacity to the ground is reached, water will go through the outlet which leads to the detention basin. This outlet could be controlled and will not allow passage of more water than what it is designed for.

The infiltration capacity of the basins will also be reduced with time. Fine particles will not settle inside the basin but eventually cause clogging in the layer of crushed rock underneath the pavement.

4.8 Detention basin

The detention basin is marked in Figure 15 as “fordrøyningsbasseng”. There is no water from the test area that goes to the municipal stormwater system. Everything that is not infiltrated to the ground will find its way to the manhole in connection with the detention basin. Figure 30 is a picture of the manhole and the detention basin. There is access through the shelter, built to protect the computers registering the measurements.

The diameter of O1 is 1400 mm and the length is 8 m, from figure 3 in (Møller-Pedersen, 2015). This makes the volume of the concrete pipe in Figure 31 equal to 12.3 m^3.

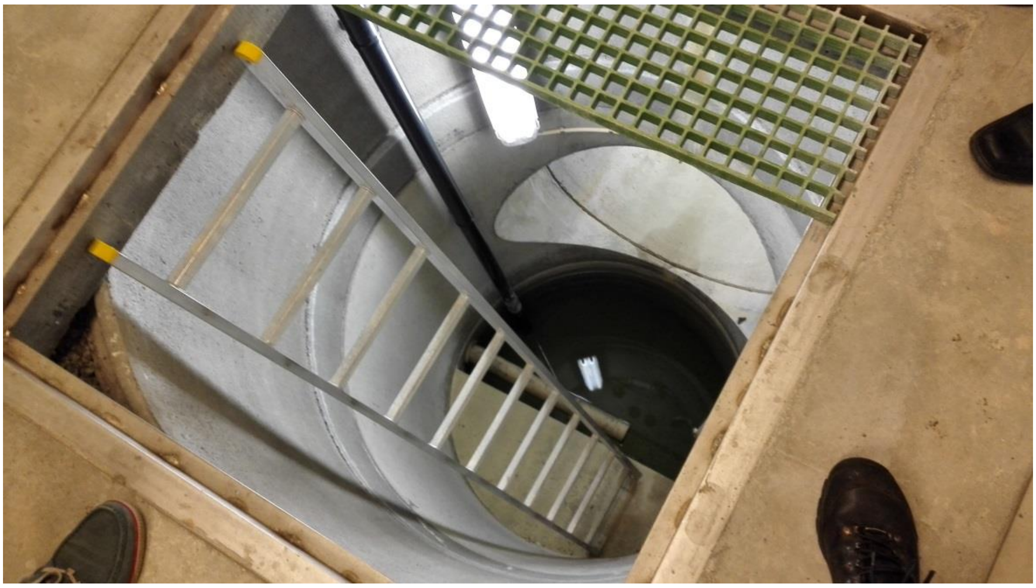

Figure 30 Manhole in connection with the detention basin (Photo: Jens H. Trandem)

The pipe in the middle of the manhole in Figure 30 is a throttled outlet which takes the water to a rock dump a bit lower in the terrain outside. If there is a lot of inflow to the manhole the level will raise and water will enter the detention basin. In Figure 30 the opening at 1 o'clock is the opening to the basin. Figure 31 is from inside the basin looking back at the manhole.

Figure 31 Detention basin (Photo: Jens H. Trandem)

4.9 Vagleskogveien

In the road outside the test facility, Skjæveland is given the opportunity to install four infiltration and sand trap basins. This is a road own by the municipality in Sandnes, but Skjæveland has made an agreement to operate the four basins installed beside the road for five years. In exchange Skjæveland get access to the basins for research. (Merkesdal, 2015)

The basins are installed with flowmeters to measure the inflow and the outflow. Between the infiltration basins there are perforated pipes to increase the infiltration. It is possible to access the infiltration chamber to see if sediments clog the system. All the runoff water from the road is taken care of by these sandtrap/infiltration basins. (Merkesdal, 2015)

The sandtrap/infiltration basins used in Vagleskogveien have the same design as SF1 and SF2. This is illustrated in Figure 29.

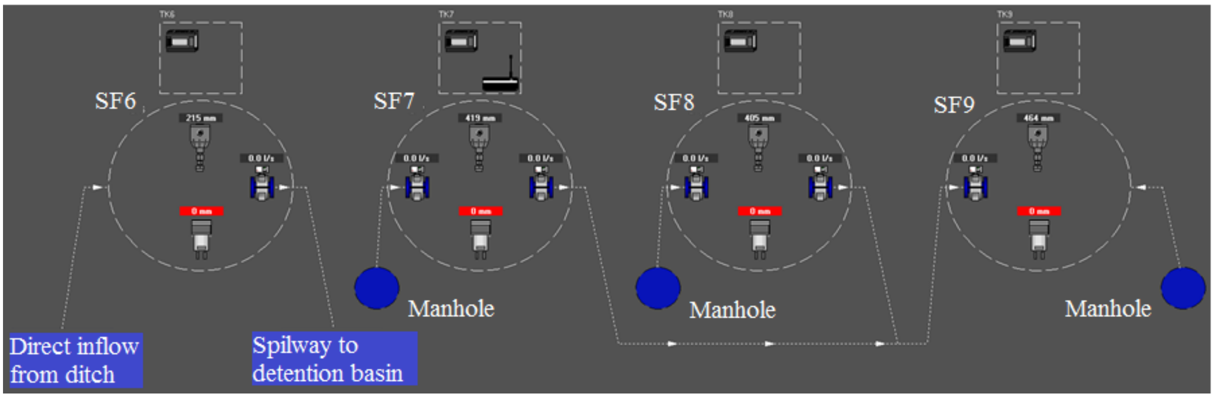

Figure 32 Details of the basins in Vagleskogveien

Figure 32 is an overview of the system in Vagleskogveien. SF6 is pictured to the left and then it follows with SF7, SF8 and SF9. All the basins are equipped with a device to measure the water level and flow meters. SF6 have only a flow meter to measure the outflow and SF9 have only a flow meter to measure the inflow. SF7 and SF8 have both flow meters to measure inflow and outflow.

The spillway from SF6 goes to the rock filling beside the parking area. The blue circles marked as manhole in Figure 32 provides inflow to SF7, SF8 and SF9.

4.10 Ground research

Some ground research where done during the construction to test the infiltration capacity of the native soil. An infiltrometer was used at six different locations and the permeability can be read from Figure 33. The tested locations at the test field are shown in Figure 34.

Location	Ground condition	Infiltration (mm/s)	Permeability (m/s)
1	Gravel	3,3	3,17E-4
2	Soil	0,1	1,03E-5
3	Crushed concrete	0,1	5,55E-6
4	Crushed concrete	0,1	5,72E-6
5	Soil	0,0	0,00E-0
6	Soil	0,0	0,00E-0

Figure 33 Infiltration capacities (Møller-Pedersen, 2015)

Soil with infiltration capacity of more than 10 cm/h can be referred to as good infiltration capacity (Paus et al., 2015). This is the same as $1.16*10^{-5}$ m/s. If we look at Figure 33, the only place tested with an infiltration capacity close to, or more than $1.16*10^{-5}$ m/s are location 1 and 2. The infiltration capacity at location 3 and 4 is half of what we consider as good infiltration capacity.

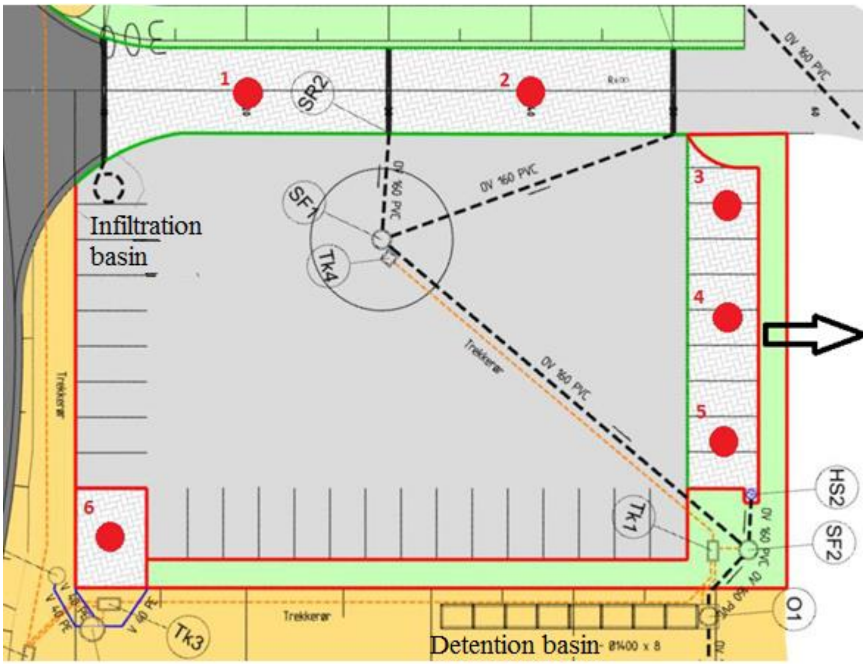

Figure 34 Locations for test of infiltration, adjusted from (Møller-Pedersen, 2015)

From the infiltration tests, the soil underneath the access road is the only place where infiltration is considered to be almost good. When water is not infiltrated to the ground, it will follow the terrain underneath the pavement structure. And drain to the same rock dump where the water from the basin is going. The arrow in Figure 34 is pointing the direction of the slope.

5 Results from test field in Sandnes

The test field in Sandnes started to register data in May 2015. The measurements have been carried out continuously and are still ongoing, but because of failures in the system and maintenance some of the data are corrupted.

It is of great interest to look at the performance of the three test areas. Questions like: How well will they perform over time? Will winter conditions be of any problem? How long time will it take before the pavement is clogged? How can the structure of the pavement be stabilized? What it necessary thickness of the pavement? Can there be any improvements to the measuring equipment? Additional measurements required? What is a good sampling interval for different data?

In general, study all the measurements done so far and comment on the results.

5.1 Temperature

The temperature where first measured at four different locations. One located at the cabin which measures the air temperature, and then the three others at different levels in the ground at respectively 100 mm, 300 mm and 500 mm depth underneath the ground level at the same spot close to the cabin. In addition to those three, another three was placed out a the 5th of October 2015. One is placed under the permeable concrete block pavement, another one is under the regular concrete pavement and the last one is in the ground close to the cabin. They are all at 500 mm depth below the surface. Figure 35 shows the approximately positions for the three last temperature meters.

Figure 35 Approximately positions for the temperature meters at 500 mm depth, from the left permeable pavement, regular pavement and in the ground with no pavement (Photo: Jens H. Trandem)

Data for the three temperature meters at 100 mm, 300 mm and 500 mm can be extracted to excel. The first data are from May 2015, but there is not any good data before July 2015. The data that are found unlikely are listed below.

The criteria for removal of temperature data are:

1) Identical values are registered in several consecutive hours.
2) The registered value is more than 40 °C. The highest registered temperature in Norway is 35.6 °C (Lippestad, 2009)
3) More than one temperature value is registered at the same hour.

Temp_cabin

- Data from 07.05.2015 – 01.07.2015 are all 17 °C it is unlikely and they are removed.
- Data from 01.07.2015 – 15.07.2015 are all too high values to be correct measurements for the temperature, so they are removed.
- 18.10.2015 high values are removed
- 21.10.2015 – 23.10.2015 high values are removed
- 26.11.2015 high values are removed
- 13.12.2015- 14.12.2015 high values are removed
- 26.12.2015-28.12.2015 high values are removed
- 02.01.2015 – 09.01.2016 high values are removed
- 13.01.2015 – 21.01.2016 high values are removed
- 04.02.2016 – 05.02.2016 high values are removed
- 24.02.2016 19.56 removed extra measurement
- 13.03.2016 12.55 missing data, mean value is used
- 23.04.2016 07.55 removed extra measurement
- 23.04.2016 19.55 removed extra measurement

Temp_100

- Data from 07.05.2015 – 01.07.2015 have only values of 0 °C. They are not accounted for, because it is not likely that this is correct measurements.
- Data from 02.07.2015 – 14.07.2015 have unlikely high values and are not accounted for.
- Data from 06.01.2016 – 08.01.2016 have unlikely high values and are not accounted for.

Temp_300

- Data from 07.05.2015 – 01.07.2015 have only values of 0 °C. They are not accounted for, because it is not likely that this is correct measurements.
- Data from 02.07.2015 – 14.07.2015 have unlikely high values and are not accounted for.

- Data from 06.01.2016 have unlikely high values and are not accounted for.

Temp_500

- Data from 07.05.2015 – 01.07.2015 have only values of 0 °C. They are not accounted for, because it is not likely that this is correct measurements.
- Data from 02.07.2015 – 14.07.2015 have unlikely high values and are not accounted for.
- Data from 06.01.2016 have unlikely high values and are not accounted for.

The mean values are found for each day where reasonable data are collected. The air temperature is a mean value of measurements every hour. The temperature measured in the ground is a mean value out of two measurements, one in the morning at 08.00 and one in the afternoon at 20.00. The results can be seen in Figure 36.

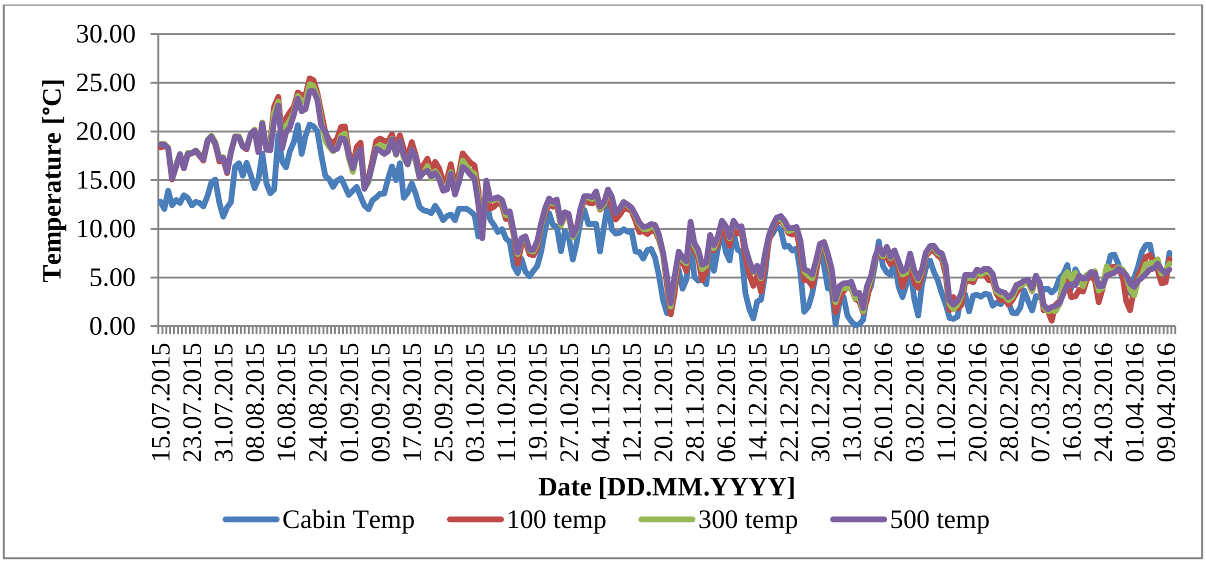

Figure 36 Daily variation of the temperature in the air, 100 mm, 300 mm and 500 mm below ground level. (Data from the test facility at Skjæveland and Multiblokk)

There is a significant difference between the air temperature and the temperature measured in the ground, but between the different depths there is no such clear difference. The overall mean values can be seen in Table 9.

Table 9 Mean values for temperature. (Data from the test facility at Skjæveland and Multiblokk)

	Air temp cabin	100mm temp	300mm temp	500mm temp
Mean value [C]	8.4	10.0	10.1	10.1

The temperature in the air is always colder than the temperature in the ground with a few exceptions. The one that stands out is the event in January where the air temperature is warmer than the ground temperature for several days in a row. From Figure 36 it seems to be the only one which is such significant. It could be the sun warming the air, but not enough to heat the ground. This is not unlikely to happen and don’t need to be wrong.

Are two measurements of temperature every day good enough?

It is good enough with one measurement of the temperature at 08.00 in the morning and one measurement at 20.00 in the evening to represent the air temperature? This will save storage capacity and there will be less data to process.

The temperature values plotted in Figure 36 as “Cabin Temp” is the mean value of measurements every hour. In Figure 37 it is possible to see the effect on the air temperature when only two measurements every day are used. One in the morning at 08.00 and one in the evening at 20.00 like it is done for the ground temperatures. High unreasonable values like the ones that are listed above are not accounted for. There are a few more values missing for the mean value of the temperature with only two measurements. These missing days are taken away from the comparison.

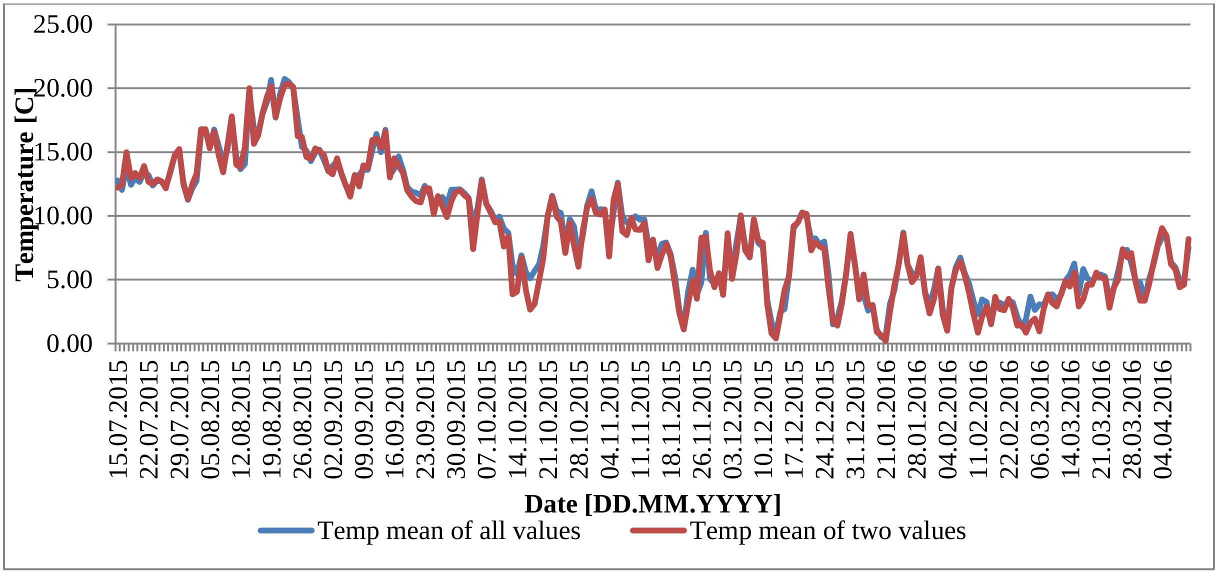

Figure 37 Comparison of sampling interval of every hour and for two times a day for air temperature. (Data from the test facility at Skjæveland and Multiblokk)

The two graphs are very similar with a few differences at some dates. From only this comparison it looks like it can be enough with only two measurements for the temperature. There is no need to collect data for every hour if you look at a daily perspective. To register possible melt and freezing periods during daytime, a measurement between 12.00 and 14.00 in the afternoon could be necessary. This will only be for the time period where the pavement is covered with snow.

Comparing the values from PLC with values from datafarm

PLC is the computer system where all the measurements are registered. Datafarm is just a webpage where values for every hour is extracted from the PLC system and put into excel documents which can be downloaded.

In Figure 38 there is one set of data plotted from values obtained from datafarm where they can be extracted to excel. The others are directly from the PLC system and are found by placing two markers at the time resolution you want. It could be minutes, days or months. So in order to find the mean value for July, one marker where placed at 01.07.2016 and as close to 00.00 as possible and the other at 31.07.2016 as close to 23.59 as possible. Then the min, max and mean value for the month are calculated. The two plots should be the same, since they are from the same set of data. The difference is just how they are extracted.

From datafarm it is possible to look at the values and remove the corrupted data. When the markers are used in the PLC system, there is no way to leave out the measurements that seems wrong if you want a certain time step like a month. The dates corrected for is listed earlier and there is some values almost every month that can possibly be the difference between the two graphs in Figure 38.

It is tried with not removing the data that seems wrong from datafarm when the mean value is calculated and compare them with data from PLC, but this is not giving good results, so there must be something else that makes the difference. Because if none of the values are removed from the datafarm data, the difference is getting very large.

