Assessing and modeling the hydrological performance of DIT sewers

A case study at Kuiperstraat and H.A.J.M. Schaepmanstraat, Gouda, Netherlands

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Delft – August 2019

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By

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in partial fulfilment of the requirements for the degree of

Master of Science
in Civil Engineering

at the Delft University of Technology,
to be defended publicly on Friday August 23, 2019 at 3:30 PM.

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This thesis is confidential and cannot be made public until August 23, 2019.

TU Delft

Preface

This study was conducted as a graduation research for the Master Program "Civil Engineering" at the Delft University of Technology. The subject for this MSc thesis was established in cooperation with the municipality of Gouda and Deltares as a part of the Dutch coalition "Stevige stad op slappe bodem".

For over a year, I have worked on this particular subject, of course with the usual ups and downs. This study has given me motivation and still interests me thanks in part to the balance between theoretical and practical work. I learned a lot during this study, varying from working in the "real world" with contractors, setting up a fieldwork independently and learn to think and act responsibly on an academic level. I would like to thank the TU Delft, Deltares and the municipality of Gouda for giving me the opportunity to work on this subject.

Furthermore, I want to thank my supervisors Frans van de Ven and Thom Bogaard for their guidance throughout this MSc research. I thank Jan Prinsen for his advice and guidance within the municipality of Gouda. I also thank Jeroen Langeveld and Riemer for their input in this study. I would like to thank Gebroeders van Vliet, especially Harold who was fun to work with and of great help during the performed tests. Further, I would like to thank everyone who in some manner helped me in this project, so family, friends, fellow students, or basically everyone who had to put up with me during this thesis.

Summary

Due to its foundation on a peat layer, Gouda deals with land subsidence. This has caused, and still causes problems in the inner city and other neighborhoods of Gouda, mostly due to the compaction of the peat caused by an increasing human-induced load. This subsidence leads to damage to infrastructures and houses and consequently high costs for the municipality of Gouda. It also strongly influences the water and groundwater management in Gouda. To avoid flooding, the surface water and groundwater table was already lowered several times in the past. Further lowering would lead to extra land subsidence and the exposure of wooden pile foundations to oxygen, which start to rot. Locally, groundwater can have very low levels in dry periods, worsening these problems. To regulate the groundwater table at vulnerable locations in Gouda, several Drainage-Infiltration-Transport (DIT) sewers were constructed. These pipes were placed in a gravel casing just below the intended groundwater level and discharge stormwater to the surface water in case of rainfall. If the groundwater level is too low, these sewers can infiltrate water into the soil and restore the groundwater level. If the groundwater level is too high, the sewer acts like a subsurface drain and discharges the groundwater to the surface water.

Research on the operation of DIT sewers is lacking. This study was initiated in order to know the hydrological performance of DIT sewers and have a better understanding of the processes influencing its operation. Two case studies were done on chosen locations in Gouda, namely in the Kuiperstraat in the inner city and in the H.A.J.M. Schaepmanstraat, Korte Akkeren. To know the potential infiltration of rainwater via the pavement, influencing the operation of the DIT sewer, inundation tests were performed on both locations. These tests resulted in an infiltration capacity of the pavement of 3,8 mm/h and 29 mm/h for the Kuiperstraat and Schaepmanstraat, respectively.

To know the infiltration and drainage capacity of the specific DIT sewers, an infiltration test and a drainage test were performed on a specified segment of the pipe. The DIT sewer of the Kuiperstraat showed an infiltration capacity up to 84,5 L/h on an infiltration area of 9,3 m^2. A k-value, defined as the infiltration rate over the area of infiltration (the perforated pipe wall surface) at a given potential difference between the water level in the pipe and the groundwater level at 10 cm perpendicular to the DIT sewer pipe, is 9,0 $L/m^2/h$ at dH = 0,28 m. The drainage capacity of this DIT sewer can be up to 33,0 L/h, corresponding with a k-value of 3,5 $L/m^2/h$ at dH = 0,24 m. From these values and groundwater analysis can be concluded, that the DIT sewer in the Kuiperstraat is fulfilling its intended purpose.

The infiltration test performed on a segment of DIT sewer in the Schaepmanstraat showed an infiltration capacity up to 2,4 L/s on an infiltration area of 18,6 m^2. The corresponding k-value is 465 $L/m^2/h$ at dH = 0,10 m. The drainage capacity could not be exactly measured but is approached with groundwater modeling in Hydrus2D and estimated on 0,5 L/s, corresponding with a k-value of 101 $L/m^2/h$ at dH = 0,03 m. From these values and groundwater analysis can be concluded, that the DIT sewer in the Schaepmanstraat is excellently fulfilling its intended purpose. The big difference between with the Kuiperstraat is largely caused by the difference in subgrade: silty fine sand in the Kuiperstraat versus pumice in the Schaepmanstraat.

A groundwater flow model was built in Hydrus2D to better understand the governing processes during the tests at both locations. The model was calibrated with the field data and the soil hydraulic parameters K_s and θ_s (saturated hydraulic conductivity and saturated volumetric water content) were found. This model showed a high permeability of the pumice subgrade soil in the Schaepmanstraat compared to a low permeability at the Kuiperstraat, which leads to more infiltration/drainage capacity and faster regulation of the groundwater table in the Schaepmanstraat. Model simulations further showed the importance of the drain envelope which consisted of Argex granules. It also revealed anisotropy in the soil of the Kuiperstraat, presumably caused by the presence of silt in the sandy soil.

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Introduction

Gouda is a city in the Rine-Meuse Delta and has to deal with land subsidence since its founding in 1272, due to settling of the peat layer on which it is located. Most historical buildings from the 16^{th} till the 20^{th} century in the inner city have foundations on steel and are hence subsiding with the soil. Therefore, the surface – and groundwater table was already reduced with half a meter in the past. Some buildings, however, are founded on (wooden) piles. This results in uneven subsidence of buildings, which has major damage as a consequence. In case of low groundwater levels, this land subsidence is accelerated and wooden piles are exposed to oxygen and start to rot. Further lowering of the groundwater table is, therefore, not an option. Consequently, the difference between the surface level and the groundwater level becomes smaller over time, leading to flooding at certain locations due to inundation from surface water and flooding in cellars and crawl spaces. Since 2014, several parties are working together to conquer these problems. This coalition "Stevige stad op slappe bodem" consists of the municipality of Gouda, Hoogheemraadschap van Rijnland, Deltares, Rijkswaterstaat, KCAF (Kenniscentrum Aanpak Funderingsproblematiek), the Delft University of Technology, Stichting RIONED and STOWA. Also together with amongst others Royal Haskoning and Wareco solutions are developed to secure the future of Gouda.

One solution to regulate the groundwater table is the construction of several Drainage-Infiltration-Transportation (DIT) sewers in the inner city. These sewers discharge stormwater in case of heavy rainfall and drain the surplus of stormwater to the surface water. If the groundwater level is too low, these sewers can infiltrate water into the soil and restore the groundwater level. If the groundwater level is too high, the sewer acts like a drain and discharges the groundwater to the surface water. However, after the constructing of these DIT sewers, there has never been a check if they fulfill their intended function, contribute to the groundwater level regulation and are consequently worth the investment.

The main research question in this thesis reads:

What is the current hydrological performance of the DIT sewer and how can this performance be improved?

The secondary research questions read:

1. What is the current knowledge and experience on the performance of DIT sewers?
2. What is the rainwater infiltration through the pavement above the DIT sewer in the Kuiperstraat and Schaepmanstraat?
3. What is the infiltration and drainage capacity of the DIT sewer in the Kuiperstraat and Schaepmanstraat and is it fulfilling its intended purpose?
4. What is the influence of the soil composition on the performance of the DIT sewer in the Kuiperstraat and Schaepmanstraat?
5. What other processes/parameters do influence the performance of a DIT sewer?

The main aim of this thesis is to test the DIT sewer on its infiltration and drainage capacity. Two situations will be studied in detail, namely in the Kuiperstraat in the inner city of Gouda and the H.A.J.M. Schaepmanstraat in Korte Akkeren, Gouda. To get an estimation of the rainwater infiltration through the pavement into the soil and, consequently, to the DIT sewer, an inundation test was performed on a part of these streets. Since both locations have a different soil composition, a comparison can be made on how the different characteristics influence the performance of a DIT sewer. After the fieldwork, a model was made in Hydrus 2D to mimic the performed tests and in this manner confirm and improve our understanding of the governing processes by finding and comparing the soil hydraulic parameters at both locations.

To achieve the aim of this study a literature study was done on the background of the problems in Gouda focusing on its history, land subsidence and drivers, and foundations in the first chapter. The second chapter dives into the theory and practice behind the DIT sewers. Chapter 3 gives an explanation of the methods used to execute the performed tests, gives an impression of the surroundings at the tested locations and gives an introduction to the groundwater flow modeling software of Hydrus2D. The fourth chapter gives the results and first analysis of the performed tests. Chapter 5 gives the model in Hydrus 2D with its outcomes. A discussion on the performed tests and the groundwater flow model is given in Chapter 6. The last chapter draws the conclusions and gives some recommendations.

1. Background and problems

1.1 History of Gouda

Gouda was founded on a marshy peatland around the year 1000 and got its city rights in 1272 AD. It is strategically situated on the Hollandse IJssel and the Gouwe, which is an artificial channel connected to the Oude Rijn. Its foundation is located on an elevated sandy layer in the low-lying peatlands of the Gouwe (Nienhuis, 2010). Not long after its foundation, the owners of the area, as the count of Holland and the bishop of Utrecht, started with the reclamation of the peatlands. The resulting drainage of the peat layer resulted in a fertile subsurface, very well suited for agriculture. However, subtracting water from peat also causes loss of volume and exposure to air induces oxidation. Consequently, the peatland started to subside and flooding became an issue due to the rising groundwater level (Coalitie: Stevige stad op slappe bodem, 2015).

Figure 1 Map of Gouda illustrated by Braun and Hoogenberg around 1585 (Willemse, 2017)

Around the year 1225, de Gouwe became connected with the Oude Rijn through the construction of a canal. Therefore, de Gouwe was an important connection between the Oude Rijn and the Hollandsche IJssel. In addition, the construction of a port gave Gouda an even more important position. It caused an expansion of the city to the inner city as we know it now, by constructing the Turfsingel, Kattensingel, Blekersingel, and Fluwelensingel in 1350. Inside these canals, the city walls and gates were built and these formed the border of the city until the 19th century (van Winsen, 2015).

In 2012, core samples were taken in different parts of Gouda. These profiles give an accurate image of the origin of the city and its natural soil structure. In the images below, two cross-sections are shown. These images clearly show the varying thickness of the peat layer in different parts of the city. Parts of the city with a thicker peat layer are more sensitive to subsidence and were, therefore, more elevated in the past. The first pink elevation dates back to the Late Middle Ages (LME), the lighter pink elevation is constructed in the 16th century (NT) and the top layer is a sand layer, constructed in the 19th and 20th century. Profile A – A' starts south of the inner city of Gouda at the bank of the Hollands Ijssel and goes up north until it reaches the Oude Gouwe. Profile C – C' starts west, just above the Hollandse IJssel and goes east passing the Kuiperstraat and the Peperstraat, which is of particular interest in this project. The exact path of the profile images can be seen in Figure 3 and Figure 4.

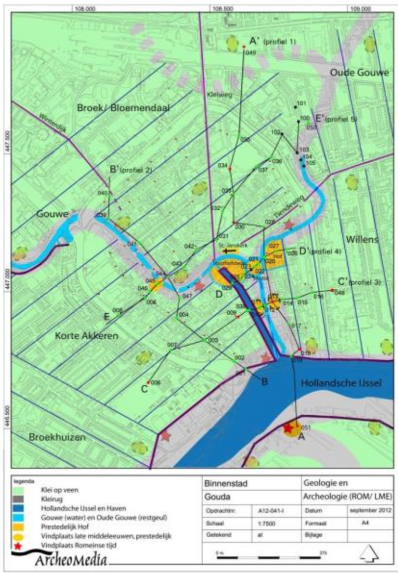

Figure 2 The deposits of the Hollandse IJssel with the core sample locations and paths (van Winsen, 2015)