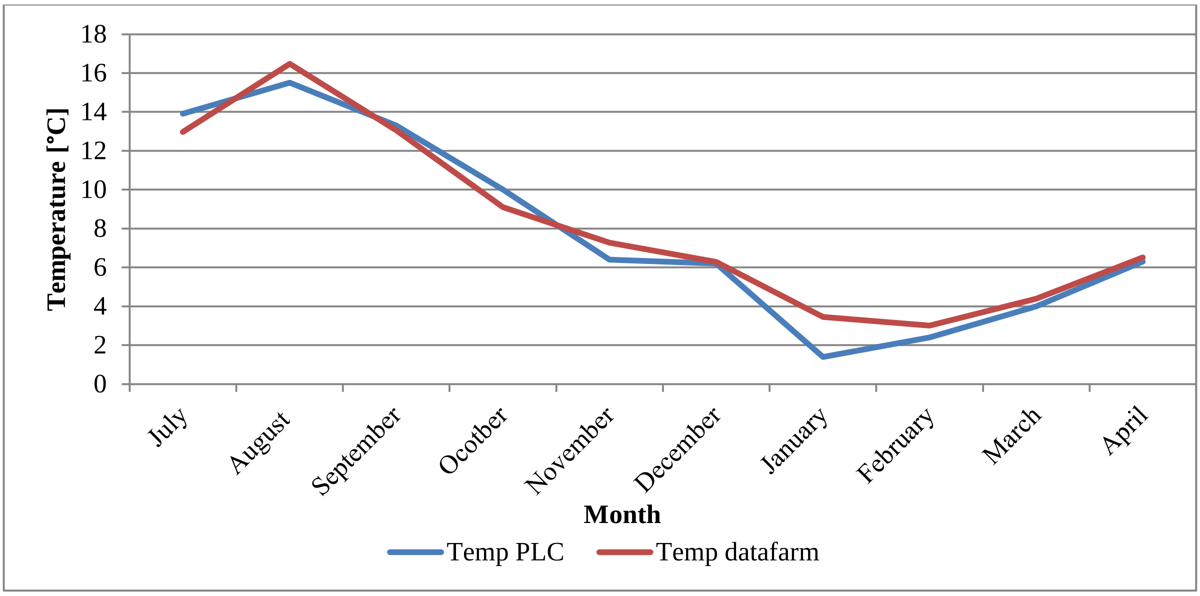

Figure 38 Comparing temperature values from PLC with temperature values from datafarm. (Data from the test facility at Skjæveland and Multiblokk)

The data are similar, but to be from the same set of data the two graphs should fit even better. The company "ECS Automasjon" visited the test area 13th of May and told that the values extracted to datafarm is only the value at the time it is extracted and not a mean value for the whole hour. This could explain the difference between the two graphs in Figure 38.

Comparison of temperature underneath permeable pavement, regular pavement and on-site masses

Data for the temperature meters at 500 mm depth below the permeable pavement, the regular pavement and the ground can only be extracted directly from the PLC system. The graph in

Figure 39 represents the monthly mean values for temperature under each type of cover, together with the air temperature.

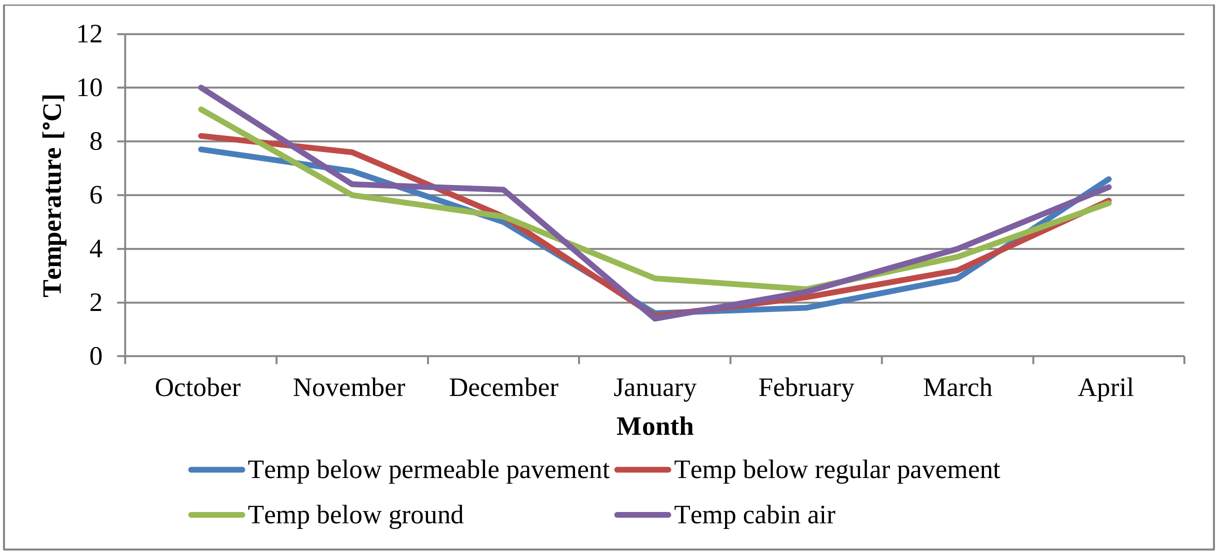

Figure 39 Monthly mean values for air temperature and the temperature at 500 mm depth for permeable pavement, regular pavement and normal on site soil. (Data from the test facility at Skjæveland and Multiblokk)

The temperature measured below the permeable pavement is not very different from the others, but it is slightly colder than the on-site soil for the coldest temperatures between December and March. Then again it is a bit warmer than the on-site soil for the higher temperatures in October, November and April. The temperature in the regular pavement and the permeable pavement is very close to each other.

In Chapter 3.9, it is referred to some articles looking at resisting against freezing. In the article by (Rohne and Lebens, 2009) and (Guthrie et al., 2013) both studies indicated that permeable pavement had greater resistance against freezing compared to regular pavement. The difference was noticed in the subgrade. For the surface temperature checked by (Guthrie et al., 2013) no significant difference where noticed. The comparison done in Sandnes is not from the subgrade but somewhere in the subbase reservoir, this is something to take into consideration when comparing the results with the other studies. The regular and permeable pavement in Sandnes is also constructed in the same way except for the surface layer. From the comparison done in Sandnes it is not possible to conclude with greater resistance against freezing for the permeable pavement compared to impermeable pavement.

5.2 Rainfall

Rainfall is measured on top of the cabin housing the PLC logger.

Not all the data collected seems to be correct. Data that looks unlikely or wrong is listed below and removed from the dataset.

The criteria for removal of precipitation data are:

1) Identical values are registered in several consecutive hours.
2) More than one rainfall intensity value is registered at the same hour. The value registered at a different minute than the other measurements within the hour is removed.

- Data from 07.05.2015 - 01.07.2015 have all the same value of 5 mm of rainfall each hour. This is unlikely, so they are not accounted for.
- 04.07.2015 18.39 removed extra measurement
- 05.07.2015 21.09 removed extra measurement
- 06.07.2015 07.44 and 07.49 removed extra measurement
- 07.07.2015 13.54 removed extra measurement
- 08.07.2015 06.54 removed extra measurement
- 19.07.2015 13.24 removed extra measurement
- 01.08.2015 12.09 removed extra measurement
- 21.08.2015 19.59 removed extra measurement
- 13.09.2015 11.53 removed extra measurement
- 25.09.2015 01.43 removed extra measurement
- 26.10.2015 07.58 removed extra measurement
- 30.11.2015 14.52 removed extra measurement
- 23.12.2015 07.57 removed extra measurement
- 23.12.2015 19.57 removed extra measurement
- 20.02.2016 17.51 removed extra measurement, high value
- 24.02.2016 19.56 removed extra measurement
- 13.03.2016 12.55 missing data, use 0 which is surrounding values
- 23.04.2016 07.55 removed extra measurement
- 23.04.2016 19.55 removed extra measurement

When all this is removed from the dataset, the values are still very high and it is unlikely that they represent the precipitation at the test area in Sandnes.

The mean value is taken for the precipitation each hour and made into precipitation for each day. This is compared to data found on eklima (Meteorologisk institutt, 2016). Sandnes use precipitation data from a station in Rovik, Sandnes (Köz and Sigrist, 2015). Only the IVF curves existed on eklima for this station, so the data from Sola station is used instead.

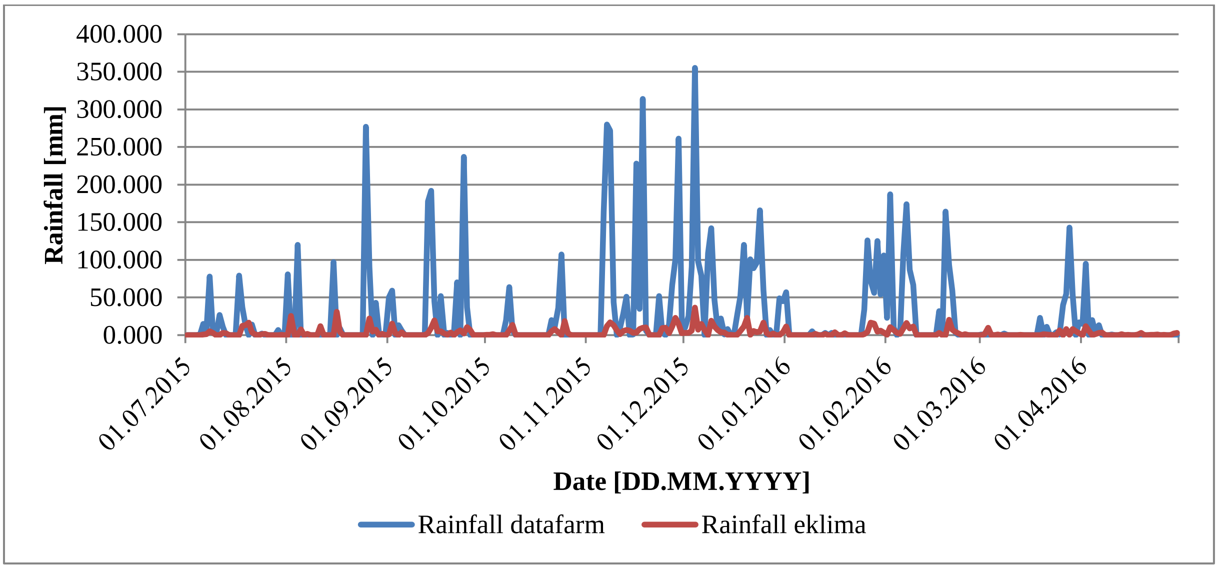

Figure 40 Precipitation measured at the test area in Sandnes compared to data collected from Sola weather station. (Data from the test facility at Skjæveland and Multiblokk)

In Figure 40 the precipitation measured at the test area in Sandnes are plotted against the precipitation measured at Sola. From the figure, it seems to be something wrong with the values from the test field. They are about ten times as much as the recorded values from Sola.

If the measured rainfall at the test area is divided by ten, it matches with the rainfall measured at Sola. They are not identical, but they are located approximately 10 kilometers away from each other, and the location to the ocean and topography is also different. So when comparing the two of them they are not expected to be the same, but similar. In Figure 41 they are plotted against each other.

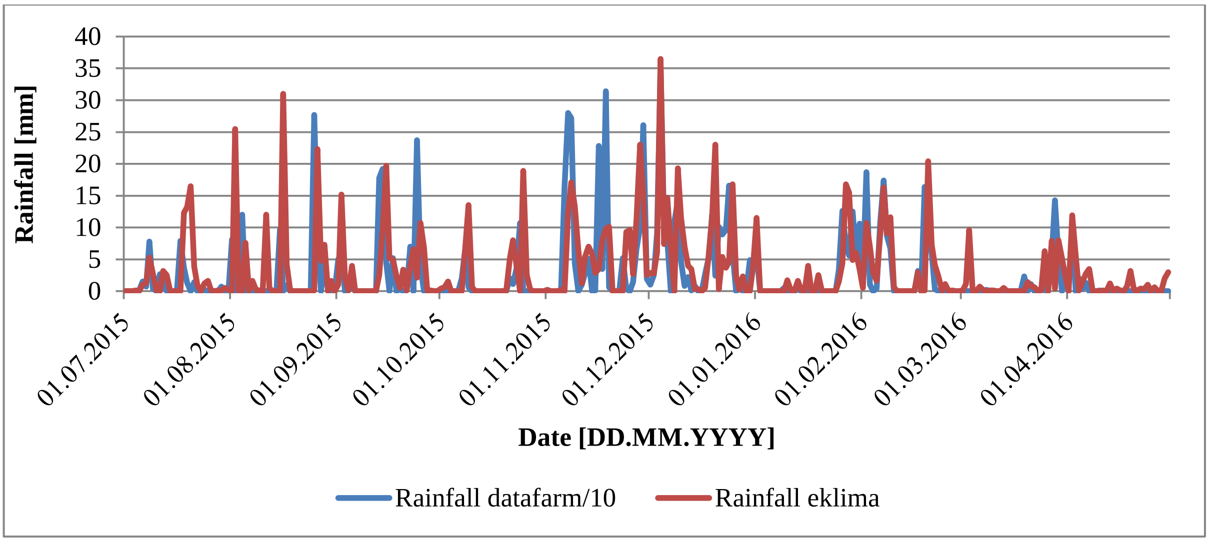

Figure 41 Rainfall at the test area divided by ten compared to rainfall measured at Sola. (Data from the test facility at Skjæveland and Multiblokk)

The total amount of measured rainfall each month is very similar to the registered data at Sola, when the measured rainfall from the test area is divided by ten. See Figure 42.

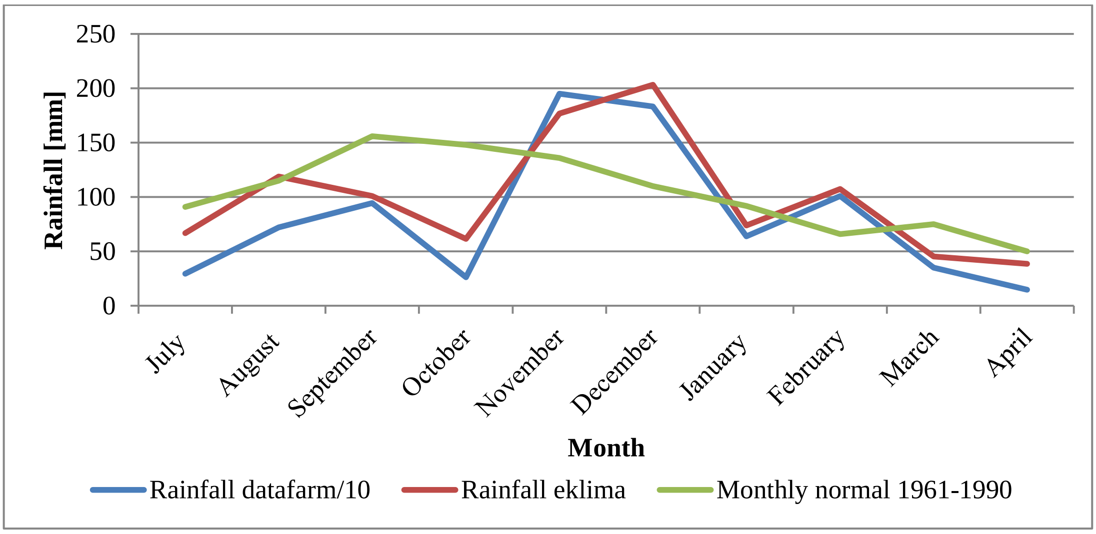

Figure 42 Total amount of rainfall measured for each month at Sola, test area divided by ten and monthly normal value from eklima. (Data from the test facility at Skjæveland and Multiblokk)

There is no doubt that there is something wrong with the measured rainfall at the test area, but it is not certain that when divided by ten gives the correct measurements. It is likely that there is something wrong either with the calibration of the rain gauge or with the processing of the data since the solution by dividing the data by ten fits so well to other measurements that are done close by.

The rainfall data for each month are collected directly from the PLC system and graphed in Figure 43 together with rainfall data from eklima and data collected from datafarm. The difference between the data extracted directly from PLC and datafarm are bigger than the difference between the two others. It is hard to say which is the most correct.

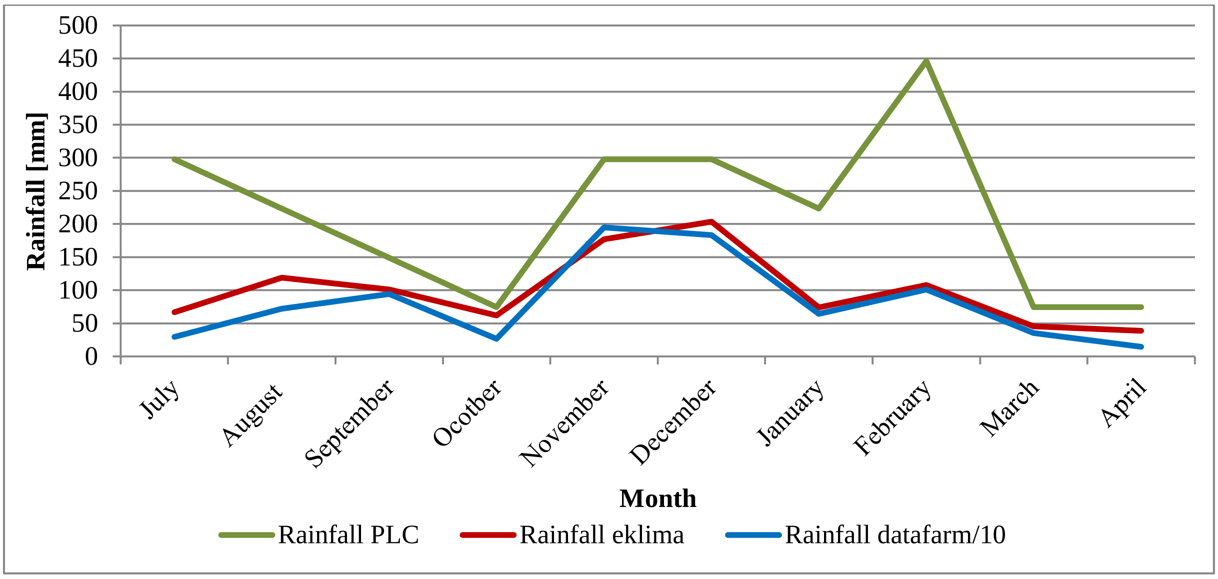

Figure 43 Rainfall data collected directly from PLC system compared to rainfall data from eklima and datafarm. (Data from the test facility at Skjæveland and Multiblokk)

To see if it was possible to find an explanation for why the PLC data and datafarm data are not the same, July 2015 was more closely inspected in PLC. Mean values for each day was found and again graphed against each other. The difference between the graphs in Figure 44 is quite big. In Figure 45 it is zoomed closer in on one day, to try to find the reason for why they are so different.

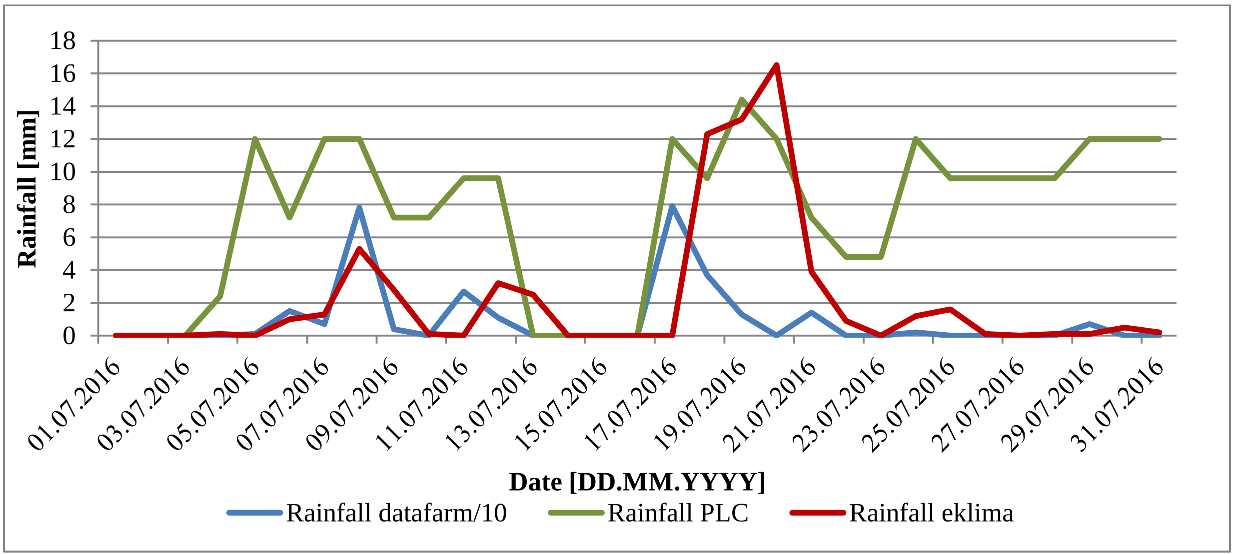

Figure 44 Measured rainfall for each day during July 2015 is plotted for the data from PLC, eklima and datafarm. (Data from the test facility at Skjæveland and Multiblokk)

During the 08.07.2015 there were registered rainfall at a certain hour in the data from datafarm, but this was not registered in PLC, where the precipitation was registered in other hours of the day. Something else that seems incorrect is long time intervals with the same rainfall intensity. After the registered rainfall around 09.00, 08.07.2015 it is registered a constant rainfall of 0.5 mm/h until the next day. The data from PLC register the rainfall intensity every 12 second, so it should be able to notice if it stops raining for a short period or the intensity changes. In Figure 45 the rainfall registered each hour from datafarm and the PLC is plotted against each other. The measured rain is not from the same hour, it is not the same amount and the constant 0.5 mm/h intensity for the PLC graph is questionable.