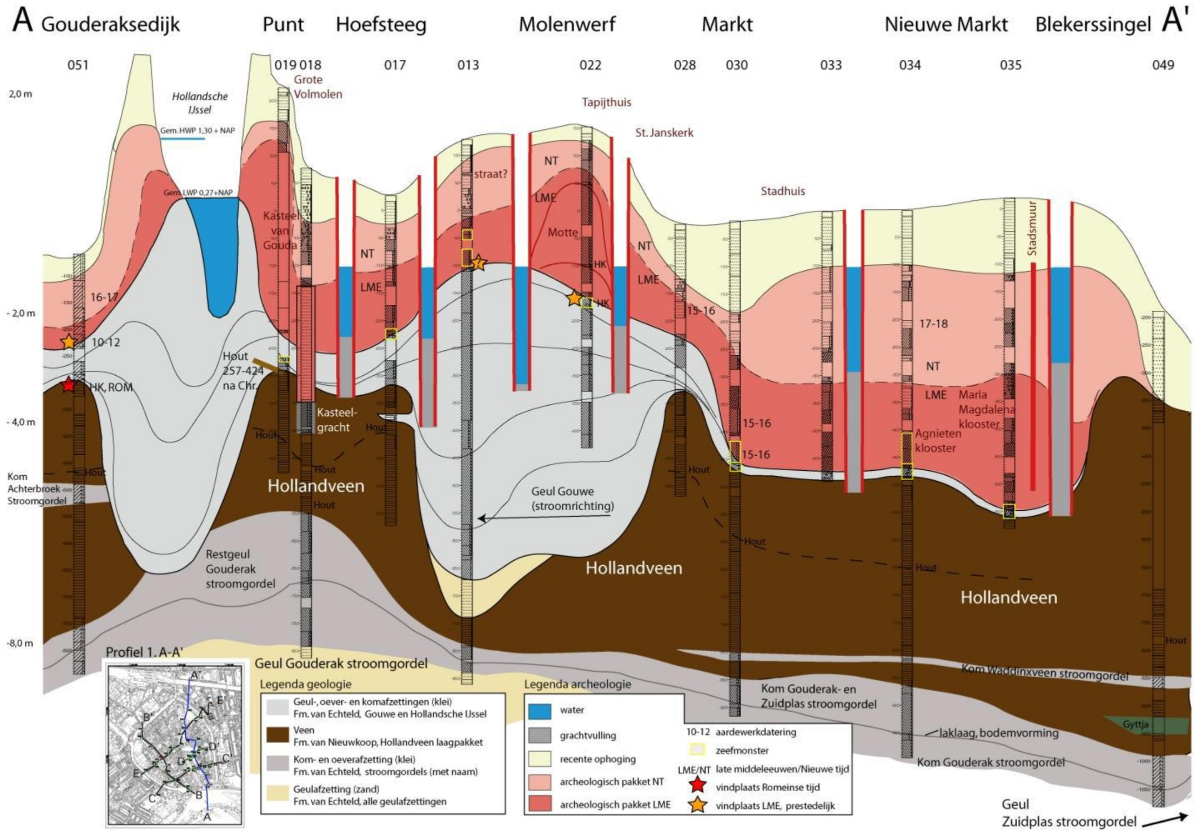

Figure 3 Core sample profile A - A' (van Winsen, 2015)

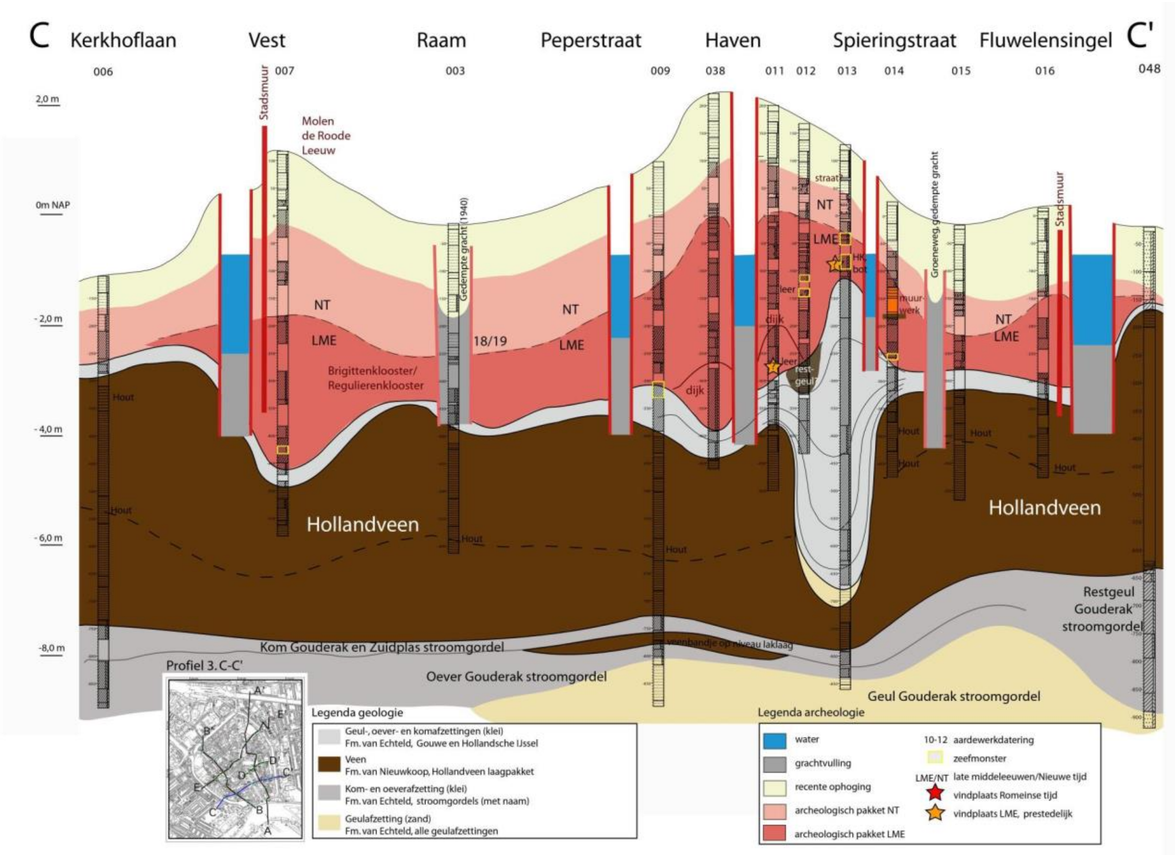

Figure 4 Core sample profile C -C' (van Winsen, 2015)

1.2 Subsidence of Gouda

The soil under the inner city of Gouda is subsiding, because of the 5 to 8 meters of peat layer on which it is founded. Natural processes causing land subsidence are for instance oxidation and sediment compaction. Also, humans can induce subsidence by the withdrawal of hydrocarbons, extraction of groundwater, loading on soft soils and by lowering the groundwater table (Stouthamer E. v., 2015). In the Rhine-Meuse Delta, a significant part of the land subsidence is caused by peat oxidation, peat compaction (due to consolidation and settling), and shrinkage. This paragraph explains the different mechanisms of subsidence relevant to the city of Gouda and its relevant drivers for this subsidence.

1.2.1 Causes of subsidence

Oxidation

Since peat is composed of a mixture of decomposed plant material, it is sensitive to subsidence due to oxidation. When the groundwater level is low (for instance due to high evaporation rates during summer) and the peat is exposed to oxygen, it will decrease in soil volume due to decomposition of the preserved organic matter into CO_2 by biochemical processes (Wösten, 1997). This loss of soil carbon results in a net loss of soil mass and an increase in porosity. This higher porosity is compensated for by compaction under the weight of the peat and the weight of overlying layers (Yuill, 2009). All these processes together produce subsidence of the land.

This mechanism is not relevant for the city of Gouda since the peat layer is completely submerged in the groundwater. Even with the lowest groundwater levels in dry periods, the top of the peat layers in the inner city and Korte Akkeren is at least 0,65 m lower than the groundwater table (van Laarhoven, 2017).

Consolidation and compaction

The terms compaction and consolidation are used interchangeably in the geology field. The slight difference is based on the medium that is forced from the soil pores. Compaction is defined as the loss of pore volume due to the instantaneous expulsion of air from pores, due to overburden pressure (Higgins, 2016). Therefore, compaction does not occur below the phreatic surface, as the soil is completely saturated. Consolidation is referred to as the mechanical compression of permanently saturated layers below the groundwater level (Wösten, 1997). This physical compaction occurs for two reasons: through the expulsion of pore fluid and through the reorientation of sediment grains into a more tightly packed alignment, which is also referred to as primary and secondary consolidation, respectively. The expulsion of the soil pore fluid causes lowering of the hydrostatic component of the internal pore pressure. When this pressure is below the pressure exerted by the total weight of the overlying material, the pore starts collapsing and the volume of the soil is reduced. The second mechanism of consolidation is the gradual steady process of reorganization of the grains, which occurs as grains shifts into an arrangement which is more tightly packed as before, as is often the case in peat layers. Consequently, the rate and the equilibrium conditions of consolidation depends on the weight of the overburden and thickness and compressibility of the layer. (Yuill, 2009).

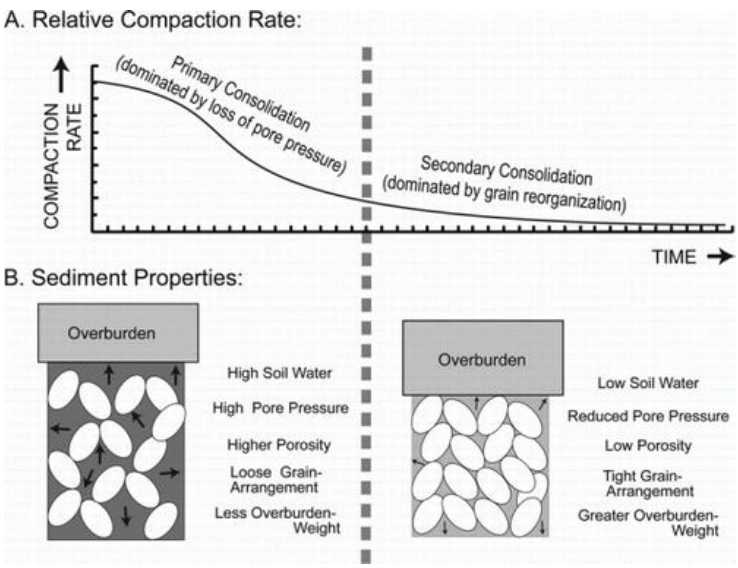

Figure 5 Plot of the A) relative compaction rates and B) sediment properties of delta sediments in time (Yuill, 2009)

Shrinking

Shrinkage of peat layers occurs when drainage is applied or natural causes of desiccation happen. The volume of the soil reduces due to the disappearance of the water and is, therefore, the cause for subsidence above the groundwater level (Wösten, 1997).

These different causes which influence the subsidence are not strictly separated causes. For instance, by lowering the groundwater level not only oxidation will occur, but also shrinkage will take place (Stouthamer & Berendsen, 2008).

1.2.2 Drivers of subsidence in Gouda

Several drivers are influencing the subsidence of peat layers. The withdrawal of hydrocarbons (such as natural gas) and the extraction of groundwater at large depths are common drivers in peat areas around the world. In coastal areas, salinization of groundwater can influence the rate of consolidation of peat layers. The main drivers of the subsidence of the city of Gouda are urban loading and surface water drainage (van Laarhoven, 2017).

Urban loading

When changing farmland into new urban built-up areas, a new load is introduced to the layers below. This load, which can be elevated areas, buildings, and infrastructures, accelerates the compaction and consolidation of the peat layers below (Tosi, 2009). In Venice, this urban loading is believed to be one of the major factors contributing to the present subsidence. Research in the Rhine-Meuse Delta also shows that loading in built-up areas leads to high local subsidence rates due to peat compaction (van Asselen, 2011).

Whether a building structure subsides with the underlying layers depends on the foundation on which it is built. Most buildings in the inner city of Gouda are built on a ‘steel’ foundation. Another type of foundation found in this area is wooden end-bearing piles, which are founded on the sand layer underneath the peat layer (van Winsen, 2015). When the peat layer is compressed, the buildings with a spread footing foundation subside at the same rate as this layer. The buildings on the end-bearing piles remain elevated. However, since these piles are made of wood, they can be negatively affected by the lowering of groundwater. In dry conditions, the piles start to rot leading to loss of bearing capacity of the foundation and structural damage to the building.

Surface water drainage

As told before, from around 1100 AD on the drainage of the peatland started to cultivate the area. The surface water drainage in these cultivated areas leads to accelerated subsidence due to amplified loading and oxidation. Land fillings from the Late Middle Ages on covered the peat layer, resulting in conditions in which oxidation no longer occurs. Only if the groundwater table is lowered until the current depth where the peat layer starts (which never happens), this process could take place. By lowering the groundwater table, the effective stress increases in deeper layers causing consolidation of these layers and hence extra subsidence.

1.2.3 Velocity of subsidence

The velocity and amount of subsidence which is caused by oxidation, consolidation, compaction, and shrinkage varies. Over the past several hundred years, the total subsidence in the peatlands in West-Netherlands is over 2 meters, leading to an average value of 5 mm/year (Schothorst, 1977). Around 1970, the water levels in the ditches in the polders of West-Netherlands were lowered until 60 cm below ground level. This caused an increase of the subsidence in the area until 13 mm/year (Akker, 2007). In general, the weak soil causes a subsidence of 0,5 to 2 cm a year and is predicted to increase with 0,3 to 0,7 cm a year due to climate change (Born, 2016). To illustrate its importance: the current sea level rise amounts 0,3 cm a year, which can increase to 1,0 cm a year according to recent predictions (NASA, 2018). Based on this information, subsidence of land seems more worrying than sea-level rise.