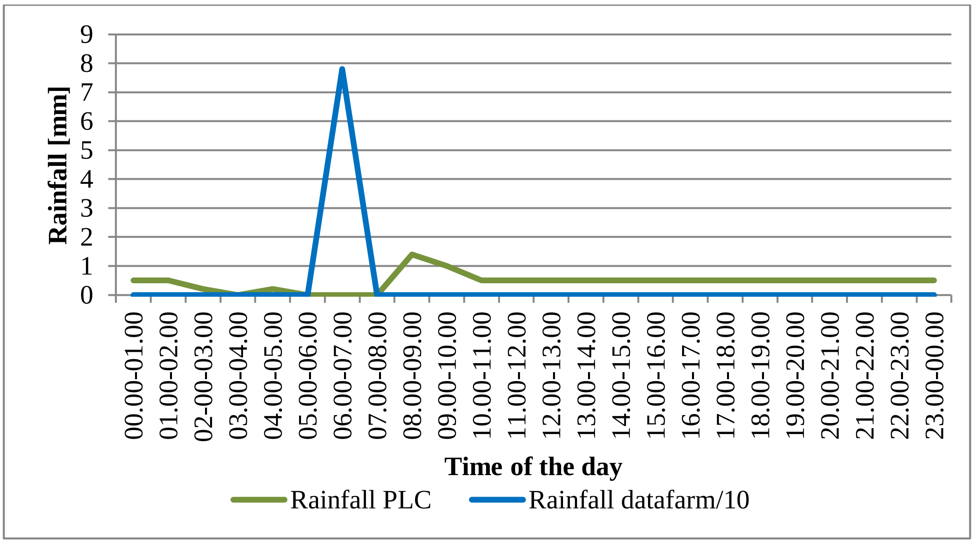

Figure 45 Measured rainfall for each hour during July 8th is plotted for data from PLC and datafarm. (Data from the test facility at Skjæveland and Multiblokk)

The same comparison of data from PLC and datafarm are done for August with daily sampling interval. This is also showing great variation between the two dataset. Some of the reason is the small amount of rainfall measured for a long period, something which seems incorrect. Other differences are more difficult to point out. There are for example just flat graphs and no registered rainfall from 18.08 to 31.08 because of something that looks like a failure to the PLC system, but still datafarm is able to provide rainfall data with changing amount of precipitation.

Explaining some of the problems with precipitation data

The company "ECS Automasjon" that made the PLC system visited the test area Friday 13. May. They checked the weather station to see if they could explain the strange measurements of rainfall. From the control, they found out that the values used in datafarm are multiplied by ten to get the desired amount of decimals. So the data should be divided by ten to get [mm/h]. The values extracted to datafarm are only the measured value at the time it is extracted. This means that it might not be representative for the rainfall period. It also explains why the monthly mean values from PLC do not go along with the monthly mean values from datafarm.

The data from PLC is more accurate than the data from datafarm, because all the values for rainfall within a period are accounted for. The hourly precipitation value which is registered in datafarm should be registered as a mean value instead of a just a single value from the time extracted, to be more accurate. In PLC there is not possible to remove corrupted data which could lead to unreliable results, so none of the options are perfect.

5.2 Wind speed

The wind speed [m/s] is measured at the cabin housing the instrumentation.

Not all the data collected seems to be correct. Data that looks unlikely or wrong is listed below.

The criteria for removal of wind speed data are:

1) Identical values are registered in several consecutive hours.
2) More than one value is registered for wind speed at the same hour. The value registered at a different minute than the other measurements within the hour is removed.

- The data collected from May until July 1st is all zero which seems to be highly unlikely.
- 21.08.15 19.59 removed (double measurement for this hour)
- 26.10.15 07.58 removed (double measurement for this hour)
- 23.12.15 07.57 removed (double measurement for this hour)

- 23.12.15 19.57 removed (double measurement for this hour)
- 24.02.16 19.56 removed (double measurement for this hour)
- 13.03.16 12.55 is missing use mean value of the two closest
- 23.04.16 07.55 removed (double measurement for this hour)
- 23.04.16 19.55 removed (double measurement for this hour)

The wind speed is found as a mean value of the measurements done every hour for each day. Figure 46 is showing the variation in wind speed from day to day.

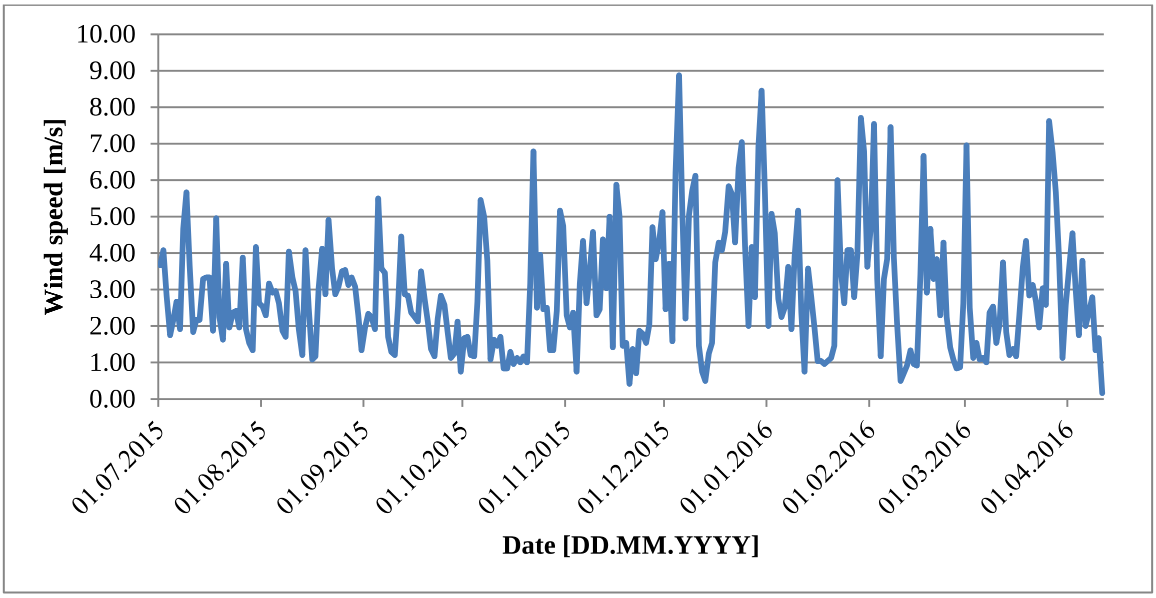

Figure 46 Wind speed in [m/s] given for each day from 01.07.15 until 11.04.16. (Data from the test facility at Skjæveland and Multiblokk)

5.3 Relative humidity

Both the actual vapor pressure and the saturation vapor pressure are needed in the Penman formula for evaporation.

The relative humidity (RH) is used to find the actual vapor pressure (e_a) from Equation 11 (Zotarelli et al., 2010)

Where the saturation vapor pressure (e_s) is given by Equation 10 (Zotarelli et al., 2010).

The data that seems to be wrong for the relative humidity is listed below.

The criteria for removal of relative humidity data are:

1) Identical values are registered in several consecutive hours.
2) More than one value is registered for the relative humidity at the same hour. The value registered at a different minute than the other measurements within the hour is removed.

3) The measured value is less than 4% which is the lowest registered value in Norway. Usually the relative humidity will be between 50 – 90 %. (Dannevig and Harstveit, 2009)

- From May until July 1st all the values are at 80 %. They are assumed to be wrong.
- 21.08.15 19.59 removed
- 26.10.15 07.58 removed
- 23.12.15 07.57 removed
- 23.12.15 19.57 removed
- There are some measurements the 21.11.15, 22.11.15 and 23.11.15 which are 0 [%] this gives very low values for the mean value and is not accounted for. The same problem is for the 02.01.16, 03.01.16, 04.01.16 and the 15.12.15.
- 24.02.16 19.56 removed (double measurement for this hour)
- 13.03.16 12.55 is missing use mean value of the two closest
- 23.04.16 07.55 removed (double measurement for this hour)
- 23.04.16 19.55 removed (double measurement for this hour)

The relative humidity is shown in Figure 47.

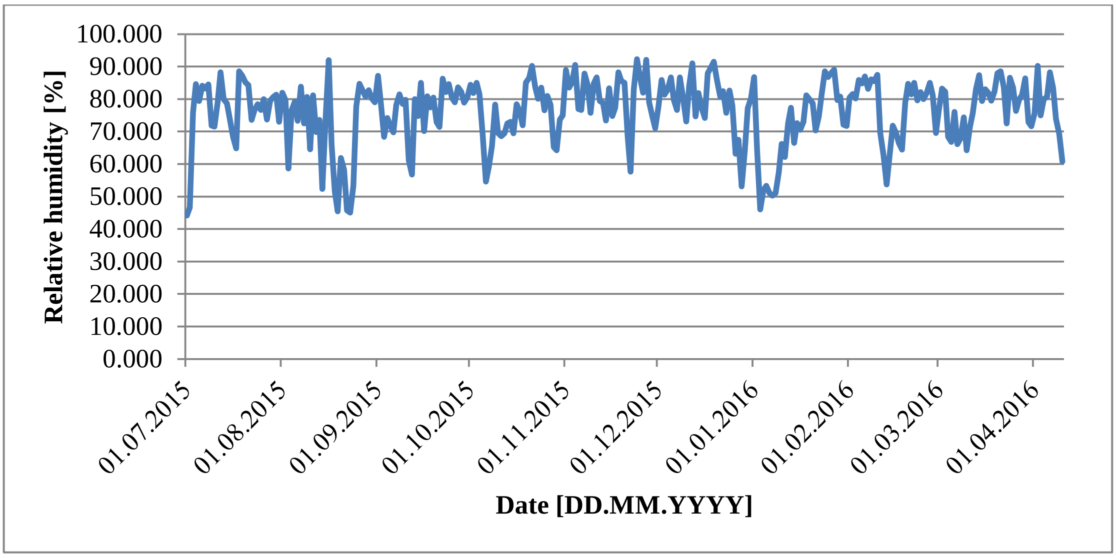

Figure 47 Relative humidity [%]. (Data from the test facility at Skjæveland and Multiblokk)

5.4 Air pressure (P)

The atmospheric air pressure is needed in the formula for the psychrometric constant (ɣ) (Equation 9) which is used in the formula for evaporation (Equation 6).

The data that seems to be wrong is listed below.

The criteria for removal of air pressure data are:

1) Identical values are registered in several consecutive hours.
2) More than one value is registered for the air pressure at the same hour. The value registered at a different minute than the other measurements within the hour is removed.
3) The lowest registered value for air pressure in Norway is 94.0 kPa (Harstveit and Store norske leksikon, 2009), so values of 0 [mbar] is removed.

- From May until July 1st all the values are at 1001 mbar. They are assumed to be wrong.
- 21.08.15 19.59 removed
- 26.10.15 07.58 removed
- 23.12.15 07.57 removed
- 23.12.15 19.57 removed
- There are some measurements the 21.11.15, 22.11.15 and 23.11.15 which are 0 [mbar] this gives very low values for the mean value and is not accounted for. The same problem is for the 02.01.16, 03.01.16, 04.01.16 and the 15.12.15.
- 24.02.16 19.56 removed (double measurement for this hour)
- 13.03.16 12.55 is missing, mean value of the two closest is used
- 23.04.16 07.55 removed (double measurement for this hour)
- 23.04.16 19.55 removed (double measurement for this hour)

The data from datafarm is measured in [mbar] and in the formula the air pressure input is in [kPa]. This is why Figure 48 is plotted with [kPa] on the y-axes.

1 [mbar] = 0.1 [kPa]

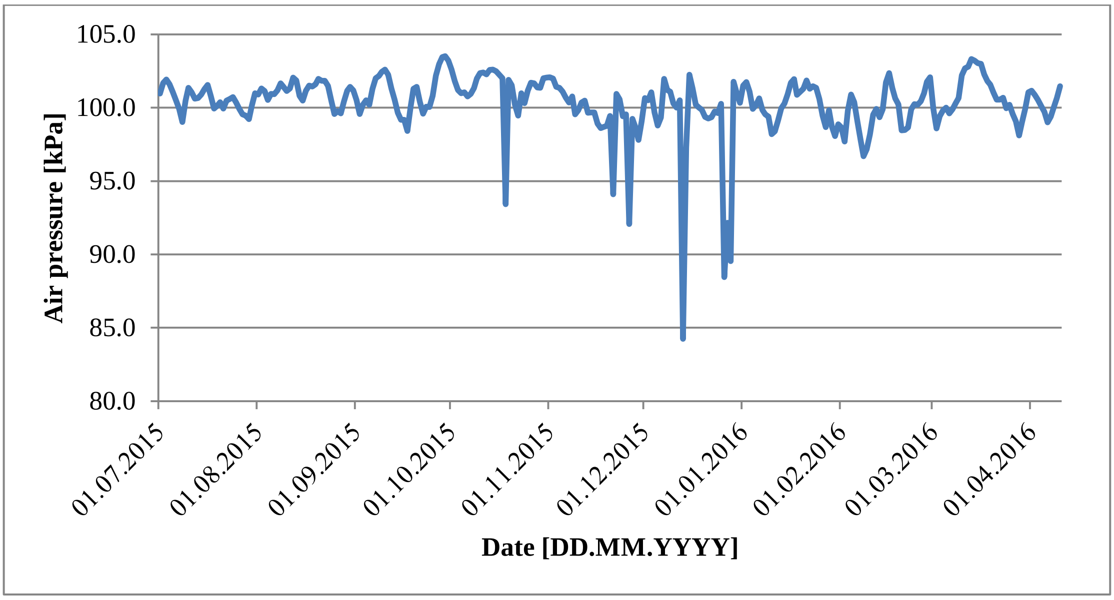

Figure 48 Air pressure measured in [kPa]. (Data from the test facility at Skjæveland and Multiblokk)

5.5 Sandtrap/infiltration basins SF1 and SF2

5.5.1 SF1

Test area 1 and test area 2 are the only areas draining to SF1. If inflow is measured in the sandtrap/infiltration basin, this is excess runoff from the road section. The data registered in datafarm and extracted to excel is only constantly repeated values of high inflow, this could not be correct. Something must be wrong with the computer code. Directly reading of the values from PLC is used instead, to see if there is any inflow registered.

The flow meters installed in SF1 and SF2 needs a full pipe flow to give correct measurements, so they are installed below the outlet for the infiltration chamber and below the outlet leading to SF2. In this way the pipe will always be full so it is not necessary with big amount of excess water or a small pipe diameter to get a full pipe flow.

For the data registered in PLC the water level is almost constant around 400 – 403 mm after installation. Then the 8th of July the water level drops down to 45 mm, because of some further installations in the manhole. The pipe where the inflow is measured is no longer filled with water, and the flow meter is not giving correct results. You can see measured inflow before rain events, because the water level is too low. Incorrect readings as shown in Figure 49 will appear. The green graph is measured inflow and the white graph is measured rainfall.

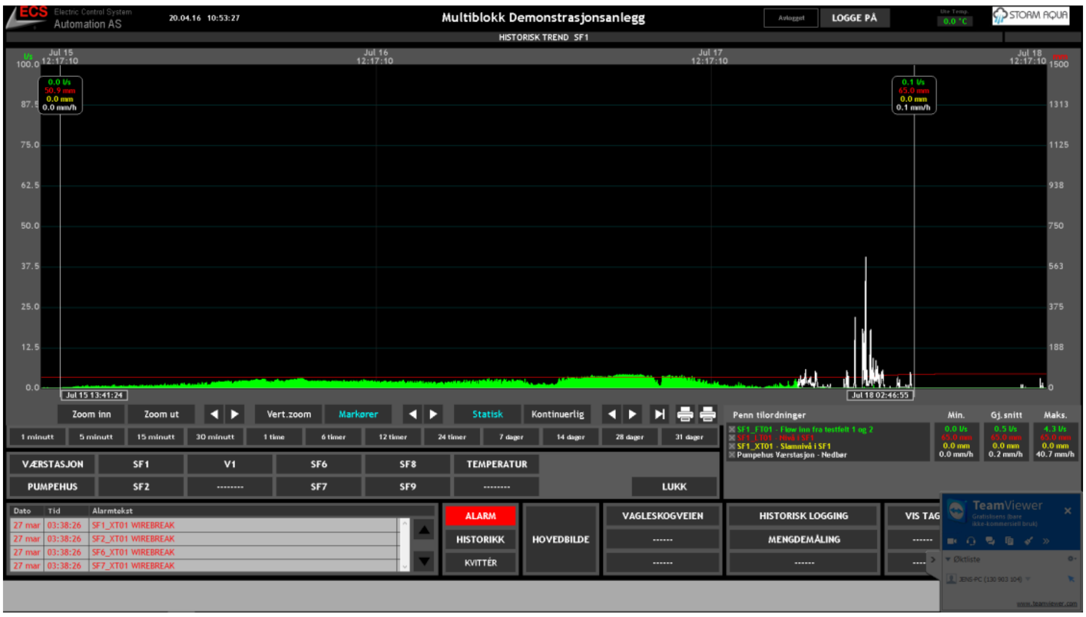

Figure 49 Measurements from SF1 the 16th and 17th of July 2015. Green graph is inflow, red graph is water level and white graph is rainfall.

Inflow before rain events appear several times when the water level is too low for the flow meters to give correct results. The water level is slowly increasing and the 17th of September it is back around 396 mm. From the 17th of September and until April the water level stays constant around 396 mm and no inflow is registered except for a few incidents. There is one in October, one in January and one in February. They are just small spikes on the graph and not necessarily actual inflow. It is hard to explain why the water level in the basin is increasing without any inflow. The manhole is either leaking so groundwater can infiltrate, or the groundwater level is so high that water flows back into the manhole from the infiltration basin in the bottom and through the submerged outlet (personal communication Per Møller-Pedersen, 2016).

Table 10 is monthly values of the measurements done for SF1 since October when the water level was back at where it should be to get correct measurements for inflow. There is some variation in the water level, but not much. The few incidents where some inflow is detected are very small, and are not in any connection with a bigger rainfall event. As mentioned it is not for sure it is actual inflow.

Table 10 Monthly data collected for SF1. (Data from the test facility at Skjæveland and Multiblokk)

	Flow			**Level**			**Rainfall**		
Month	Min [l/s]	Mean [l/s]	Max [l/s]	Min [mm]	Mean [mm]	Max [mm]	Min [mm/h]	Mean [mm/h]	Max [mm/h]
October	0	0	0.8	394.4	397.7	404.7	0	0.1	34.7
November	0	0	0	396.4	398.5	404.7	0	0.4	83.3
December	0	0	0	393.4	398.4	406.3	0	0.4	98.8
January	0	0	0.3	384.8	397.2	402.3	0	0.3	68.1
February	0	0	0.1	384.8	397.3	401.6	0	0.5	100
March	0	0	0	385.8	396.4	398.8	0	0.1	28.6
April	0	0	0	394.1	396.1	402.5	0	0	4.8

From the data collected, it indicates that the permeable pavement in test area 1 and 2 are still able to infiltrate the rain water and maintain an acceptable infiltration capacity.

5.5.2 SF2

The water going into SF2 is from test area 3 and SF1 if the water level in SF1 gets too high. Since no water is detected as inflow to SF1, and no big changes in the water level is noticed, the water measured as inflow to SF2 is only from test area 3. Again the data from datafarm is incorrect. It is the same problems as with the flow data from SF1.

SF2 and SF1 have the same flow meters installed. This means that it is important with a water level around 400 mm to get full flow and correct measurements for the flow data. In Figure 50 the outflow data in red is not reliable before the 17th of September when the water level is back at 405 mm.

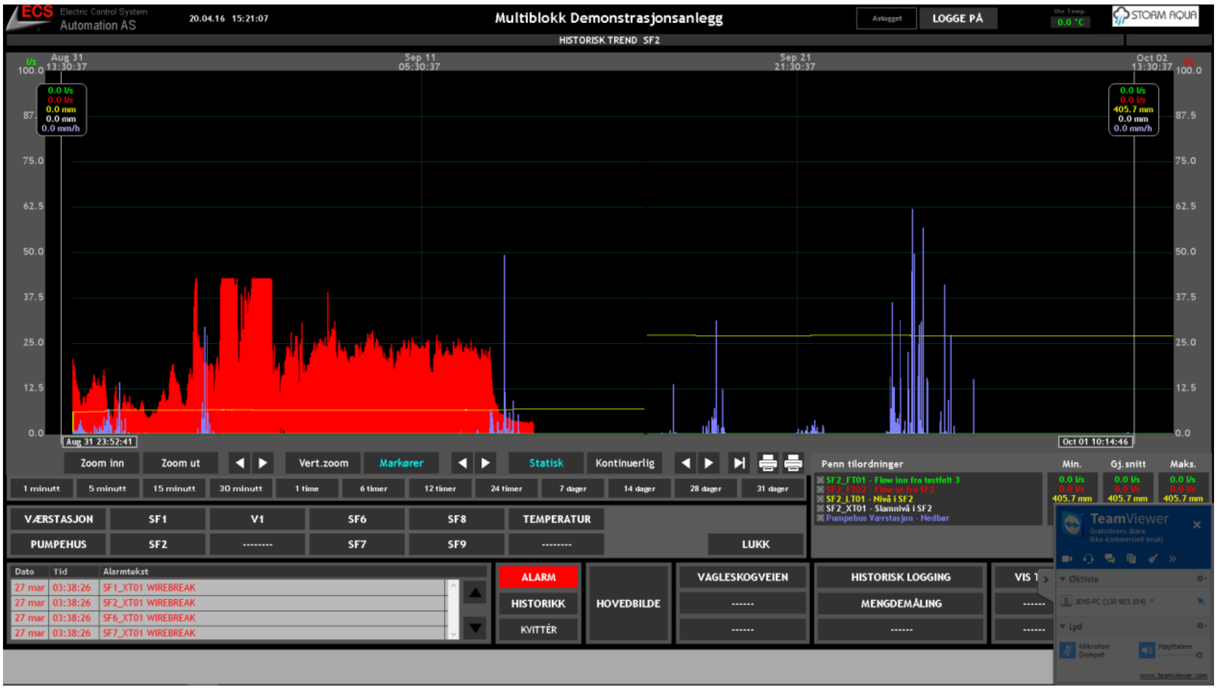

Figure 50 Example of wrong measurements of outflow data (in red), when the water level (in yellow) is too low.

The first time the water level in SF2 rise above 406 mm is the 24.10.2015 kl. 21.27. This is after a rain event, but no inflow is measured from test are 3. The same rise in water level happens 3 times in November with no registered inflow and another five times in December before there is registered some inflow. Figure 51 from 20.12.2015 is the first event with registered inflow.

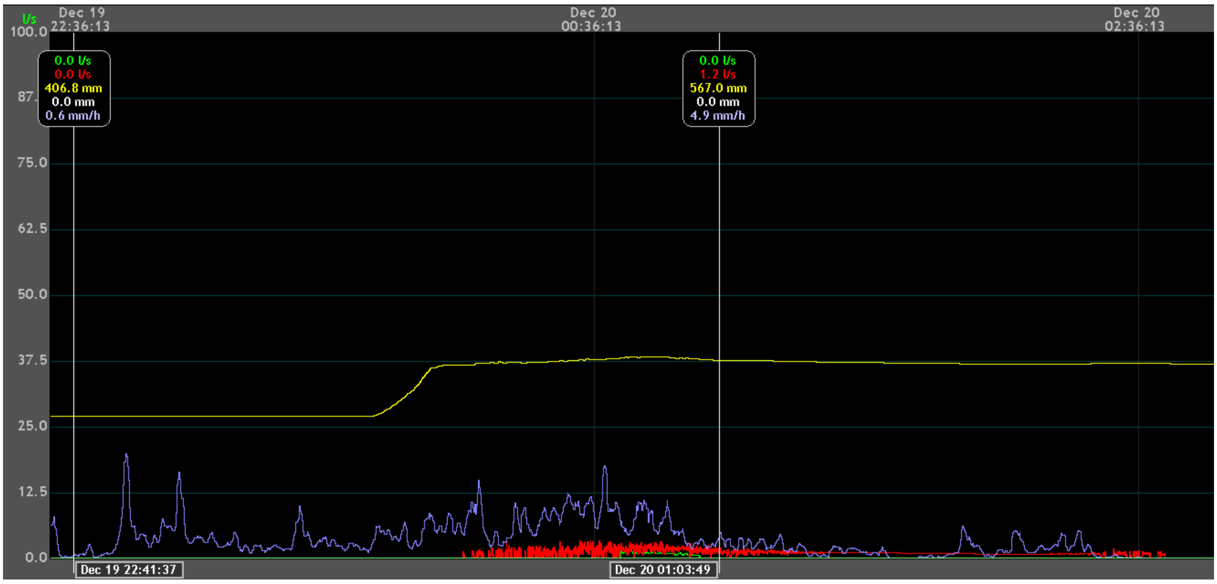

Figure 51 Rain event with increased water level and outflow before inflow. Yellow = water level, Blue = rainfall, Red = outflow, Green = inflow

The increased water level is after some time with registered rainfall. When the water level gets constant and the rain continues, the water starts to flow out of SF2 and into the detention basin. This is how you would expect the curves to behave, except for the inflow. It is only registered inflow for a short period after the raise in water level and after the water has started to flow out of SF2.

Since there are a lot of incidents where the water level is increasing without any inflow it must be something wrong with the flow meters or the water is going into the sandtrap/infiltration basin from other openings. Another possibility is that groundwater is entering through the infiltration basin in the bottom. This is possible if the groundwater level outside the manhole is above the submerged outlet inside the basin. It is hard to say if all the events are lack of capacity for the permeable pavement in test area 3 or something else when no inflow is measured.

The rainfall event 24.12.2015 in Figure 52 is how you would expect the graphs to behave. It starts to rain and you can see the rise in water level and inflow before the outflow is registered. Then the rainfall intensity increases and the inflow, the outflow and the water level respond to this.

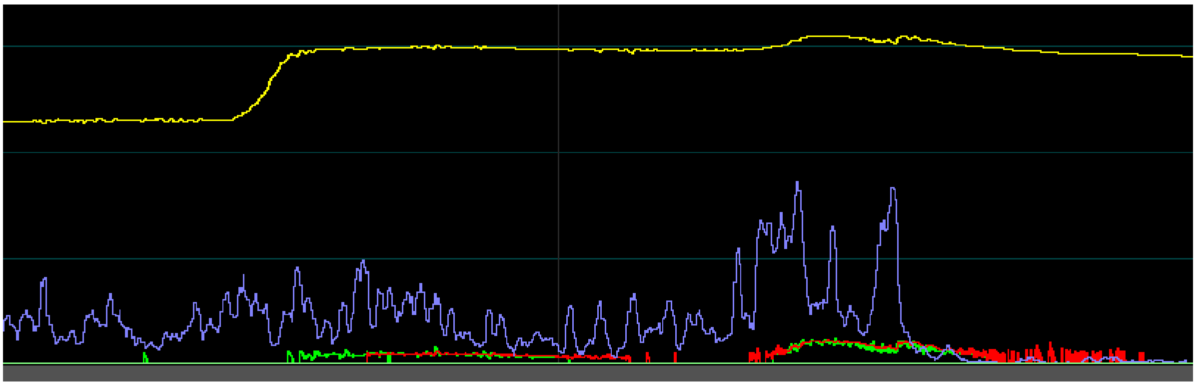

Figure 52 24.12.2015 Yellow = water level, Red = outflow, Green = inflow and Blue = rainfall

In Figure 53 the number of events where the water level in SF2 is increasing is plotted against the total measured rainfall for this month. The numbers of events are following the amount of precipitation. An increase in monthly rainfall gives an increase in water level incidents. Another correlation between the data compared, is that there are more incidents with water level increase after September without increased amount of rainfall. Could be indications of clogging of the permeable pavement, or just rainfall with higher intensity occurring in these months.

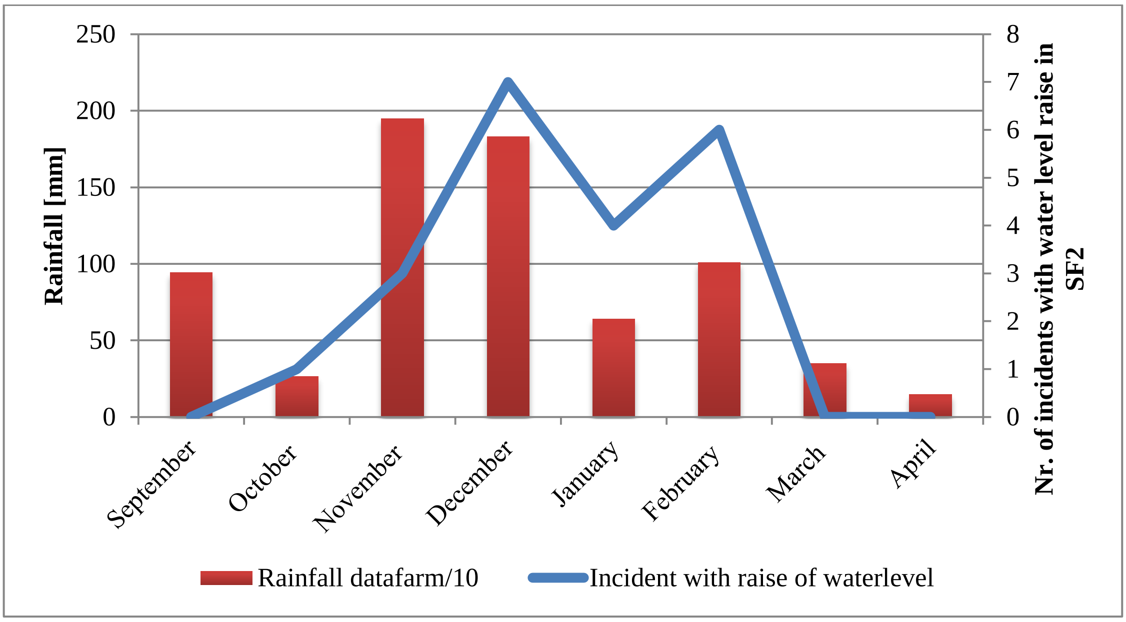

Figure 53 Number of events with increased water level for SF2 and the amount of rain the corresponding month. (Data from the test facility at Skjæveland and Multiblokk)

Figure 54 is looking closer into the data for the registered raise of water level in SF2. The graphs plotted are mean values for all the incidents within the associated month. The mean value for the rainfall events, hours with rainfall, level before rise and max water level rise is almost the same for all months. The max intensity during the rainfall and total rain is decreasing. The rainfall intensity measured for the events with water level rise in SF2 from October is reduced with 64 % in February. The total rain for the event is only reduced with 5 % from October but with 38 % if we look at November. It could be the change in climate from October to February causing the water level in SF2 to rise more frequently for events with lower rainfall intensity and total rainfall, or maybe it is indications of clogging.

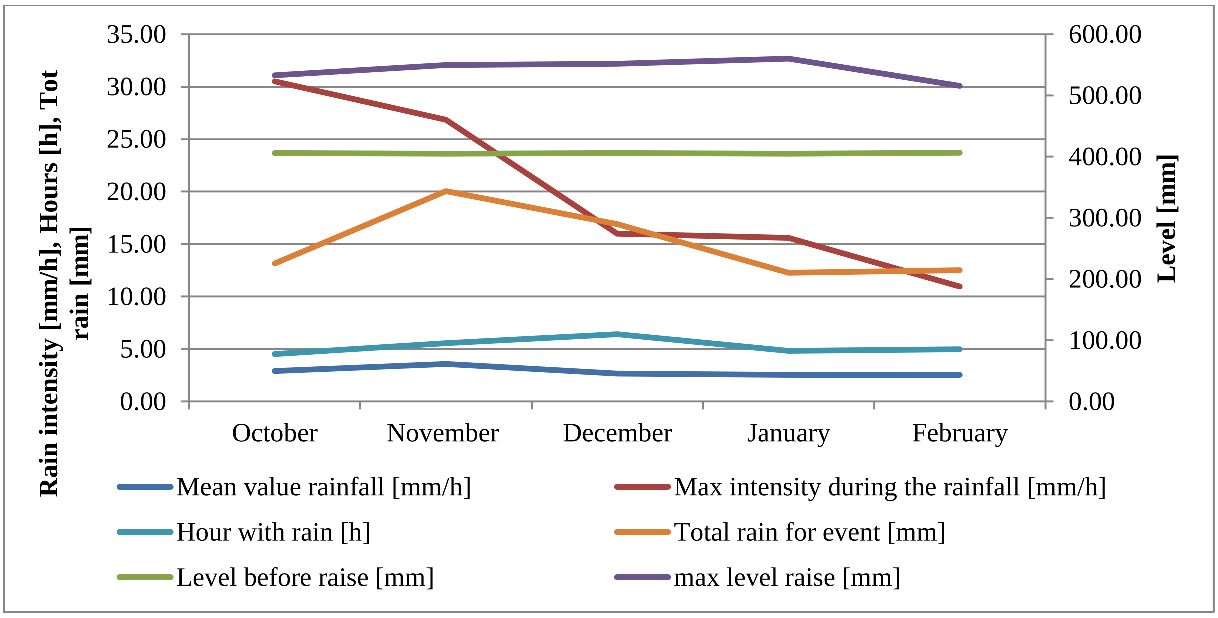

Figure 54 Mean values for the water level raise events within the same month. (Data from the test facility at Skjæveland and Multiblokk)

For some of the incidents with increased water level in SF2 the rate of water level increase and water level decrease are calculated from the data. Both the decreasing and increasing rate are approximate values. For both the decreasing and increasing values it is decided to use start as where the graph makes a noticeable change in the water level and stop when there is another noticeable change in the water level. Figure 55 is an example of how an increasing and decreasing episode is evaluated.

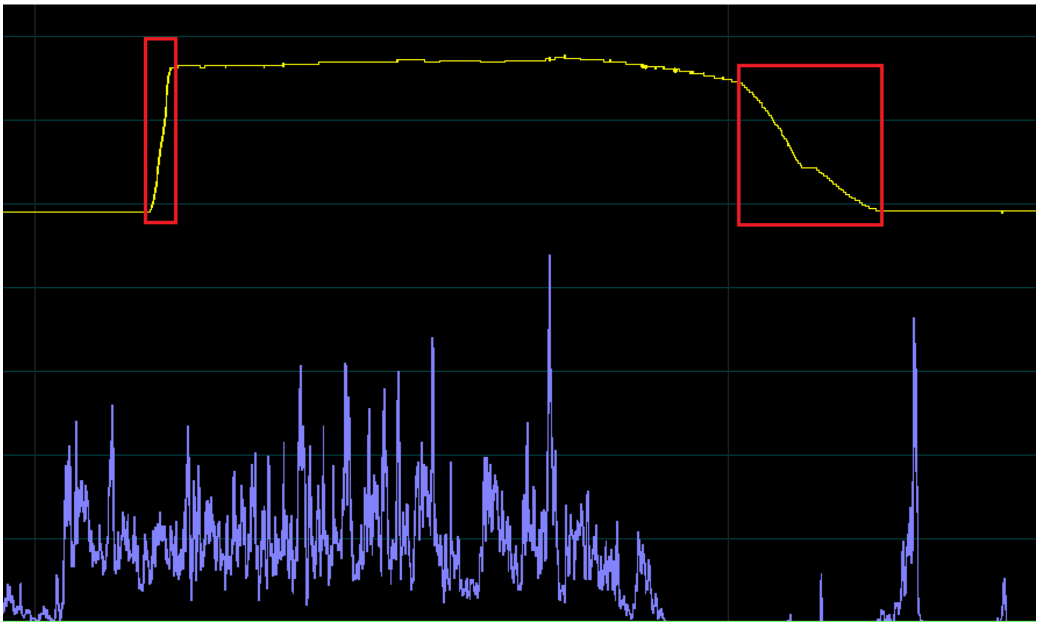

Figure 55 The red squares indicates the period of time evaluated for increasing and decreasing.

The rate of water level increase and decrease in SF2 is presented in Table 11. It is considered as good infiltration when the infiltration rate is 10 cm/h. For all the events in Table 11 the only event with an outflow at the same time as the water level decreases is 07.11.2015. The infiltration rate for SF2 is less than what is considered as good infiltration for four out of five events, but close to 10 cm/h for three of those four.

Table 11 Rate of water level increase and decrease for some events in SF2. (Data from the test facility at Skjæveland and Multiblokk)

	07.11.2015	08.11.2015	04.12.2015	19.12.2015	24.12.2015
Level increase [cm/h]	69.81	63.18	56.80	64.15	91.97
Level decrease [cm/h]	-4.82	-11.07	-7.91	-8.26	-9.67

When a sandtrap/infiltration basin like SF2 is installed in connection with the permeable pavement, excess water will be infiltrated to the ground, but the amount of water will be limited by the infiltration rate of the soil and the infiltration area underneath the basin.

5.6 Infiltration of runoff water and the impact on the groundwater level

A lot of runoff is created from the impermeable part of the parking area. This runoff is supposed to be infiltrated at a relatively small area. The infiltration capacity underneath the permeable pavement in Test area 3 was tested during construction (see Figure 33) and is not considered as very good. The water will follow the formation in the ground and the groundwater level will increase further down.

To maintain control of the groundwater level and where the water drains, some suggestions were made for the positioning of groundwater wells to register the movement of the groundwater level. Figure 56 is only suggestions, but it would be useful to have control close to the infiltration areas and also the detention basin with the throttled outlet.

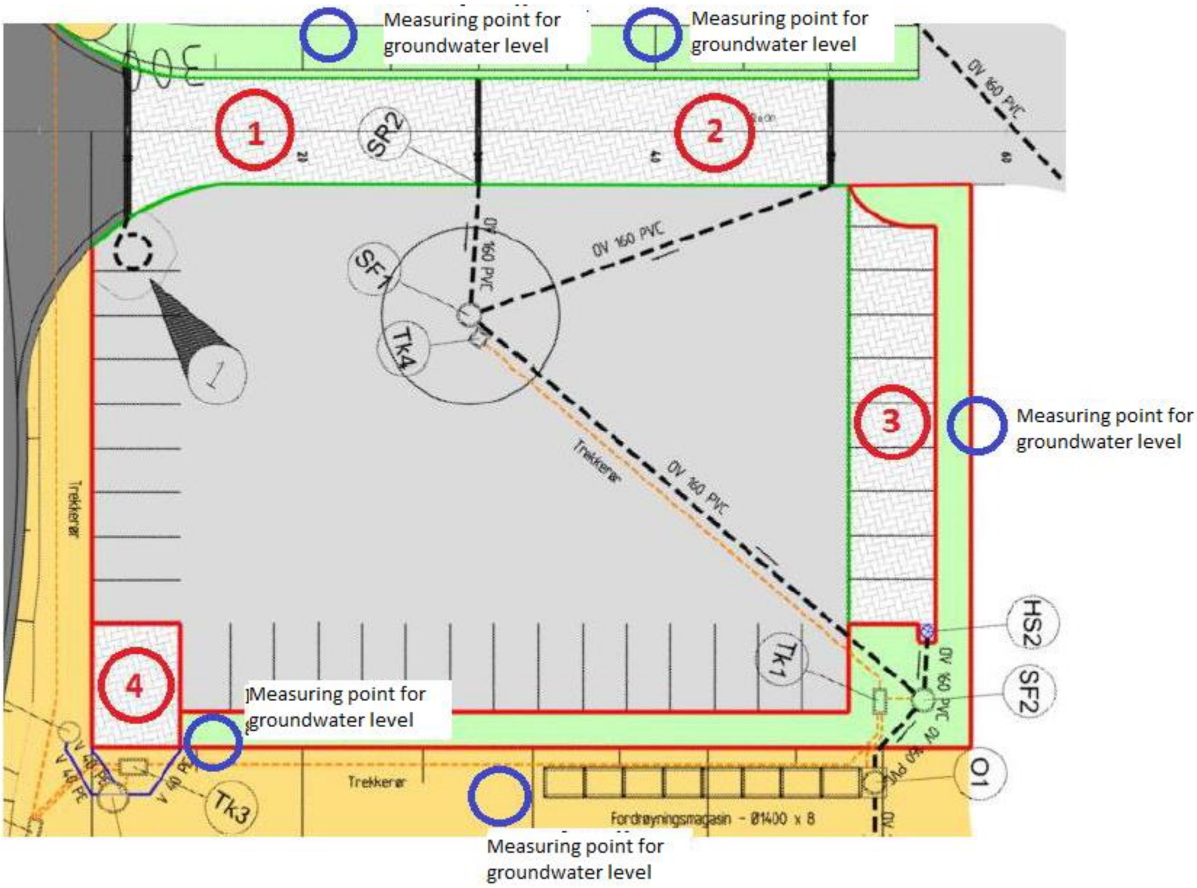

Figure 56 Suggestions for positioning of groundwater wells, to measure groundwater level. Adjusted from (Møller-Pedersen, 2015)

From the registrations of water level in the detention basin O1, the water level increased several times during rainfall events. Some of it was water going out of SF2 and into O1. Other registrations of increased water level could be explained with inflow because of the high groundwater level. The draining pipe out of the basin O1 is not protected from backflow. If the groundwater level exceeds the outlet, water will flow back in.