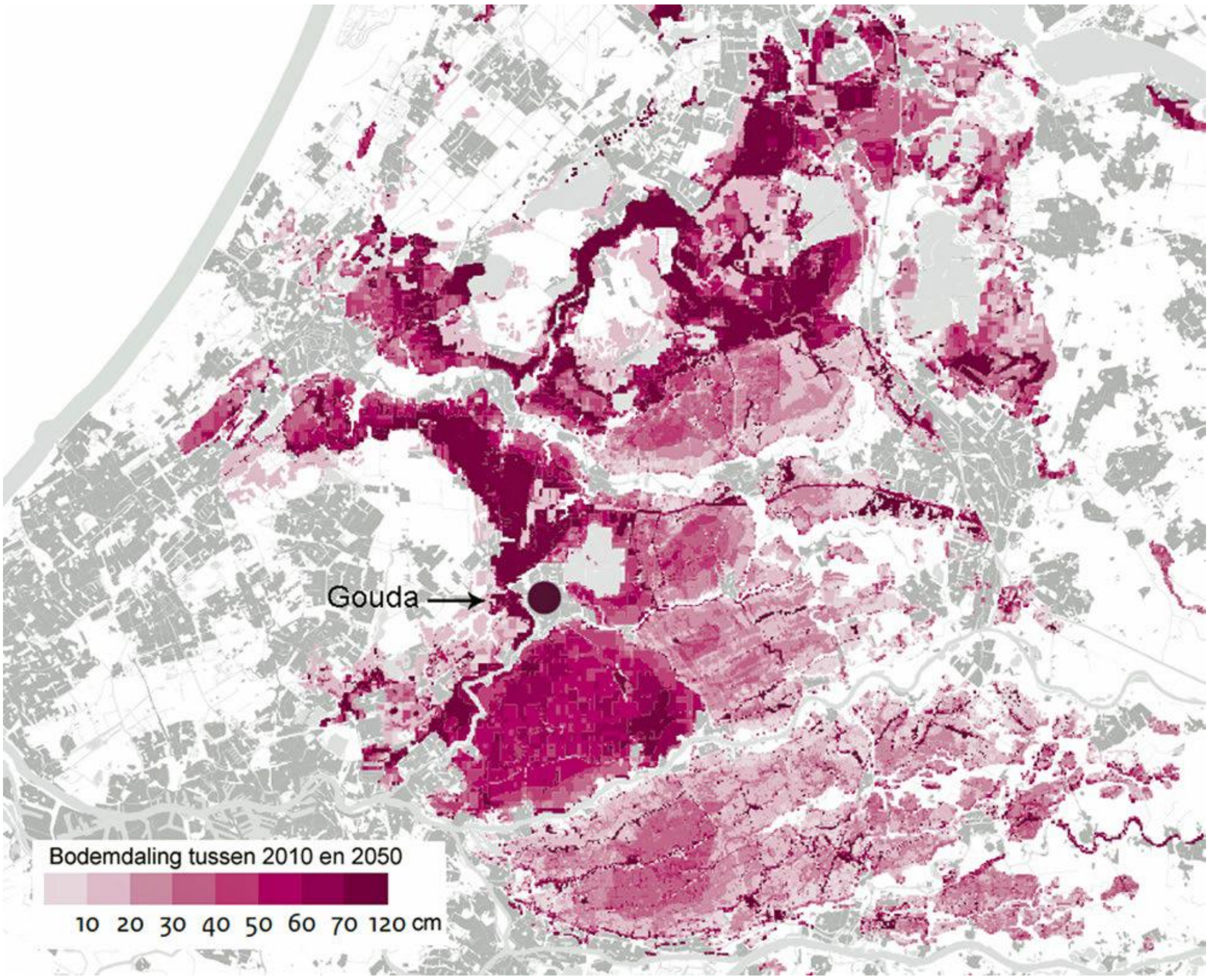

Figure 6 Land subsidence of urban areas until 2050 (Pieterse, 2015)

1.2.4 Effects of land subsidence

The main consequence of the compaction of the soil is the subsidence of houses and infrastructure due to their own weight if they are not firmly founded on a stable (sand)layer. In the inner city of Gouda, houses are built around 1900, when foundations on wooden piles were common. When due to the land subsidence, a lowering of the groundwater level is needed, the wooden piles are exposed to oxygen, leading to rot.

Due to subsidence of the ground level and the peat dikes, the probability of flooding increases significantly. The weight of the layers above the first aquiferous layer has to be sufficient to compensate for the overpressure of the head relative to the groundwater level. This balance changes when land subsides and the groundwater level is lowered. When the head of this layer is higher than the ground level, water comes out as seepage (de Lange, 2006). Also, the soil can burst open due to the increased water pressure. Besides, large amounts of nutrients from the peat oxidation enter the groundwater and the surface water system. This can have a large impact on the water quality, and European standards for these water systems cannot be achieved.

Since Gouda is an old city, many archaeological remains are stored in its soil and preserved by the current conditions. When these remains are exposed to air, the degradation process accelerates (Pieterse, 2015). Also, monumental buildings were built on wooden piles which need to be submerged in the groundwater. Land subsidence has, therefore, also a significant effect on the history of our heritage and the conservation obligation of monuments as recorded in the Erfgoedwet.

1.3 Foundations in Gouda

As mentioned above, a low groundwater level can affect the wooden foundations of buildings. To determine the severity of this process, the most vulnerable areas of Gouda have to be determined. Therefore, a research was started in which the different foundations were determined (van Winsen, 2015). Since these were not documented and it is too expensive to dig up every foundation, a prediction model was developed. This research combined literature about the foundation history in other cities in the Netherlands with the same age, as Dordrecht and Amsterdam, with the documents about the buildings in Gouda and test samples taken in the inner city.

The project revealed the foundation history in Gouda to some extent, but foundations of houses build in certain periods remain unknown. The foundation history can be roughly divided into:

- Before 1828: unknown depth, mostly foundations "on steel"
- 1828 – 1878: unknown depth, presumably mostly foundations "on steel"
- 1879 – 1902: foundation depth on 1,1 m – NAP
- 1903 – 1927: foundation depth on 1,0 m – NAP
- 1928 – 1949: foundation depth on 0,8 m – NAP
- 1949 – present: foundation depth on 1,05 m – NAP

From the applied method can be concluded that most buildings were built in the period before 1828. The type of foundation can only be estimated and only location-specific research can give an unambiguous conclusion. The most reliable estimation of the number of wooden piles foundations in the inner city of Gouda gave a value of 450. Another risk comes from the houses built in the period from 1928 till 1949. These wooden foundations are theoretically the most shallow (0,8 m – NAP) and it is unknown which foundations are built with or without concrete casings.

For the inner city of Gouda, the most vulnerable area is the highest situated part, according to Willemse (2017). Here, the buildings are oldest and the foundations were wooden piled. Several consequences of subsidence of houses on "steel" foundations, on wooden piles or a combination, can be found in Figure 7.

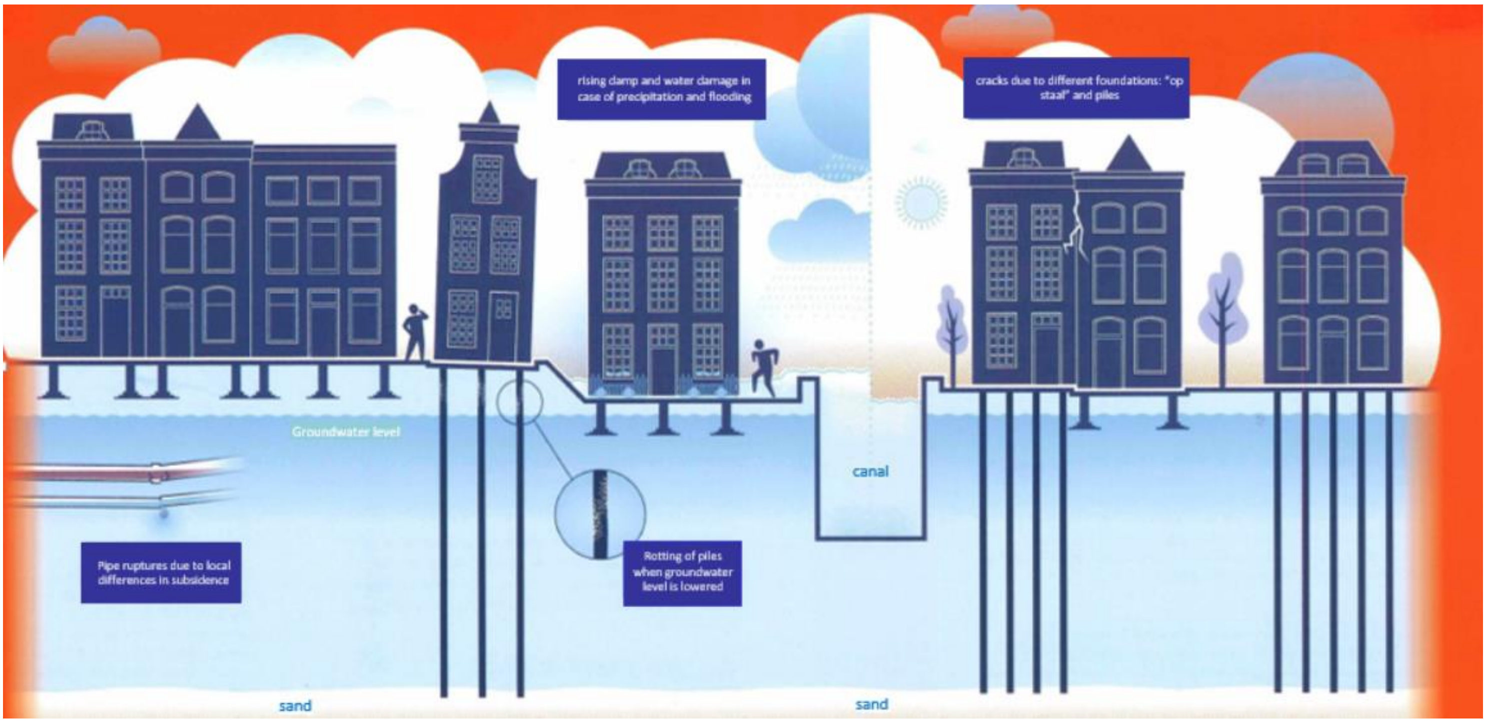

Figure 7 Consequences of land subsidence on housing and infrastructure (Coalitie Stevige Stad, 2018)

1.4 Water management in Gouda

The water system of Gouda is connected to five polders which surround the city. The inner city has its own water system, called 'Stadsboezem". In moderate conditions, a vulnerable balance exists between wet and dry conditions. In case of rainfall, the discharge of water must be sufficient and flooding cannot occur. On the other hand, the groundwater level must be high enough to conserve the wooden pile foundations and prevent extra subsidence. Therefore, fluctuations in the groundwater level are undesirable. In the inner city, surface water, groundwater, and the sewer system are the specific influences on water management. Other influences are precipitation, evaporation and subsurface drainage, as depicted in Figure 8.

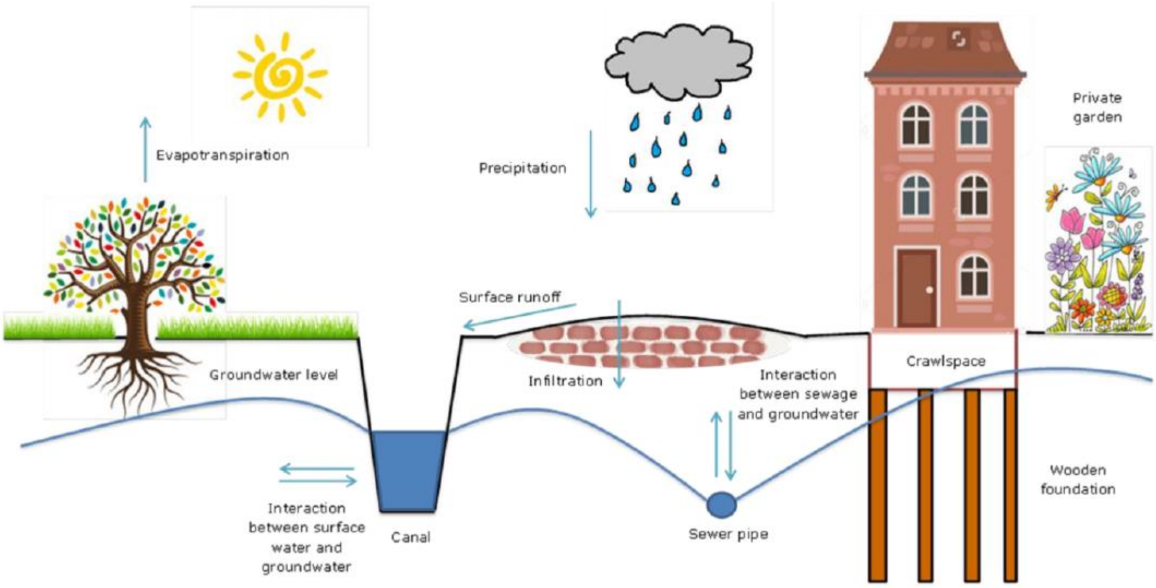

Figure 8 Influential factors on the groundwater level in an urban area (Wang, 2016)

1.4.1 Surface water system

The surface water system in Gouda is dependent on the polders, where it varies between -1,0 m NAP and -2,40 m NAP. In Korte Akkeren, the current surface water level is -2,39 m NAP (Minnen, 2013). In the Stadsboezem, the surface water level is higher, namely -0,72 m NAP. This is a separated water system, with its own water level, controlled by the Kock van Leeuwen lock (Gemeente Gouda, 2011).

In Figure 9, the planned surface water levels are shown for every individual area of Gouda.