Four events where the water level in O1 is increasing and decreasing are looked closer into. In Table 12 the events are listed together with the water level before and after the first increase, the amount of rain causing the increase, total amount of rain before water level is back to low level, the level where it stabilize and time from level start to level end.

Table 12 Four events where water level in O1 is increasing and decreasing (Data from the test facility at Skjæveland and Multiblokk)

Date	Level before[mm]	Level after[mm]	Rain during first level raise [mm]	Tot rain [mm]	Level end [mm]	Time [h]
07. Nov	512.9	804.1	52.896	85.5	528.6	171
03. Dec	534.5	976.2	42.588	121.24	532.9	173.2
19. Dec	542.9	964.5	15.028	66.6	528	222
26. Jan	511.2	872.1	21.78	273.7	527.5	391

It is difficult to see some kind of correlation between the events analyzed in Table 12, measurements of the groundwater level would have been interesting to look at for these events.

Figure 57 shows the water level and precipitation for 07.11.2015. The first major increase in water level is at the same time as water is flowing from SF2 and into O1. The second increase in water level could be from high groundwater level forcing water to go back into the basin, it is not measured any inflow from SF2 at this point.

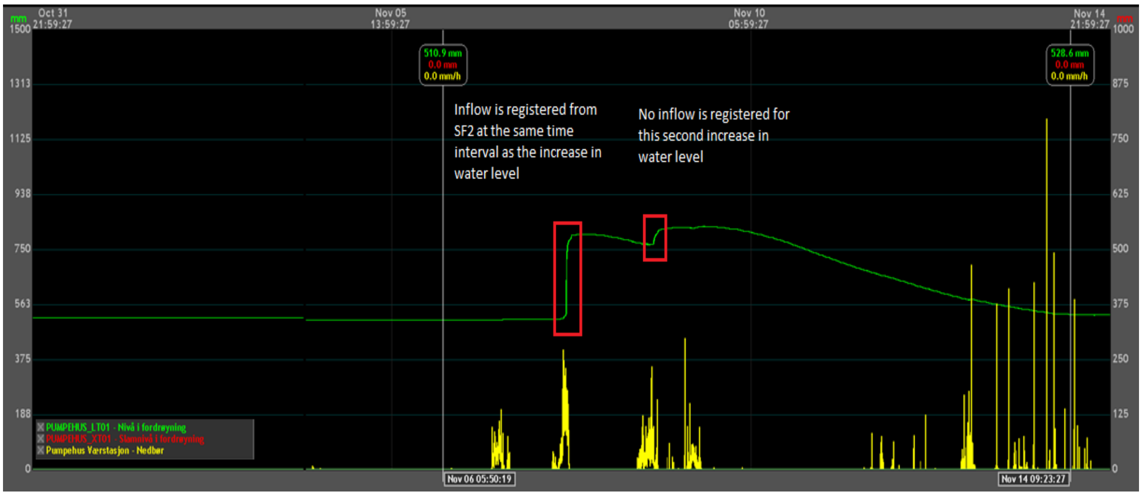

Figure 57 07.11.2015 Green = water level O1, Yellow = precipitation.

For the events looked into in Table 12 most of the increases you can see in the water level take place at the same time as inflow is registered, but as it is pictured in Figure 57 the water level is still increasing when no inflow is registered. When ground water wells are installed at the test area, this could be more closely looked into.

The event lasting from 26.01.16 to 12.02.16 has several increase and decrease in water level. The decreasing rate is found for all five incidents marked in Figure 58. The values are in Table 13.

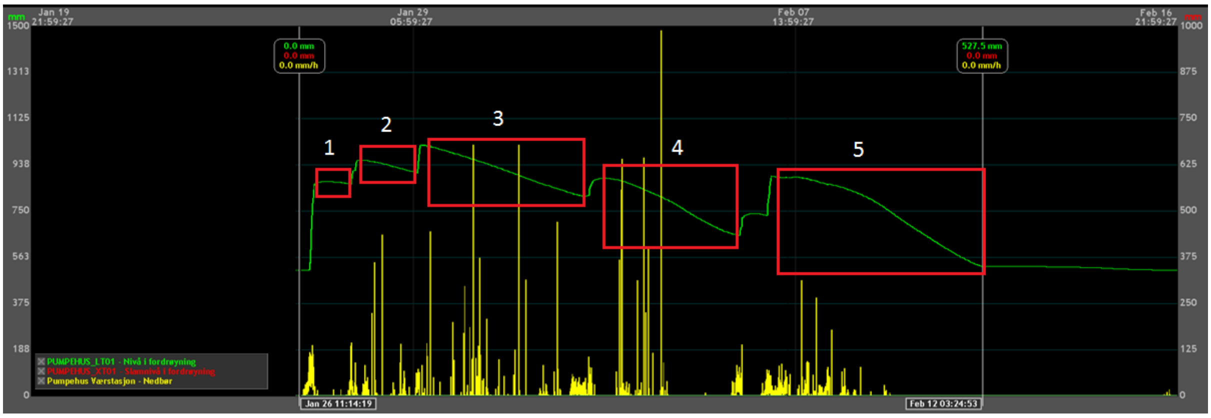

Figure 58 Several incidents of increase and decrease in water level before it stabilize at a normal level. Green = water level O1, Yellow = precipitation

Table 13 Rate of decrease for the five incidents marked in Figure 58 (Data from the test facility at Skjæveland and Multiblokk)

Nr.	Rate [mm/h]
1	0.91
2	1.46
3	2.16
4	2.99
5	2.96

The fastest decreasing water level is 2.99 mm/h. It could be the ground water level going back to normal after the precipitation events with a lot of water infiltrated to the ground.

When infiltration solutions are constructed to replace ordinary stormwater drains and pipes, it is important to have control of the area and all the hydrological aspects. It is not only the pavement who needs to be able to handle the stormwater, also the soil and the surroundings need supervision.

6 Assessment of evaporation

The evaporation is needed as a part of the water balance equation in Chapter 9. It is desirable to know everything about the pavement and what happens to the water when it reaches the permeable pavement. There are many approaches to calculate the evaporation, and for the calculations done in this thesis the Penman formula is selected.

6.1 Penman formula (Marasco et al., 2015)

PET_p is the potential evapotranspiration in (mm day^{-1})

$$PET_p = 0.408 * \frac{\Delta}{\Delta + \gamma * (R_n + G)} + \frac{\gamma}{\gamma + \Delta} * E_A$$

Equation 6

$$E_A = 2.6 * (1 + 0.54 * u_2) * (e_s - e_a)$$

Equation 7

R_n is the net radiation (MJ m^{-2} d^{-1})

G is the soil heat flux density at the soil surface (MJ m^{-2} d^{-1})

γ is the psychometric constant (kPa °C^{-1})

Δ is the slope of the saturation vapor pressure-temperature curve (kPa °C^{-1})

u_2 is the wind speed (m s^{-1})

e_s is the saturation vapor pressure (kPa)

e_a is the vapor pressure (kPa)

From (Zotarelli et al., 2010) an equation for the slope of the saturation vapor pressure-temperature curve (Δ), the psychometric constant (γ), the saturation vapor pressure (e_s) and the vapor pressure (e_a) is obtained.

$$\Delta = \frac{4098 \left[0.6108 * e^{\left(\frac{17.27 * T_{mean}}{T_{mean} + 237.3} \right)} \right]}{(T_{mean} + 237.3)^2}$$

Equation 8

T_{mean} is the mean daily air temperature (°C)

e is the base of the natural logarithm (2.7183)

$$\gamma = 0.000665 * P$$

Equation 9

P is the atmospheric pressure (kPa)

$$e_s = 0.611 * e^{\left(\frac{17.27*T}{T+237.3}\right)}$$

Equation 10

T is the air temperature (°C)

$$e_a = e_s * \frac{RH}{100}$$

Equation 11

6.2 Missing values needed for the Penman formula

6.2.1 Solar radiation (R_a) and soil heat flux density at the soil surface (G)

For solar radiation the values for short waved global radiation is found from (Meteorologisk institutt, 2016).

The total short wave radiation on the earth is called global radiation. This global radiation consists of direct sunlight and short wave diffuse radiation. (UiO, 2011) For the calculations the short waved global radiation is used as the net radiation (R_n).

Figure 59 from (Moreo, 2009) is showing the energy surface budget. When the soil heat flux (G) is subtracted from the net radiation (R_n) like it is done in the Penman formula, the remaining parts are the latent heat flux (λE) and the sensible heat flux (H).The latent heat flux is the energy component used for the evapotranspiration. The sensible heat flux can be accounted for by the Bowen ratio, but in the Penman formula we are assuming a water surface, which means that the Bowen ratio would not make a difference. (Moreo, 2009)

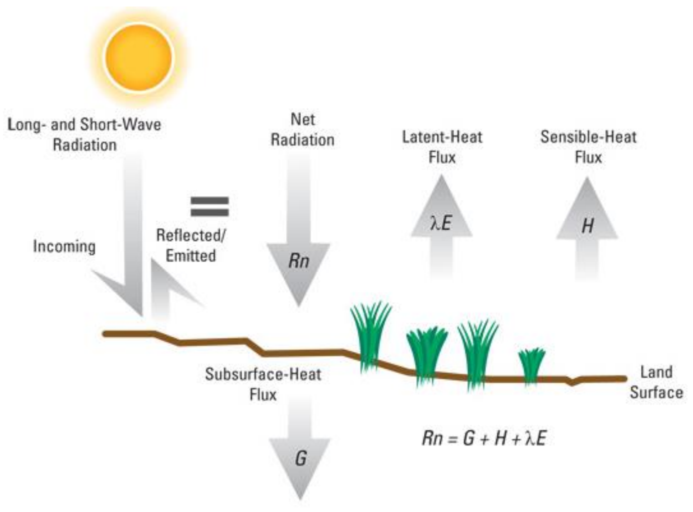

Figure 59 Energy surface budget (Moreo, 2009)

The soil heat flux density at the soil surface can be expressed as in Equation 12 and Equation 13 (CRA.Clima.Evapotranspiration, 2009).

$$G = c_d * R_n$$

Equation 12

and

$$G = c_n * R_n$$

Equation 13

Where c_d = 0.1 is daytime and c_n = 0.5 is night time. The mean value of c_d and c_n is used for the whole day, in the calculations.(CRA.Clima.Evapotranspiration, 2009)

6.3 Evaporation from permeable pavements, literature studies

Quantifying evaporation rates from annual evaporation is something that needs more focus and research (Brown and Borst, 2015). In the study by (Brown and Borst, 2015) they are looking at how the evaporation rate changes with antecedent dry period (ADP) and month, and also trying to quantify annual evaporation.

Evaporation from the permeable pavement will be limited. There is no vegetation that contributes to evapotranspiration. Studies of evaporation done in laboratories inside have the drawback of not using real climate conditions. The climate data needs to be created and difficulties with scaling the model and detecting the evaporation losses could also occur. (Brown and Borst, 2015)

Results from (Brown and Borst, 2015) showed that porous concrete (PC) had more evaporation then Permeable Interlocking Concrete Pavement (PICP) and porous asphalt (PA). Estimated evaporation ranged from 2.4 – 7.6 % of cumulative rainfall volume.

Hypothesis that smallest albedo (PA) would give the hottest surface and most evaporation. Results from (Brown and Borst, 2015) doesn’t show any sign of that. Not necessarily the temperature on the pavement surface that determines the evaporation. (Brown and Borst, 2015)

Experiments show that the evaporation rate could be high from pervious pavements if the weather conditions are favorable. The problem is that the permeable pavements are designed to infiltrate the water, and before the weather conditions are switched from rain to sun the water is already infiltrated through the pavement. (Nemirovsky et al., 2013)

The evaporation from an area would increase with increasing percentage of the area being pervious pavement. Another impact factor on the evaporation is the rainfall event size. Small and infrequent rainfall events will help increasing the evaporation. Other factors are depth to ponded water in the structure and the time since last rainfall event. (Nemirovsky et al., 2013)

In the article by (Nemirovsky et al., 2013) several suggestions to increase the evaporation is come up with.

- Shallow structure, not deeper than 254 mm
- Allow for water to raise almost up to the pavement surface for design storm event

Some more suggestions are also listed, but the principle is to bring the water stored in the structure closer to the surface for evaporation. This way of maximizing the evaporation in the design would cause troubles in colder climates where the water would freeze. (Nemirovsky et al., 2013)

Evaporation from depression storage could increase the evaporation. The permeable pavements are designed to infiltrate the water from the surface, but still there could be some depressions in the surface where water will stay for a short time before it evaporates. Then you have the surface evaporation, which is water evaporating from the surface of the pavement. The difference between this and the evaporation from the depression storage is that water deeper in the profile of the pavement would have to move towards the surface to replace the water evaporated. (Water Environment Federation (WEF), 2012)

6.4 Potential evaporation

The potential evaporation is calculated for the test area in Sandnes. Some of the parameters needed in the formula are measured at the test field, others are found from eklima the webpage of (Meteorologisk institutt, 2016).

R_n and G, are from the eklima (Meteorologisk institutt, 2016).

γ, Δ, e_s, e_a, u_2, T and P are all from values obtained at the test field with the instrumentations which are there.

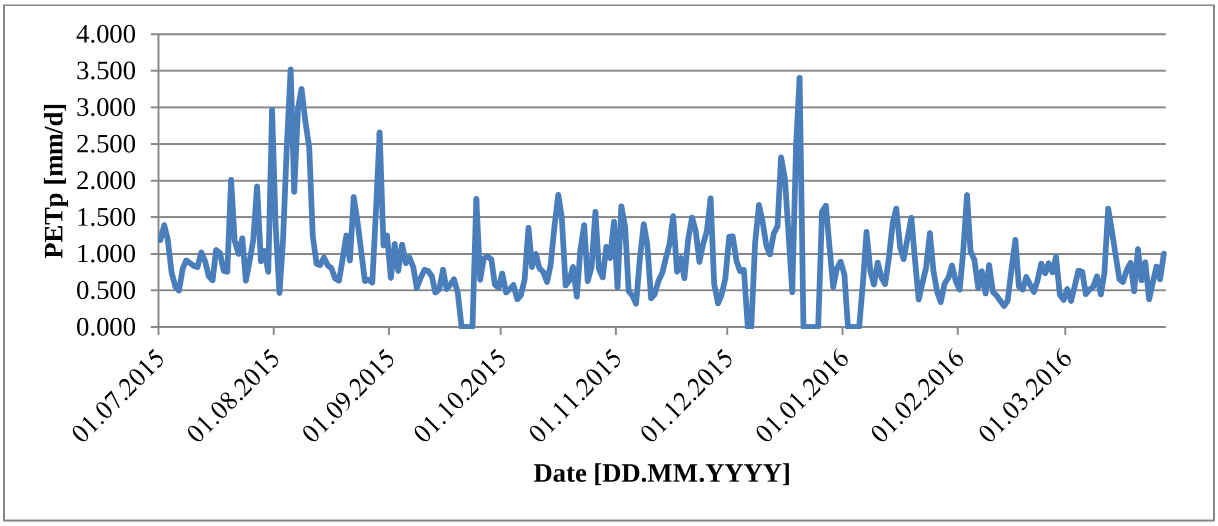

Figure 60 Potential evaporation calculated with Penmans formula at the test area in Sandnes. (Data from the test facility at Skjæveland and Multiblokk)

There are some missing values for R_n and G the 04.10.2015 – 07.10.2015 and 20.12.2015 – 21.12.2015. This gives no evaporation in the graph. This problem is also with missing values for T the 04.01.2016 – 08.01.2016 and 16.01.2016 – 20.01.2016.

The mean evaporation for the test area is 0.968 mm/d. From (Ødegaard et al., 2012) Chapter 2 the yearly evaporation in Norway can be set to 200 – 500 mm/year which is 0.548 – 1.370 mm/d. The potential evaporation found for the test area is in the range of the evaporation you can expect in a Norwegian climate. So a mean evaporation of 0.968 mm/d could be likely.

Figure 61 shows the mean evaporation for each month. The calculated potential evaporation is almost the same for all the months except August, but it is only used data from one year of registrations so further registration is necessary to say if this is representative for the test area.

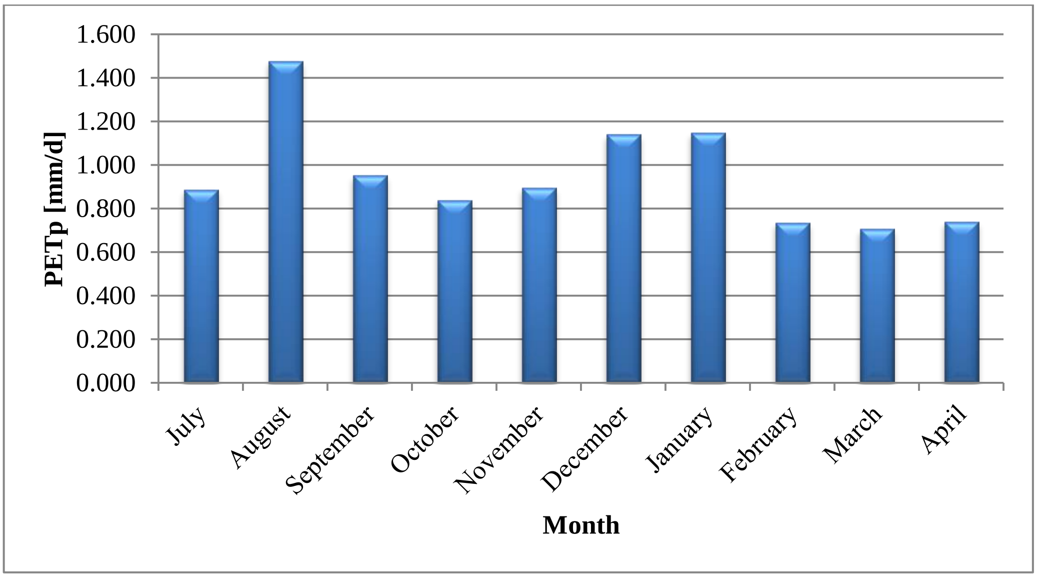

Figure 61 Calculated potential mean evaporation for each month for the test area in Sandnes. (Data from the test facility at Skjæveland and Multiblokk)

7 Test of Saturated Hydraulic Conductivity

7.1 Infiltrometer test, background and purpose

Infiltrometer tests were carried out at the test area in Sandnes in the end of April. The permeable pavement in test area 3 was starting to clog, and for some time water had been observed on top of the pavement after bigger rain events. Especially in the lowest part of the pavement the water gathered close to the curbstone before the water level reached the drain. This caused some algae growth at the area where the water stayed for shorter periods. Figure 62 show the difference between bottom pavement and top pavement.

Figure 62 Picture of the joint fillings for the permeable pavement in test area 3. Lowest part to the left with green algae on top of the filling material and the upper part to the right with no algae on top of the filling material. (Photo: Jens H. Trandem)

Since the pavement was no longer able to perform as supposed, it was planned to do some tests to find the saturated hydraulic conductivity. The saturated hydraulic conductivity were measured for the joint filling material and for the bedding layer underneath the concrete blocks.

The permeable pavement was tested to answer three different questions.

1) See if it was any difference between the lowest part where water gathered and the highest part where the water first reached after draining from the bigger parking area.
2) If the clogging was in the joint filling material or further down to the bedding layer.
3) The difference between algae and no algae.

The infiltration tests were done by using a “Modified Philip-Dunne Infiltrometer” (MPD), to determine the infiltration capacity which can be written as the saturated hydraulic conductivity (K_{sat}). The principle is simple; water is poured into the cylinder, a measuring tape is attached on the side, and based on the recorded water depth with time, the saturated hydraulic conductivity can be calculated.

7.2 Description of the setup

7.2.1 Test 1

Figure 63 is from the lower part of test area 3. The three MPDs are marked from 1 to 3 and tested for different conditions.

1) Approximately 5 cm of the joint filling material is removed. (MPD 1)
2) Nothing is done. The algae are still on top of the joint filling material. (MPD 2)
3) The algae are removed, but nothing else is done. (MPD 3)

Figure 63 Test 1 at the lower part of the pavement. (Photo: Jens H. Trandem)

7.2.2 Test 2

The bedding layer was tested close to the curbstone for the lower part and then further towards the upper part. To get access to the bedding layer, some concrete unit blocks had to be removed. The only way to remove them was to break them and remove the pieces. This resulted in some extra fine particles on top of the bedding layer which had to be removed.

1) The MPD is placed 18 cm from the curbstone in the lower part of test area 3. (MPD 1)
2) The MPD is placed 118 cm from the curbstone. (MPD 2)
3) The MPD is placed 218 cm from the curbstone. (MPD 3)

The infiltrometers were placed with some distance between each other from the lowest part of the permeable pavement to the highest elevated part of the pavement to see if the bedding layer preformed different with regards to infiltration. It is likely to believe that more particles will have time to infiltrate further down through the structure at the lowest part were water pond on top of the pavement for bigger rain events.

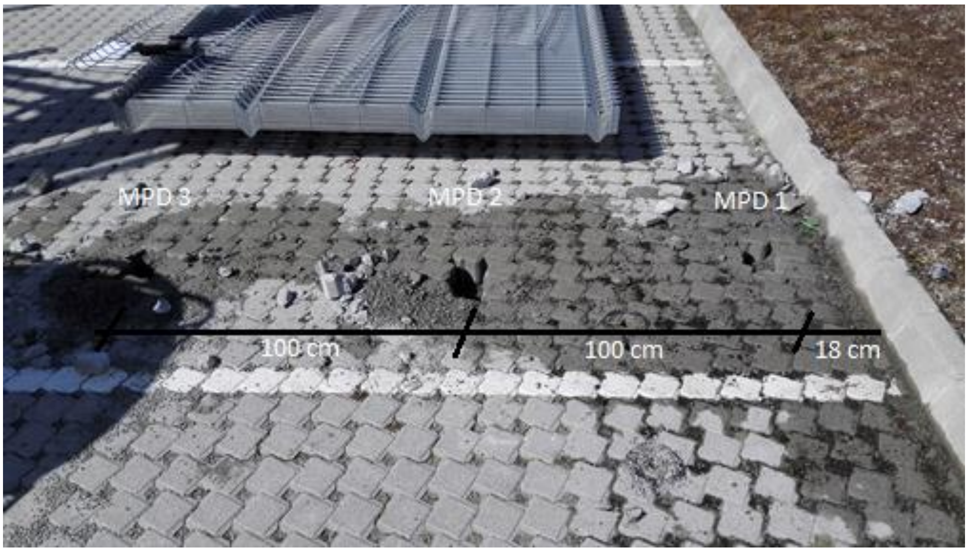

Figure 64 Test 2 infiltration through the bedding layer. The MPDs are removed, so only the holes in the permeable pavement are left. The distances between the infiltrometers are marked in the picture. (Photo: Jens H. Trandem)

7.2.3 Test 3

Figure 65 is from the upper part of the test area. Again the infiltrometers test for different conditions. It is almost the same as for the lower part, except there are no algae on top of the joint filling material.

1) Approximately 5 cm of the joint filling material is removed. (MPD 1)
2) Nothing is done. (MPD 2)
3) Just scratched a little bit in the surface. (MPD 3)

Figure 65 Test 3 from the upper part of the permeable pavement in test area 3. (Photo: Jens H. Trandem)

7.3 Description of how it’s done

To measure the infiltration through the joint filling material the infiltrometers had to be placed on top of the pavement. Sealant where used on the pavement and around the infiltrometers to prevent the water from leaking out. In Figure 66 it is a picture of the sealing and how it is done. The sealant needed some time to harden before it was water proof, but during the tests it was not possible to see any water leaking out.

Figure 66 Left: picture of the sealing. Right: picture of the MPD placed in the bedding layer. (Photo: Jens H. Trandem)

For the bedding layer it was possible to push the infiltrometer down to the black line marked on the infiltrometer to the left in Figure 66. To the right in Figure 66 the infiltrometer is standing in the bedding layer with water inside. Another difference is the permeable area inside the MPD. On top of the pavement the cylinder was placed around one of the squared openings with joint filling material. In the bedding layer the whole area inside is permeable.

When the MPDs were in place, water was taken from a big water tank and poured into the infiltrometers with a smaller bucket. The starting level of water was recorded together with the level as it decreased with time. A stopwatch was used and the water level was recorded for every 5, 10 or 30 seconds depending on how fast the water level decreased. Especially for the bedding layer the infiltration was so fast that manually reading was not possible. Instead of using a stopwatch and taking notes, a video camera was used to record the infiltration. The water level with time was later registered from the video recording.

Figure 67 Overview of the area and infiltration tests (Photo: Jens H. Trandem)

Several trial runs were done for each of the MPDs. Number of trial runs for each test is in Table 14.

Table 14 Number of trial runs done with each MPD for each Test.

	Infiltrometer	Number of trial runs
Test 1		
	MPD 1	4 trial runs were completed. All of them were done by using a stop watch and taking notes.
	MPD 2	4 trial runs were completed. All of them done by using stop watch and taking notes. Before the last 3 trials, an iron rod were used to punch through the filling material.
	MPD 3	1 trial run was completed. Stop watch was used for the measurements.
Test 2		
	MPD 1	3 trial runs were completed. All of them from video recording.
	MPD 2	3 trial runs were completed. Only 1 infiltrated slow enough to calculate K_{sat}
	MPD 3	3 trial runs completed. Video recording is used for all of them.
Test 3		
	MPD 1	3 trial runs completed. Video recording is used for all of them.
	MPD 2	3 trial runs completed. The first two are registered with a stopwatch and notes. The last two are from video recording, and an iron rod is used to punch through the filling material.
	MPD 3	4 trial runs completed. The first two with stop watch and notes. The last two from video recording.

7.4 Calculations

A Matlab code (from Tone M. Muthanna) is used to convert the measurements of water level with time to find the saturated hydraulic conductivity. The input data required for the Matlab code is listed in Table 15. The excel sheet for input values and the Matlab code is in the appendix.

Table 15 Input data required to calculate the saturated hydraulic conductivity with use of the Matlab code.

Parameters	Description
Initial volumetric water content [%]	The initial volumetric water content is not measured, so a value of 20 % is used. The joint filling material was not completely dry and the tests were done several times which made the material wet after the first test.
Final volumetric water content [%]	The final volumetric water content is not measured, so a value of 50 % is used. The material is mostly gravel with grainsize of 2 mm or larger with some finer particles and sediments. For this reason the volumetric water content is not expected to be more than 50 % after the infiltration.
Length of device below surface [cm]	5 cm is used for all the calculations. When the MPDs are placed on top of the pavement they are not 5 cm below surface, but the opening is surrounded by the pavement blocks which leads the water further down.
Radius of device [cm]	The radius is 4.3 cm.
Phase one initial height [cm]	The starting height of the measurements. The level of water when the infiltration test started at time 00:00:00.
Head [cm]	The recorded height of water in the cylinder during the test and how it changes with time.
Time [hh.mm.ss]	The time since the measurements started for every height of water level recorded.

7.5 Results

The saturated hydraulic conductivity (Ksat) calculated for all the tests are in Table 16.

Table 16 Saturated hydraulic conductivity calculated for all three tests.
(*) an iron rod is used to punch hole in the joint filling material
(-) infiltration too fast to measure
(Data from the test facility at Skjæveland and Multiblokk)

Infiltrometer		**Ksat [cm/h]**			
Test 1		Trial 1	Trial 2	Trial 3	Trial 4
	MPD 1	670.9	730.0	934.6	1292.8
	MPD 2	78.9	682.9*	660.1*	945.0*
	MPD 3	90.6			
Test 2					
	MPD 1	4060.8	5119.8	2309.0	
	MPD 2	338.4	-	-	
	MPD 3	5053.8	2881.1	9796.9	
Test 3					
	MPD 1	1000.1	991.1	1058.4	
	MPD 2	107.2	995.4*	1027.2*	
	MPD 3	554.6	617.9	491.7	596.6

Test 1 and Test 3 are both testing the infiltration through the joint filling material. The difference is that Test 1 is from the lower part of the pavement and Test 3 is from the upper part. Before the tests it would be reasonable to believe that the lowest part with algae growth would be more clogged than the upper part. More water in terms of volume will go through the lower part when the upper part starts to clog and water pond in the lower part. Since the water will start to infiltrate at once it reaches the permeable pavement, another theory is that the most polluted stormwater in terms of particles and dust will infiltrate in the upper part where the water first reach.

The first trials for Test 1 and Test 3 are plotted against each other in Figure 68 and from this graph it seems like the permeable pavement in the lower part are more clogged with a lower hydraulic conductivity.

It is only done a few tests in one part of the pavement and just one permeable pavement is observed in this study, but the results gives some indications of what to expect when you build your permeable pavement with an angel of inclination and allow water to pond on top of it.

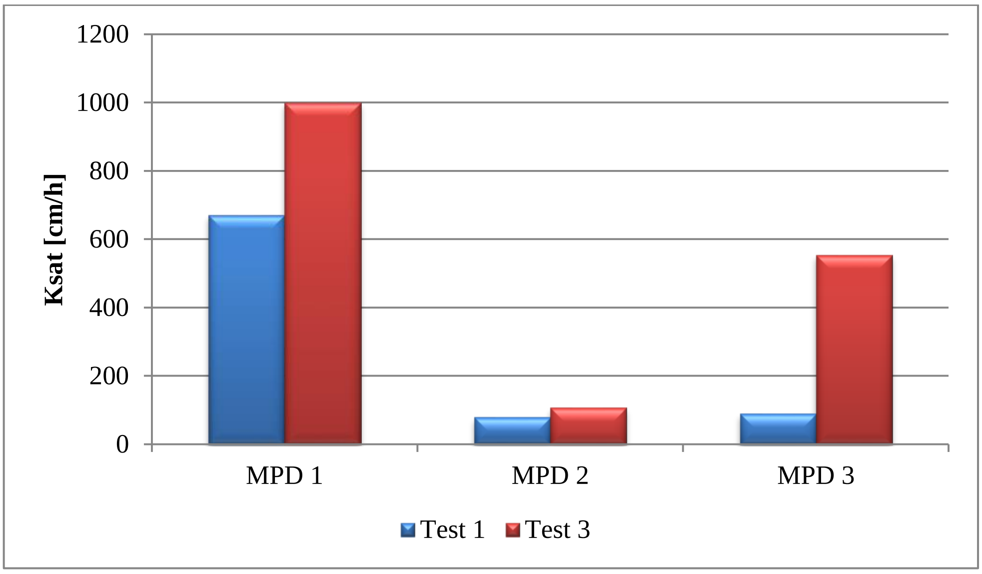

Figure 68 Measured saturated hydraulic conductivity (Ksat) for Test 1 and Test 3. (Data from the test facility at Skjæveland and Multiblokk)

When only the first trials are compared, the lowest part of the permeable pavement has a lower infiltration capacity than the upper part. If the mean value for Ksat is calculated for each MPD, the upper part will still have a higher Ksat, but the Ksat values for MPD 1 Test 1 will be more similar to the MPD 1 Test 3 value.

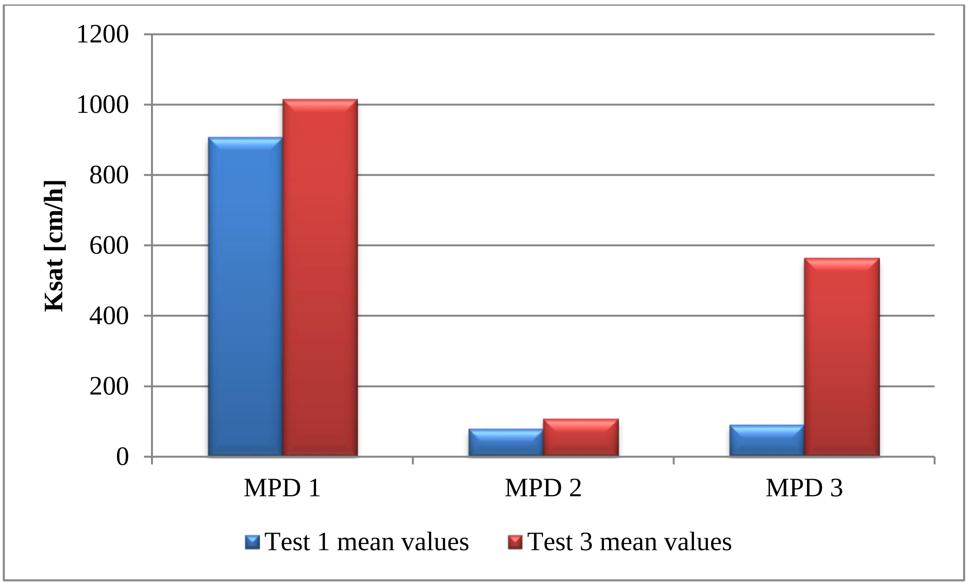

Figure 69 Mean values for measured saturated hydraulic conductivity for Test 1 and Test 3. (Data from the test facility at Skjæveland and Multiblokk)

The calculated Ksat values are very different, especially for the MPD 1 at Test 1. It is hard to say why, because when the measured height with time is more closely inspected in Table 17 which is the basis for the Ksat calculations, they are almost the same.

Table 17 Time and level of water for MPD 1 at Test 1. (Data from the test facility at Skjæveland and Multiblokk)

	Trial 1	Trial 2	Trial 3	Trial 4
Tid [s]	**Head[cm]**	**Head[cm]**	**Head[cm]**	**Head[cm]**
0	45	45	45	45
5	34	33	34	34
10	27	25	25	25
15	19	18	17.5	17
20	13	11	11	11
25	7.5	6	6	5.5

The saturated hydraulic conductivity measured for the bedding layer was very good. If we look at Table 16 and Test 2, the trials range from 2300 cm/h to almost 10000 cm/h. The only exception is MPD 2 and trial 1. The dust and finer particles were not removed properly after the concrete unit block was crushed. The water was very turbid and had a grey color compared to other trials. And when the water had infiltrated, the dirt could be seen as a layer on top of the infiltration surface. When this layer of dirt was removed, the infiltration went so fast that the MPD could not be filled with water fast enough to measure the infiltration.

To the left in Figure 70 the Test 2 MPD 2 Trial 1 is pictured. To the right, one of the other tests done for the bedding layer is pictured and you can see the difference in terms of particles and dirt mixed in the water.

Figure 70 Left: Test 2 MPD 2 Trial 1. Right: Another test done for the bedding layer. The difference in water quality with regards to fine particles and dust is significant. (Photo: Jens H. Trandem)

One of the initial questions was to see how deep the particles had moved into the pavement structure and where the clogging occurred. The difference between Ksat calculated for MPD 1 with approximately 5 cm of the joint filling material removed and MPD 2 where the joint filling material remained untouched was significant for both upper and lower part of the pavement. It seems like most of the fine particles stays in the joint filling material at the top 5 cm. The good infiltration measured for the bedding layer support these observations.

For Test 1 the algae on top of the pavement had nothing to say for the infiltration. It was dry because of the sun, and when water was poured into the infiltrometer it got mixed into the water and started to float. For Test 3 the difference between the scratching in the surface and untouched is much bigger. It could be that water poured into the cylinder mixed up the joint filling material and improved the infiltration or just a local variation. The iron rod showed that if only the layer with finer particles is broken, the infiltration rate will increase.

The infiltration measured for the bedding layer was very good for all three MPDs. There was some variation between the Ksat values found for each MPD, but there were also great variation between each trial for the same MPD. The Ksat values are found in Table 16 where Test 2 is done for the bedding layer. MPD 2 have only one Ksat value in the table, this is because after removing some more particles left from the crushing of the pavement, the infiltration went too fast to measure. The results for the measured Ksat values give no reason to say that the bedding layer is different for any of the three locations tested. Since the concrete stones had to be crushed in order to access the bedding layer and disturbed the most upper part, the test only show that the bedding layer is not clogged deeper into the structure.

Studies done by others compared with observations from Sandnes

In the article by (Pezzaniti et al., 2009) field tests and laboratory test where done to understand the decrease of hydraulic conductivity with time. The hydraulic conductivity for the laboratory test went from $1.7 * 10^4$ to $6.5 * 10^3$ cm/h for the form pave tested. This is a reduction of 59%. The hydraulic conductivity is very high compared to the measurements done in Sandnes, but the pavement in the study by (Pezzaniti et al., 2009) is constructed differently, so they cannot be directly compared. The field tests done by (Pezzaniti et al., 2009) have much lower hydraulic conductivity after the end of the monitoring period. The form pave tested in the field had only an infiltration rate of 20 cm/h at the end of the period. This shows the importance of doing field studies. Not every aspect of real life conditions can be simulated. If Test 1 and Trial 1 in Table 16 are used to calculate reduction in hydraulic conductivity and the test with 5 cm of the joint filling material is used as initial infiltration, the reduction is 88.4 %.

It is interesting to see the difference between laboratory studies where clogging by sediments are simulated compared to field studies. In the field studies they observed that the pavement was more clogged in the upper part where runoff first enters (Pezzaniti et al., 2009). In Sandnes the difference between the upper part and the lower part is not that significant. This is because the area of the permeable pavement is small, so at the time of ponding particles have also reached the lower part of the pavement and clogged the joint filling material. To prevent sediments from entering the upstream part of the permeable pavement, when other impermeable surfaces provide additional runoff is a key factor to maintain the effective life of the permeable pavement.

In the article by (Pezzaniti et al., 2009) it is referred to a study done by (Berry, 1995) where the infiltration rate where influenced by sediments in the void spaces in the upper 5 cm of the infiltration inlet zone. The storage potential of the bedding layer was not influenced by the sediments. For the infiltration tests done at the test area in Sandnes, the same experience where done. When the upper 5 cm of joint filling material were removed, the measured hydraulic conductivity increased from a mean value of 93.1 cm/h to 954.0 cm/h.

8 Grain size distribution analyzes

It is of interest to look at the grain size distribution of particles in the joint filling material of Test area 3. The size of the joint filling material used during construction is between 2 – 8 mm, so the particles less than 2 mm will be from the surrounding areas and wearing of the material. The construction took place in October – December 2014 (Møller-Pedersen, 2015) and the sample material was collected in the end of April 2016.

The upper 5 cm of the joint filling material were removed within a small area. A spoon was used to take out the joint filling material, so as you see in Figure 71 there is some gravel left on the edges around the holes.

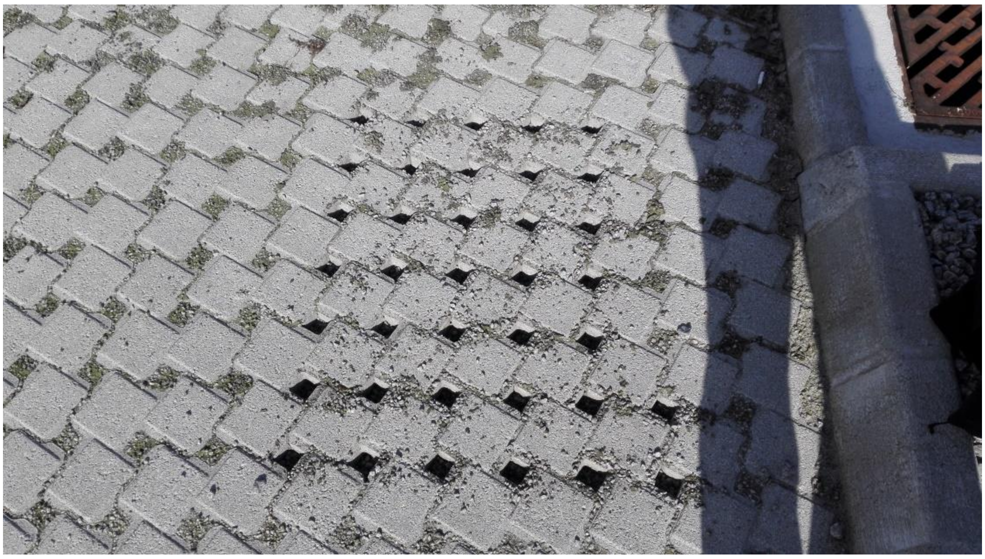

Figure 71 The sample area at Test area 3, Skjæveland. (Photo: Jens H. Trandem)

8.1 Grain size distribution analysis, procedure

The collected sample was dried in the oven for 24 hours at 105 °C to remove all the water. Since the sample had a total weight of 1663.58 g it was mixed and split into two samples with a weight of 831.51 g and 832.07 g. Table 2 in the (ISO, 2004) says that with the largest particle diameter of 8 mm, the minimum required mass for a sieving sample is 600 g.