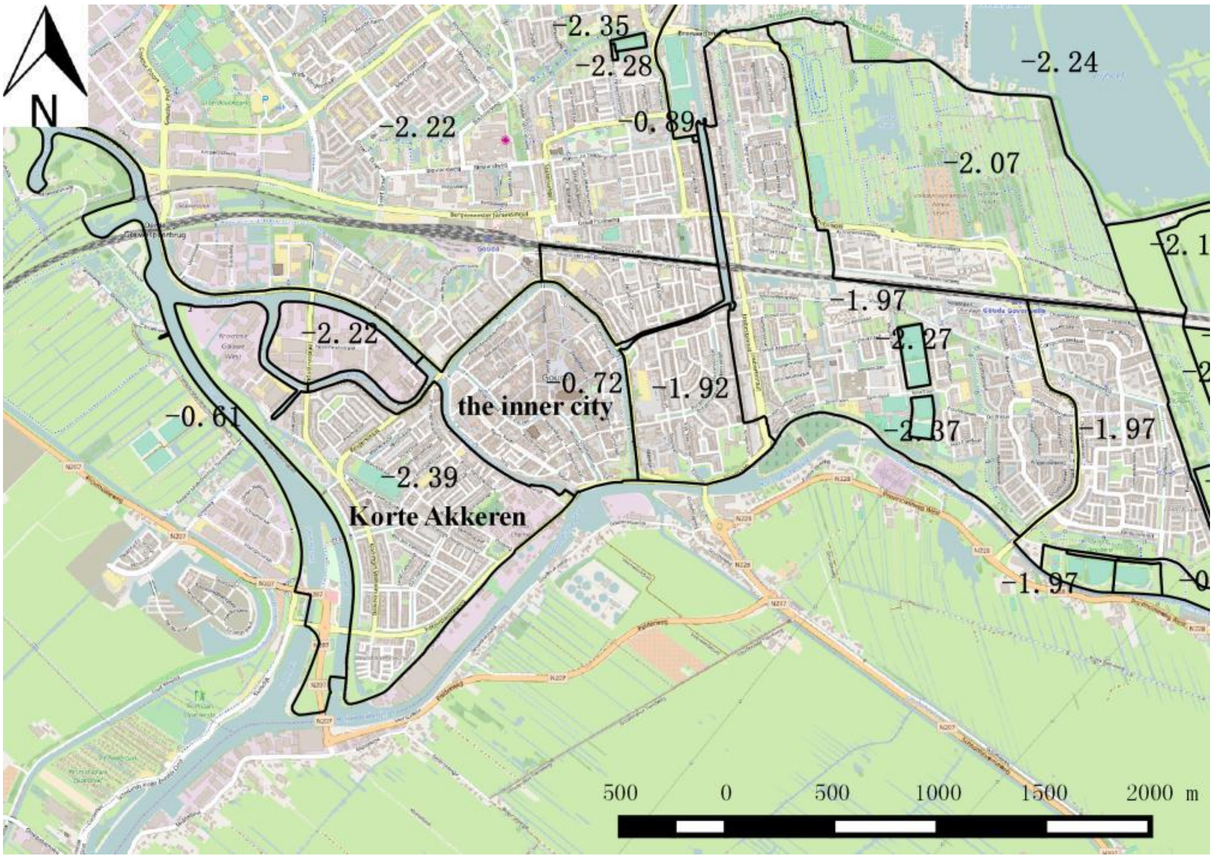

Figure 9 Surface water levels in the polders and Stadsboezem (Chen, 2017)

1.4.2 Sewer system

In the inner city and other prewar neighborhoods, the sewer system is mixed, with both a combined sewer system and a separated sewer system (Tamboer, 2007). Due to, amongst others, aging, subsidence and construction errors, sewer systems are prone to degradation of the structure and cracks in or in between pipes. Depending on the level of the groundwater table and the water content of the sewer trenches, two phenomena can occur, namely infiltration or exfiltration (De Benedittis, 2005). Both have an economical, technical and environmental impact on the performance of the sewer systems. Infiltration gives a dilution of the wastewater leading to a decreasing efficiency of the wastewater treatment plants and hydraulic overloading of the system. The leakage of groundwater into the sewer has also a drainage effect on the groundwater table (van de Ven, 1999). Especially, in the inner city of Gouda, this should be avoided. A lower groundwater table leads to more compaction and can expose wooden pile foundations to oxygen, leading to rot. Exfiltration leads to pollution of soil and groundwater and affects the rise of the groundwater table (Adelana, 2008).

To prevent leakage of groundwater into the sewer system, a back-stowed system exists in the inner city (Tamboer, 2007). For each back-stowed system, two weirs make sure that the sewer pipes are full so less groundwater can enter. During dry weather, the water level in the sewer pipes is similar to the surface water level. In case of an upcoming rainfall event, the weir at the end of the system is opened and the sewer is emptied. As such, the back-stowed system helps to maintain the groundwater level in Gouda. However, most of the back-stowed pipes are part of the combined sewer system. Once the groundwater level is lower than the water level in these pipes, wastewater starts to leak out and pollutes the surrounding soil and groundwater.

1.4.3 Groundwater system

The groundwater system is mainly dependent on the surface water level, the leakage of the sewer system, infiltration to deeper aquifers and infiltration into the soil by precipitation.

To illustrate how groundwater levels vary over time, Figure 10 shows the groundwater levels of four monitoring wells from Wareco spread over the lower inner city of Gouda (see Figure 11 for the locations) for the period from January 2018 until July 2018. The graph shows how the groundwater levels fluctuate, especially far away from the surface water as monitoring well 1-11.1. Note that monitoring wells 1-1.12 and 1-1.14 were placed next to a small canal, which is not shown in the picture. From half June until half July a long drought occurred in Gouda and the rest of the Netherlands. The effects of this drought can be noticed in the graph, especially in monitoring well 1-1.11.

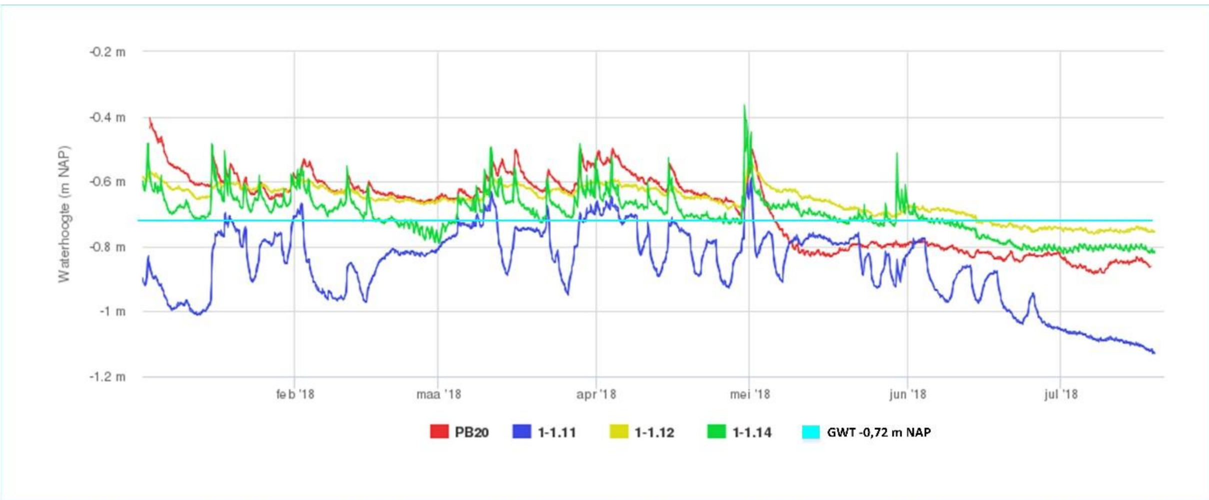

Figure 10 Groundwater levels in the lower inner city of Gouda from January 2018 until July 2018