Sieves with openings of 8 mm, 4 mm, 2 mm, 1 mm, 500 µm, 250 µm, 63 µm and sealed bottom where used. The setup of the machine is pictured in Figure 72. The machine was used for 10 minutes with amplitude of approximately 65.

Figure 72 The machine and sieves used for the analysis. (Photo: Jens H. Trandem)

8.2 Results

The formula for calculating the fraction of soil passing each sieve is given in chapter 6.1 (ISO, 2004).

$$f_n = \frac{m_1 + m_2 + .. + m_n}{m} * 100\%$$

Equation 14

f_n = fraction passing the sieve (%)
m_1 = mass of soil passing the smallest mesh size (g)
m_2, m_n = mass of soil passing the consecutive sieves, up to the sieve considered (g)
m_n = total dry mass of the specimen (g)

In Table 18 the weighted material left on the sieve and the fraction of total mass passing the sieve is calculated.

Table 18 Results from the sieving analysis

	Test 1		**Test 2**	
Sieve size [mm]	Weight material [g]	Fraction passing the sieve [%]	Weight material [g]	Fraction passing the sieve [%]
8	20.4	97.54	20.08	97.58
4	495.78	37.68	454.46	42.80
2	239.18	8.80	261.71	11.25
1	26.48	5.60	27.79	7.90
0.5	18.53	3.37	24.02	5.01
0.25	11.22	2.01	16.14	3.06
0.063	10.58	0.73	16.34	1.09
Bottom	6.08	0.00	9.06	0.00

The grains size distribution is calculated and plotted for both tests in Figure 73. From the tested material 90 % is between 2 – 8 mm which is the grain size used in the joint filling material. The amount of particles with diameter less than 2 mm is 10 %. This 10 % of the particles is not meant to be in the joint filling material and will reduce the infiltration capacity of the pavement.

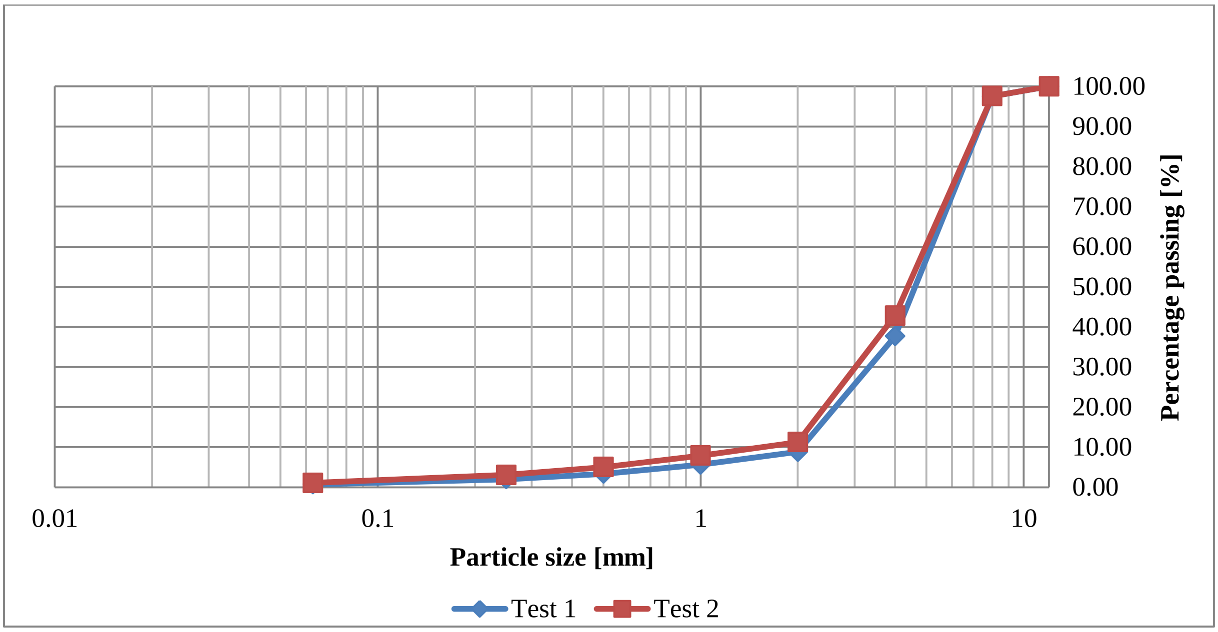

Figure 73 Grain size distribution curve of the joint filling material from Test area 3 at Skjæveland.

In Figure 74 the sieved samples are pictured to give a better understanding of the size of the particles. The 4 mm sample is in the bottom, so in Figure 74 a picture of 4 mm gravel is placed on top of 2mm to show all the samples in one picture.

Figure 74 The different grain size after sieving of the sample from Test area 3, Skjæveland. (Photo: Jens H. Trandem)

8.3 Quality check of the analysis

From chapter 5.1 in (ISO, 2004) the sieving is good as a test method if 90 % or more of the particles are larger than 0.063mm. This is the case for both test 1 with 99.3 % of the particles larger than 0.063 mm and for test 2 with 98.9 % of the particles larger than 0.063 mm.

From chapter 5.2.3.1 in (ISO, 2004) the masses retained on each sieve shall not exceed the values listed in Table 2 from the (ISO, 2004). Then the sample should be split in smaller portion and sieved separately. The total mass left on the 4 mm sieve and the 2 mm sieve is larger than the values in Table 2 from the (ISO, 2004), but it was not found necessary to split the samples and sieve them separately.

From chapter 5.2.3.2 (ISO, 2004) if the total mass weighted after sieving differs with more than 1 % from the total mass before sieving, the sieving shall be repeated. For both test 1 and test 2 the difference is less than 1%.

The results are quiet good when checked up against the quality criteria's from the (ISO, 2004) standard. It fails on one of them, but the interest in doing the grain size distribution is just to get an idea of how much fine particles there is in the joint filling material since the permeable pavement in Test are 3 was relatively clogged.

8.4 Conclusion

The material collected for the grain size distribution analysis is only from a small area of the pavement, but it is likely to believe that the result would have been similar from a bigger sample area. When the amount of particles less than 2 mm reach 10 % of the joint filling material for the upper 5 cm, the infiltration capacity of the permeable pavement is reduced to what is measured in Chapter 7. This infiltration rate is not sufficient to prevent water to pond on top of the pavement from time to time when the area draining to the permeable pavement is 12-13 times larger. If the upper 5 cm of the joint filling material is removed and refilled with new material, a lot of the fine particles will be removed and infiltration capacity restored.

9 Water Balance Model

When precipitation reaches the surface of the permeable pavement, the water is in the system and different processes take place in order to remove the water and prevent flooding. To determine the capacity of the permeable pavement and understand its function a water balance equation is necessary. The water balance equation can give indications for how well the test field is monitored and see if water goes elsewhere or if we are in control and able to track where the water is draining. This is important to determine the infiltration capacity over time. If water is escaping the system in some way, it is not possible to determine the infiltration capacity, can't tell if the pavement is performing well or if it is just the surroundings that are doing the job.

The most essential parameters are used in the water balance equation. This is precipitation, infiltration, evaporation and the runoff.

$$Q = P - I - E$$

Equation 15

Q – Water that is not infiltrated through the pavement and to the ground.
P – Precipitation [mm]
I – Infiltration [mm]
E – Evaporation [mm]

For test area 1 and 2, (Q) will be the inflow to SF1. This is water running off from top of the pavement and into the slot drains. For test area 3, (Q) will be the inflow to SF2 when the outflow from SF1 is subtracted. The evaporation is found from the Penman formula which gives the potential evaporation. The precipitation is registered for the area, and with all the other parameters in the equation, the infiltration can be found.

Since test area 1 and 2 are built with slot drains to collect the runoff from the surface, no water should be able to leave the pavement other than through the pavement itself or through the drain leading into the sandtrap/infiltration basin (SF1). Similar scenario will be for test area 3, where the runoff from the parking areas are draining down to the permeable part and curbstones are placed to prevent water from escaping further. When the infiltration capacity is reached the water will den rise and go into the drain (HS2) close by, which leads to the sandtrap/infiltration basin (SF2).

If a single rainfall event is analyzed the water retained in surface depressions could be added to the water balance equation. This water will not immediately evaporate, but stay on the surface a little longer. For the evaporation during a month or week, the water in the depressions at the permeable pavement will not make a difference. From chapter 2 (Ødegaard et al., 2012) the surface depression after a rainfall event can be set to 2.5mm for asphalt and hard surfaces.

9.1 Water balance equation for test area 3

When applying the water balance model on test area 3, it is used two different approaches.

1) The registered rainfall events with the highest intensity are used together with the measured infiltration capacity from Chapter 7 to check if excess runoff will occur.
2) The registered inflow to SF2 is used together with precipitation data to see if the infiltration capacity can be calculated.

9.1.1 Using the measured infiltration capacity and rainfall data to look at overflow

The known parameters for test area 3 are the precipitation and the potential evaporation. To look at the excess water going to SF2 the infiltration capacity measured in Chapter 7 is used. The lowest measured infiltration capacity for the permeable pavement at test area 3 is 78.9 cm/h.

In Table 19 some events with high rainfall intensity is listed.

Table 19 Events with the highest maximum value for rainfall intensity [mm/h] is listed. (Data from the test facility at Skjæveland and Multiblokk)

Date [DD.MM.YYYY]	Duration [min]	Mean [mm/h]	Max [mm/h]
01.08.2015	10	35.2	100
14.11.2015	4	35.6	79.9
02.12.2015	5	26.1	98.8
04.02.2016	6	33.2	99.1

Test area 3 has water from the approximately 12 times larger impermeable part of the parking area draining to its permeable area. Because of this, the rainfall intensity at test area 3 will feel 12 times larger. In Table 20 the measured rainfall intensity is multiplied by 12 to get the experienced rainfall intensity for test area 3.

Table 20 Experienced rainfall intensity [mm/h] for test area 3. (Data from the test facility at Skjæveland and Multiblokk)

Date [DD.MM.YYYY]	Duration [min]	Mean [mm/h]	Max [mm/h]
01.08.2015	10	422.4	1200
14.11.2015	4	427.2	958.8
02.12.2015	5	313.2	1185.6
04.02.2016	6	398.4	1189.2

The measured infiltration capacity of 789 mm/h is higher than the mean rainfall intensity and no inflow to SF2 will be registered. This fits well with the observation in PLC system where no inflow to SF2 is registered for any of the events.

9.1.2 Using measured overflow and rainfall data to look at infiltration capacity

Five events where inflow to SF2 from test area 3 is registered are analyzed. This is done, to find the actual infiltration rate during a rainfall event. The total time from the beginning of the rainfall to the end of inflow is registered. Then the total inflow to SF2 is calculated and the total precipitation from the beginning of a rain event to the end of the inflow. The total area of 1354 m^2 is accounted for in the total precipitation. The amount of water which is infiltrated is found, and the capacity is calculated for the permeable openings which is 12% of test area 3.

How the infiltration rate is calculated:

$$Infiltration\ capacity[\frac{cm}{h}] = \frac{(Tot.precip. - Tot.inflow)[m^3] * 100}{Area\ of\ test\ area\ 3[m^2] * 0.12 * time\ of\ the\ event[h]}$$

Equation 16

Table 21 Calculated infiltration capacity for the events where inflow to SF2 is registered.

Date [DD.MM.YYYY]	Total duration [h]	Tot. inflow SF2[m^3]	Tot. precipitation [m^3]	Infiltrated water[m^3]	Infiltration capacity [cm/h]
24.12.2015	3.57	2.88	17.31	14.43	32.39
26.01.2016	6.02	12.96	29.12	16.16	21.50
29.01.2016	2.58	6.26	9.53	3.28	10.17
08.02.2016	1.83	1.80	4.96	3.16	13.82
30.05.2016	0.91	2.48	9.40	6.92	60.94

The calculated infiltration rates for the events in Table 21 are lower than the measured infiltration of 78.9 cm/h. A theory for this observation is that the permeable pavement will have higher infiltration rate for short rainfall events, but for longer rainfall events the infiltration rate will decrease. If the amount of water and the time before inflow is registered to SF2 is compared with the infiltration capacity (see Table 22), the hypothesis doesn't match with the five events.

Table 22 Comparison of total precipitation before inflow and the time from start of precipitation to start of inflow with calculated infiltration capacity and reduction in runoff

Date [DD.MM.YYYY]	Total precipitation before inflow is registered [m^3]	Time from start of precipitation, to start of inflow [h]	Infiltration capacity [cm/h]	Reduction in runoff [%]
24.12.2015	12.00	2.77	32.39	83.36
26.01.2016	9.24	3.25	21.50	55.48
29.01.2016	3.11	1.00	10.17	34.36
08.02.2016	3.60	1.33	13.82	63.68
30.05.2016	4.028	0.91	60.94	73.62

Four of the events with the lowest calculated infiltration rate are from December to February. There could be something with the cold temperature that affects the infiltration even if there is not registered minus degrees. The calculated infiltration rate for May is much higher than the others and is close to the measured value from April.

9.1.3 Discussion of the size of outflow and infiltration capacity

When the infiltration capacity of 78.9 cm/h is used for the permeable pavement, there will be no outflow for the rain events in Table 19. If the infiltration capacity of the permeable pavements were 78.9 cm/h there would have been no registered inflow to SF2 for events in Table 21.

Some reasons for this could be:

- The measured infiltration capacity of 78.9 cm/h can be wrong, and it might be lower than measured.
- The rain events with high intensity are just a few minutes, so it is not enough water to reach up to the excess water drain.
- The events in Table 21 are over such a long period that the groundwater level will raise and be the limiting factor for the infiltration.
- There is still water in the structure from previous events.
- The infiltration capacity is reduced with time and amount of water infiltrated to the structure.

10 Suggestions for improvements and change in design of the test area

10.1 Test area 3

Test area 3 was built with solid curbstone in the lowest elevated part of the permeable pavement to make it easy to collect and measure the excess water from the permeable pavement. This resulted in some water ponding on top of the surface when the capacity was reached. Because the permeable pavement is built with a 2 percent slope, it is not functioning optimal when ponding starts to occur. Then there is only the lowest elevated part that infiltrates the water instead of the whole permeable area, which would have been the case if the test area was flat. The permeable concrete pavement will then clog faster in the lowest end, instead of equally getting stressed. From the results of the infiltrometer tests in Chapter 7.5, the pavement downstream and upstream have a small difference in the calculated saturated hydraulic conductivity. Where the water does not pond on top the Ksat value is a bit higher.

If the slope is there to make sure that the pavement is never ponded, another design would be recommended.

1. Build a surface water drain channel to take away the excess water as soon as it gets to the lowest part of the pavement. Then lead the water to an infiltration basin.
2. Make some holes in the curbstone and let the water run through. Need to make sure that there is some control of the water further downstream.

10.2 Control of water flow in the system

Today the test area in Sandnes measures the amount of rain, the excess water from the pavement and many other parameters. It would have been interesting if the bottom of the structure was buildt with water tight membrane to collect all the water infiltrating through the structure. If this water is measured before it is infiltrated to the surrounding area, it would be possible to see how much water that evaporates and how much water that remain in the structure after a rain event.

With the data collected it is only calculated potential evaporation for the test area. It is not possible to calculate more specific evaporation without having total control of the water.

10.3 Solutions to prevent clogging of the permeable pavement

Clogging of the permeable pavement will always be of great concern. When dust and fine particles are washed from the approximately 12 times larger impermeable parking area and down to the permeable parking area, it will clog fast if nothing is there to stop the particles from entering the permeable area.

Solutions for how to remove most of the sediments before the water enter the permeable area could be tested. The behavior without any preventive measures is registered and could be compared to the methods tested. It could be technical solutions built in to the system, or just maintenance like regularly sweeping the pavement.

Some suggestions:

1) Install an infiltration/sandtrap basin to collect the first flush from the impermeable parking area. If the same design as for Vagleskogveien is used, this will be easy to maintain and clean. When it is full the water goes around and enters the permeable area.
2) Instead of leading the water directly to the permeable pavement, the water can go to each of the sides where grass and vegetation will hold back the sediments, before the water enter the permeable area.

10.4 Infiltration manholes

In SF1 and SF2 there is registered a rise in water level with no measured inflow. To find out if it is groundwater leaking into the sandtrap/infiltration basin through the bottom and the submerged outlet, measurements of groundwater level just outside SF1 and SF2 can be done (personal communication Per Møller-Pedersen, 2016). The sealing of the sandtrap/infiltration basins should also be tested, to make sure that the only water entering the sandtrap/infiltration basins is excess water from the test areas. When the flow in and out of the infiltration manholes is under control, the measured flow may behave as expected.

The flow meters are dimensioned to handle greater amount of water, so a possibility could be to replace them or calibrate for smaller flow.

10.5 Construction

The permeable area is sensitive for dust and particles clogging the upper 5 cm of the joint filling material. During construction of the parking area, sand and fine particles used for the impermeable part was carried by the wind and over to the permeable part (personal communication Per Møller-Pedersen, 2016). No extra precaution where taken in advance to prevent this from happening. During construction it would be smart to protect the permeable pavement from this type of load if possible. The permeable area at the test facility in Sandnes is small compared to the impermeable area, and should be easy to protect with some kind of cover for a short period. It is hard to estimate how big impact this have on the overall performance, but permeable pavements are sensitive for finer particles and everything that can reduce the load will help to prolong the lifetime before refilling of joints and maintenance work is required.

10.6 PLC system

The PLC system is good for a quick view at the data and to see how the measured parameters behave. It is not well suited to analyze huge amount of data in more detail. The markers available to move around is useful for single events, but excel is preferred. More options for sampling intervals in the data extracted to excel will be useful. For flow in and out of the sandtrap/infiltration basins and the precipitation, every minute and every five minutes is recommended to register change in flow and intensity.

The data extracted to datafarm and downloaded to excel from the PLC system should be controlled. As the system is today, the differences between the data in excel and the data in the PLC system is confusing without any explanation. An explaining text for how the data is extracted and the correct units would help to solve this problem.

Something to prevent failure to the system should be looked into. Almost every month with collected data, there are some days missing. It should be possible to make the system more robust and then a more complete set of data will be available.

11 Conclusion and further work

The test facility in Sandnes is well equipped with sensors to measure flow from test areas and between basins, air temperature, water level in the basins, precipitation intensity, temperature in the ground, wind speed, relative humidity and air pressure. Further it is planned to install wells to measure groundwater level. All the measured parameters are important to understand the functions of the permeable pavement and with data for the groundwater level it will be possible to gain more control of the water in the system and why there is a rise of water level in the basin when no inflow is detected.

The quality of the data collected seems to be good most of the time, but improvements are necessary to make them more reliable. It is too many events where the collected data doesn't reflect the behavior of the system. As discussed earlier the flow in and out of SF2 is sometimes hard to explain when compared to rainfall events. More accurate calibration of the flow meters or new solutions to measure the flow should be considered. The precipitation data extracted to datafarm and downloaded to excel should be corrected so they match the units. How the flow data for SF1 and SF2 is extracted to datafarm and downloaded to excel should also be controlled.

Precipitation is an important parameter registered in the PLC system. The quality of the rainfall data needs to be good. I will recommend a closer look into the setup of the rainfall measurements. A second rain gauge could be installed to compare the measured precipitation and verify that the values are accurate. Rainfall events less than one hour are common, and the change in intensity could happen fast. Therefor the values for rainfall intensity extracted to excel and the datafarm should have a sampling interval of at least every five minute and maybe even every minute to register the change in intensity. The tools for extracting data directly from the PLC system are not suited if a lot of data needs to be processed. There are no options to sort out corrupted data and the navigation and reading of data is time consuming.

The measured saturated hydraulic conductivity for test area 3 together with the grainsize distribution analysis, indicated that the joint filling material between the concrete pavement units where clogged by fine particles. The infiltration capacity measured, ranged from 78.9 – 107.2 cm/h for the untouched joint filling material. When the upper 5 cm of the joint filling material were removed, the capacity was measured to be from 670.9 – 1292.8 cm/h. It is reason to believe that if the upper 5 cm with joint filling material is removed and replaced, the permeable pavement will regain a lot of the initial infiltration capacity.

The infiltration through the permeable pavement is calculated from events where inflow is registered to SF2 from test area 3. This infiltration capacity is only between 10.2 – 60.9 cm/h and less than the measured values. The method used to measure the saturated hydraulic conductivity might not be suitable for this type of testing, or something else is causing the low infiltration during longer periods of rain.

From the study of the test facility in Sandnes, it looks like a system with permeable pavement and infiltration basins can reduce and infiltrate the runoff from rainfall events with different intensity and duration. The infiltration capacity is good, but it has some limitations. When it is built to infiltrate water from a total area much bigger than its own, it cannot handle all the events and the time before clogging will be reduced. If the purpose is to infiltrate all the water, the permeable area compared to impermeable area should be close to each other in size. As a reduction of runoff it works very well even for larger areas draining to a small area of permeable pavement. For five of the incidents where excess runoff from test area 3 where detected, the reduction in runoff was from 34 – 83 %. An evaluation of how much excess runoff that is acceptable will be a part of the decision before constructing a permeable pavement.

For further work, some suggestions are listed below.

1) It would be interesting to look at measurements of the groundwater level and if it affects the infiltration and the capacity of the permeable pavement.
2) When removing and refilling of the joint material in test area 3 is done, the regained infiltration capacity would be interesting to measure. Also how it changes with time, will it clog at the same rate?
3) The system with sandtrap /inifiltration basins in Vagleskogveien could be looked into. Control the capacity of the basins and see if this is enough to handle runoff water from the road.

12 References

2013. *ISCORD 2013 : Planning for Sustainable Cold Regions,* Reston, VA, USA, ASCE.

APLISENS Hydrostatic level probes SGE-25 and SGE-16.

BEAN, E. Z., HUNT, W. F. & BIDELSPACH, D. A. 2004. Study on the Surface Infiltration Rate of Permeable Pavements. *Critical Transitions in Water and Environmental Resources Management.*

BECKER, M. A. 2015. Beregning av mettet hydraulisk konduktivitet ved bruk av MPD-infiltrometer, for vurdering av frakopling av taknedløp.

BEELDENS, A. & VIJVERMAN, L. 2015. OVER 15 YEARS OF EXPERIENCE OF WATER-PERMEABLE PAVING BLOCKS IN BELGIUM: HOW LEGISLATION PROMOTES THE APLLICATION.

BERRY, C. T. 1995. *Control of Urban Runoff Through the Use of Permeable Pavements.* PhD.

BRATTEBO, B. O. & BOOTH, D. B. 2003. Long-term stormwater quantity and quality performance of permeable pavement systems. *Water Research,* 37**,** 4369-4376.

BROWN, R. A. & BORST, M. 2015. Quantifying evaporation in a permeable pavement system. *Hydrological Processes,* 29**,** 2100-2111.

BRUCE, K. F. 2005. Porous Pavement Structure. *Porous Pavements.* CRC Press.

CRA.CLIMA.EVAPOTRANSPIRATION. 2009. *Soil heat flux* [Online]. http://www.apesimulator.org/help/models/evapotranspiration/Soil_heat_flux.html. [Accessed 14.03.16 2016].

DANNEVIG, P. & HARSTVEIT, K. E. 2009. *Luftfuktighet* [Online]. https://snl.no/luftfuktighet: Store norske leksikon. [Accessed 28.05.16 2016].

DYE, A. D. 2014. *Design Considerations for Vegetated Permeable Pavement* [Online]. https://continuingeducation.bnpmedia.com/article.php?upgrade=new&L=184&C=969. [Accessed 02.03.2016 2016].

DYRRDAL, A. V. & FØRLAND, E. 2016. Rapport om klima ved Gardermoen.

EISENBERG, B., LINDOW, K. C. & SMITH, D. R. 1. Design Considerations Common to All Permeable Pavements. *Permeable Pavements.* American Society of Civil Engineers (ASCE).

EISENBERG, B., LINDOW, K. C. & SMITH, D. R. 2. Porous Asphalt and Permeable Friction Course Overlays. *Permeable Pavements.* American Society of Civil Engineers (ASCE).

EISENBERG, B., LINDOW, K. C. & SMITH, D. R. 3. Pervious Concrete. *Permeable Pavements.* American Society of Civil Engineers (ASCE).

EISENBERG, B., LINDOW, K. C. & SMITH, D. R. 4. Permeable Interlocking Concrete Pavement (PICP). *Permeable Pavements.* American Society of Civil Engineers (ASCE).

EISENBERG, B., LINDOW, K. C. & SMITH, D. R. 5. Grid Pavement. *Permeable Pavements.* American Society of Civil Engineers (ASCE).

FARLEY. 2016. *Permeable Pavement Design for Stormwater Management Compliance* [Online]. http://farleypavers.com/permeable-paver-installation/. [Accessed 12.02.2016 2016].

FERGUSON, B. K. 2005. *Porous pavements: Integrative Studies in Water Management and Land Development*, CRC Press.

FISCHER, C. J. & ZUBECK, H. 2013. Comparison of Theoretical Freezing Depths of Permeable and Conventional Pavements.

GUTHRIE, S. W., GURNEY, L. R., ANDERSON, D. O., GUTHRIE, A. C. & BEDFORD, C. 2013. Comparison of Surface and Subsurface Winter Temperatures in Pervious and Conventional Concrete Pavements.

HANSEN-BAUER, I., FØRLAND, E. J., HADDELAND, I., HISDAL, H., MAYER, S., NESJE, A., NILSEN, J. E. Ø., SANDVEN, S., SANDØ, A. B., SORTEBERG, A. & ÅDLANSVIK, B. 2015. Klima i Norge 2100.

HANSEN, K. 2008a. Asphalt pavements for Stormwater Management. *Design, Construction and Maintenance Guide,* National Asphalt Pavement Association, Information Series 131. Lanham, Maryland**,** 24.

HANSEN, K. 2008b. Porous Asphalt pavements for Stormwater Management. Design, Construction and Maintenance Guide. National Asphalt Pavement Association, Information Series 131. Lanham, Maryland.

HARSTVEIT, K. E. & STORE NORSKE LEKSIKON. 2009. *Lufttrykk* [Online]. https://snl.no/lufttrykk: Store norske leksikon. [Accessed 28.05.16 2016].

ILLGEN, M., HARTING, K., SCHMITT, T. G. & WELKER, A. 2007. Runoff and Infiltration characteristics of permeable pavements – Review of an intensive monitoring program.

INTERACTIVE, P. 2007. *Percent Fractured Face Test* [Online]. http://www.pavementinteractive.org/article/percent-fractured-face-test/. [Accessed 13.02.2016 2016].

INTERACTIVE, P. 2009. *Subgrade* [Online]. http://www.pavementinteractive.org/article/subgrade/. [Accessed 13.02.2016 2016].

INTERACTIVE, P. 2011. *Los Angeles Abrasion* [Online]. http://www.pavementinteractive.org/article/los-angeles-abrasion/. [Accessed 13.02.2016 2016].

ISO 2004. Geotechnical investigation and testing - Laboratory testing of soil. *Part 4: Determination of particle size distribution.*

KORKEALAAKSO, J., KUOSA, H., NIEMELÄINEN, E. & TIKANMÄKI, M. 2014. Review of pervious pavement dimensioning, hydrological models and their parameter needs. State-of-the-Art.

KROHNE 2013. Waterflux 3000 Handbook.

KUMAR, K., KOZAK, J., HUNDAL, L., COX, A., ZHANG, H. & GRANATO, T. 2016. In-situ infiltration performance of different permeable pavements in a employee used parking lot – A four-year study. *Journal of Environmental Management,* 167**,** 8-14.

KÖZ, U. & SIGRIST, A. 2015. Overvannshåndtering og flomsikring. *In:* VIAK, A. (ed.).

LEMING, L. M., MALCOM, H. R. & TENNIS, P. D. 2007. *Hydrologic design of pervious concrete* [Online]. http://www.rmcfoundation.org/images/PCRC%20Files/Hydrological%20&%20Environmental%20Design/PC_DesignManual.pdf. [Accessed 15.12.2013 2013].

LIPPESTAD, H. 2009. *Verdens varmeste dag* [Online]. http://www.yr.no/artikkel/verdens-varmeste-dag-1.6727109. [Accessed 27.05.16 2016].

LTD, G. S. 2015. *GroundGridz - Permeable Paving Grids* [Online]. https://www.specifiedby.com/groundtrax-systems/groundgridz-permeable-paving-grids. [Accessed 02.03.2016 2016].

LUCKE, T. & BEECHAM, S. 2011. Field investigation of clogging in a permeable pavement system. *Building Research & Information,* 39**,** 603-615.

MAGNUSSEN, K., WINGSTEDT, A., RASMUSSEN, I. & REINVANG, R. 2015. Kostnader og nytte ved overvannstiltak.

MARASCO, D. E., CULLIGAN, P. J. & MCGILLIS, W. R. 2015. Evaluation of common evapotranspiration models based on measurements from two extensive green roofs in New York City. *Ecological Engineering,* 84**,** 451-462.

MERKESDAL, I. A. 2015. eureka.

METEOROLOGISK INSTITUTT. 2016. *eKlima* [Online]. eklima@met.no. [Accessed 10.03.16 2016].

MINNESOTA STORMWATER MANUAL. 2013. *Minnesota Pollution Control Agency* [Online]. http://stormwater.pca.state.mn.us/index.php/Main_Page. 22.11.2013].

MOREO, M. 2009. *Evapotranspiration Studies in Nevada* [Online]. http://nevada.usgs.gov/water/et/measured.htm. [Accessed 14.03.16 2016].

MYHR, K. 2013. Dimensjonering og bruk av permeable dekker med belegningsstein.

MØLLER-PEDERSEN, P. 2015. Demonstration and measurement of infiltration through permeable concrete block pavements.

MØLLER-PEDERSEN, P. 2016. Infiltrasjon og fordrøying av overvann, Presentasjon ved EVU-kurset «Overvanns og ledningsteknologi 2016» *In:* AS, S. A. (ed.).

NEMIROVSKY, E. M., WELKER, A. L. & LEE, R. 2013. Quantifying Evaporation from Pervious Concrete Systems: Methodology and Hydrologic Perspective. *Journal of Irrigation and Drainage Engineering,* 139**,** 271-277.
NGU. 2016. *Hva er grunnvann?* [Online]. http://www.grunnvanninorge.no/grunnvann_hva_er.php. [Accessed 14.03.16 2016].
PAUS, K. H., MUTHANNA, T. M. & BRASKERUD, B. C. 2015. The hydrological performance of bioretention cells in regions with cold climates: seasonal variation and implications for design.
PEZZANITI, D., BEECHAM, S. & KANDASAMY, J. 2009. Influence of clogging on the effective life of permeable pavements. *Proceedings of the Institution of Civil Engineers - Water Management,* 162**,** 211-220.
ROHNE, R. J. & LEBENS, M. A. 2009. Subgrade Temperature and Freezing Cycles in Pervious Pavements. *Cold Regions Engineering 2009.*
ROSEEN, R. M. & BALLESTERO, T. P. 2008. Porous Asphalt Pavements for Stormwater Management in Cold Climates.
SCHOLZ, M. & GRABOWLECKI, P. 2007. Review of permeable pavement systems. *Building and Environment,* 42**,** 3830-3836.
SKJÆVELAND 2015. Skjæveland Produktkatalog 2015/2016.
SKJÆVELAND GRUPPEN 2015/2016. La det regne.
SMITH, D. R. 2006. Permeable Interlocking Concrete Pavements.
STATENS VEGVESEN 2014. Håndbok N200 Vegbygging. www.vegvesen.no.
TRONDHEIM KOMMUNE 2015. Beregning av overvannsmengde. Dimensjonering av ledning og fordrøyningsvolum.
UIO. 2011. *Solstråling* [Online]. http://www.mn.uio.no/ibv/tjenester/kunnskap/plantefys/leksikon/s/solstraaling.html. [Accessed 10.03.16 2016].
VAISALA Vaisala RAINCAP Sensor Technology.
VAISALA 2012. User's Guide Vaisala Weather Transmitter WXT520.
WATER ENVIRONMENT FEDERATION (WEF) 2012. *Design of Urban Stormwater Controls*.
YONG, C. F., MCCARTHY, D. T. & DELETIC, A. 2013. Predicting physical clogging of porous and permeable pavements. *Journal of Hydrology,* 481**,** 48-55.
ZOTARELLI, L., DUKES, M. D., ROMERO, C. C., MIGLIACCIO, K. W. & MORGAN, K. T. 2010. Step by Step Calculation of the Penman-Monteith Evapotranspiration (FAO-56 Method)1.
ØDEGAARD, H., HEISTAD, A., MOSEVOLL, G., LINDHOLM, O. G., ØSTERHUS, S. W., THOROLFSSON, S. T. & SÆGROV, S. 2012. *Vann- og avløpsteknikk*.

Appendix A

Given tasks for the master thesis

NTNU

Fakultet for ingeniørvitenskap og teknologi
Institutt for vann- og miljøteknikk

Dato	Referanse
14.01.2016 rev 02.06.2016	

Masteroppgave

Til:	Jens Hissingby Trandem
Kopi til:	Tone Merete Muthanna, Per Møller Pedersen, Aage Gjesdal
Fra:	Sveinung Sægrov
Signatur:	

Testing av infiltrasjonssystem for overvann

Masteroppgave VA-teknikk 2016 Jens Hissingby Trandem

Bakgrunn

For å møte økende nedbørsavrenning i bebygde områder utvikles nye teknikker for lokal håndtering av overvann. Hensikten er å ta vare på overvann som en ressurs og unngå overbelastning av nedstrøms ledningsnett. Skjævelandgruppen i Sandnes produserer rør og belegningsstein og utvikler nye konsepter for overvannshåndtering. De er opptatt av å sikre best mulig infiltrasjon og fordrøyning av overvann slik at avrenningen reduseres og utjevnes. Der er særlig opptatt av permeable løsninger på trafikkarealer og infiltrasjonskummer oppstrøms del av avløpssystemer, og har bygget ut et prøvefelt med omfattende instrumentering for målinger som består av nedbør, temperaturer i bærelag for parkeringsplass og vannivå i fordrøyningsanlegg. Oppgaven består i bearbeide målingene og diskutere måleprogrammet samt gi anbefalinger for hvordan slike prøvefelt bør dokumenteres.

Norges teknisk-naturvitenskapelige universitet

Dato 14.01.2016 rev 02.06.2016

Referanse

Spesifisert oppgave:

1. Presenter prøveanlegget på Skjæveland inklusive permeabelt dekke, undergrunn, grunnvannsforhold, infiltrasjonskummer og fordrøyningsanlegg og drøft hvordan de enkelte elementene bidrar til reduksjon og utjevning av avrenning fra feltet.
2. Samle inn og bearbeide resultater fra måleprogrammet mht. nedbør, avrenning, grunnvannsstand og temperatur i grunnen. Foreslå lokalisering av flere brønner for grunnvannsobservasjoner og behandle data fra målingene.
3. Bygg en vannbalansemodell for forsøksfeltet som inkluderer hovedelementene i den hydrologiske syklusen, nedbør, fordampning, infiltrasjon, fordrøyning og bortledning. Undersøk om programmet SHYFT fra Statkraft kan brukes for dette formålet.
4. Drøft funksjonsevnen til systemet og indentifiser mulige svake punkter og flaskehalser.

Assistanse

Professor Sveinung Sægrov, førsteamanuensis Tone Muthanna, Institutt for vann og miljøteknikk, NTNU vil være hovedveiledere for denne oppgaven. Opplysninger om feltet og måledokumentasjon vil bli fremskaffet av Per Møller Pedersen fra Storm Aqua i Skjævelandgruppen. Prosjektet inngår i Klima 2050

Presentasjon og leveranse

Prosjektrapporten skal leveres i henhold til gjeldende regler. Studenten er selv økonomisk ansvarlig for 3 kopier som leveres til instituttet. Ekstra kopier som er bestilt av instituttet skal betales av instituttet.

Leveringsfrist 10.juni 2016.

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Appendix B

Matlab code for calculation of saturated hydraulic conductivity

```
data = xlsread('mpddata.xlsx', 'Input');                % Reads input matrix
n = size(data);                                          % Finds the size of
the input matrix
n = n(1);                                                % Finds the number of
rows (n)
t = data(1:n,3)*60*60*24;                                % Finds time time
matrix and convert values from days to seconds
h = data(1:n,2);                                         % Finds head matrix

dteta=data(2,1)-data(1,1);                                % Finds differences
in volumetric water content (dteta)
Lmax = data(3,1);                                         % Finds length of
device below surface
rd = data(4,1);                                           % Finds radius of
device
H = data(5,1);                                            % Finds phase one
initial height
K = 0.001;
C = -1000;

tt(1,1)=0;                                               % Sets first
intermediate time value to zero
i=2:n;                                                   % Prepare integers for
the remaining intermediate time values
tt(i,1)= (t(i,1)-t(i-1,1))*0.5+t(i-1,1);                 % Finds intermediate
time values (tt)
hh = spline(t,h,tt);                                     % Cubic spline
interpolation to find intermediate h values (hh)

q(1,1)=0;                                                % Sets first
difference value to zero
qt(1,1)=0;                                               % Sets first time
difference value to zero
qh(1,1)=0;                                               % Sets first head
difference value to zero

i = 2:n;                                                 % Prepare integers for
the remaining intermediate time values
qt(i,1) = tt(i,1)-tt(i-1,1);                             % Fills in remaining
time difference values (qt)
qh(i,1) = hh(i,1)-hh(i-1,1);                             % Fills in remaining
head difference values (qh)

for i=2:n,                                               % Calculate difference
values (q)

    q(i,1)=qh(i,1)/qt(i,1);

end

i=1;
while(i<n+1)                                             % Using Newton-Rhapson
to find R values
    x = 1;
    ii = 1;
```

```
    while(ii<100000)

    f1=2*x^3+Lmax*3*x^2-Lmax^3-2*(rd/2)^3-3*rd^2*(H-hh(i,1))/(dteta);
    ff = 6*x^2+Lmax*6*x;
    x = x-f1/ff;
    ii =ii +1;
    R(i,1)=x;

    end
i=i+1;
end


for i=2:n,                                              % Calculate difference
values (q)

    q(i,1)=qh(i,1)/qt(i,1);

end

i = 1;
while(i<n+1)
    if R(i,1)<(Lmax^2+rd^2)^0.5, R(i,1)=0;              % Calculate R values
(R)
    end
    i=i+1;
end



i = 2:n;                                                % Prepare integers for
the remaining intermediate time values
dt1(i,1)=tt(i,1)-tt(i-1,1);                             % Fills in remaining
time difference values (qt)




for i=1:n,
    if R(i,1)<10^-10, ss=i;                % Calculate R values (R)
    end
end




KC0 = [0.01,-100];
f = @(KC)optt(KC,n,Lmax,dteta,R,rd,hh,dt1,ss);
[KC,f] = fminsearch(f,KC0);

Results(1,1)=KC(1);
Results(1,2)=KC(1)*60^2;
Results(1,3)=KC(2);
Results(1,4)=f;
Results(1,5)=n-ss-1;
Results(1,6)=sqrt(f/(n-ss-1));
```

```
i = 2:n;                                                  % Prepare integers for
the remaining intermediate time values
dh1(i,1)=-hh(i,1)+hh(i-1,1);                              % Fills in remaining
time difference values (qt)

KC0 = [0.01,-100];
f = @(KC)opth(KC,n,Lmax,dteta,R,rd,hh,tt,dh1,ss);
[KC,f] = fminsearch(f,KC0);

Results(2,1)=KC(1);
Results(2,2)=KC(1)*60^2;
Results(2,3)=KC(2);
Results(2,4)=f;
Results(2,5)=n-ss-1;
Results(2,6)=sqrt(f/(n-ss-1));


'Estimated parameters:'

format shortG

ForExcelSheet(1,1)=Results(1,1);
ForExcelSheet(2,1)=Results(1,2);
ForExcelSheet(3,1)=Results(1,3);
ForExcelSheet(4,1)=Results(1,4);
ForExcelSheet(5,1)=Results(1,5);
ForExcelSheet(6,1)=Results(1,6);
ForExcelSheet(7,1)=Results(2,1);
ForExcelSheet(8,1)=Results(2,2);
ForExcelSheet(9,1)=Results(2,3);
ForExcelSheet(10,1)=Results(2,4);
ForExcelSheet(11,1)=Results(2,5);
ForExcelSheet(12,1)=Results(2,6);

ForExcelSheet

disp('    Ksat[cm/s]      Ksat[cm/h]  Cap.Suc.[cm]   Sum of error    Obs.
RMS error')

disp(Results)

if Results(1,4)<Results(2,4), 'Delta T optimization gives smallest error:'
else 'Delta H optimization gives smallest error: '
end

if Results(1,4)<Results(2,4), Ksat=Results(1,2)
else Ksat=Results(2,2)
end

if Results(1,4)<Results(2,4), C=Results(1,3)
else C=Results(2,3)
end
```

Appendix C

Excel sheet with input data from MPD measurements used by the Matlab code in Appendix A

Parameters		Head [cm]	Time
Initial volumetric water content [%]	20.0 %	45	00:00:00
Final volumetric water content [%]	50.0 %	37	00:00:05
Length of device below surface [cm]	5.0	31	00:00:10
Radius of device [cm]	4.3	26	00:00:15
Phase one initial height [cm]	45.0	21.5	00:00:20
		17	00:00:25
		13	00:00:30
		9.5	00:00:35
		7	00:00:40
		5	00:00:45