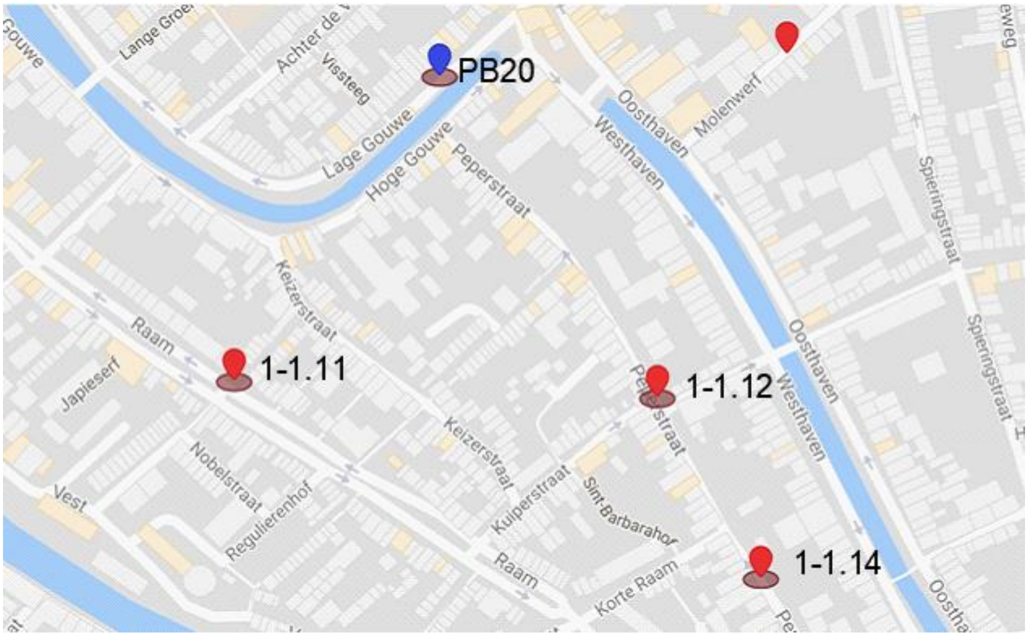

Figure 11 Locations of Wareco monitoring wells

1.5 Problems and possible solutions

There are several problems which have to be conquered in the inner city of Gouda. Outlining of the major ones reads:

- The subsidence of the peat needs to be minimized, meaning no extra loading on the surface.
- Flooding needs to be avoided, so a fast discharge or retention of stormwater is crucial.
- The groundwater level must be above the wooden pile foundations
- From the latter, large fluctuations of the groundwater levels need to be avoided.

1.5.1 Low impact development

One of the stormwater management strategies is Low Impact Development (LID). This strategy can roughly be divided into infiltration-based LID and retention-based LID. Infiltration-based LID can be characterized as techniques that assist in the restoration of baseflows through recharging of subsurface flows and groundwater. Examples are rain-gardens, swales, infiltration trenches, basins, and porous pavements. For the inner city of Gouda, most of the stormwater needs to be discharged, since a standard groundwater table needs to be maintained to avoid inundation. Therefore, options for infiltration-based LID are very limited unless combined with additional subsurface drainage capacity. Retention-based LID can be characterized as techniques that retain stormwater to reduce outflow. Examples are wetlands, ponds, green roofs and harvesting rainwater with tanks and storage basins (Eckart, 2017). Options for this type of LID are also limited in the inner city since space under and above ground is scarce. One specific option of retention could be to restore the old canals.

Restore old canals

In the specific case of Gouda, many of the canals were filled again in the 20th century. The images below show the canal system in 1830 on the left and the current canal system on the right. There were two main reasons for filling these canals, namely public health and traffic. The canals were open sewers in the past and, consequently, a breeding ground for diseases like dysentery, cholera, and typhus. Later on, when the car made its entrance, the small streets needed to expand and parking lots were needed. Nowadays, only the canals of the Turfmarkt, Zeugstraat, Gouwe, and Haven are left (Sporen, 2018). Many inhabitants of

Figure 12 Canal system of the inner city of Gouda in 1830 (left) and 2018 (right) (Sporen, 2018)

Gouda want the canals back to restore the old cityscape and keep out traffic. For the water management of the inner city, this gives an opportunity for extra water storage. However, a current estimation of extra water storage for the current plans is 1 percent (Gouda Hollandse Waterstad, 2009), which makes it a not significant solution in terms of water management. Another advantage of restoring these canals is a better influence on groundwater levels since the distance between canals is shorter.

1.5.2 Groundwater level control strategies

There are several groundwater level control strategies which can be applied in general. The strategies relevant to the city of Gouda are discussed below.

Filling/raising land

When the groundwater level is close to the surface, a simple strategy is to raise the land. This was done repeatedly in the city of Gouda from the 14th to the 16th century and even in the 20th century (Coalitie: Stevige stad op slappe bodem, 2015; van Winsen, 2015). Nowadays in a densely populated area like the inner city of Gouda raising land is not an option and with the current knowledge, it never was. Firstly, because the streets would be above the thresholds of the houses. Secondly, raising of land causes extra load on the peat layer below which will subside faster.

Subsurface drainage

Installation of subsurface drains reduces the dynamics of the groundwater table fluctuations. In wet period water drains quickly and the high groundwater levels fall. During dry periods the drains supply water and the groundwater level does not fall too much. Model results on these drain system in Dutch peat meadows show little fluctuation of the groundwater level in winter and summer (Querner E.P., 2012). These so-called Drainage Infiltration-Transport (DIT) – sewers will be discussed in Chapter 2.

2. DIT sewers

2.1 Function and construction

A Drainage Infiltration-Transport (DIT) – sewer is a perforated (mostly propylene) pipe wrapped in geotextile and embedded in a gravel casing, which is horizontally placed. It combines three functions depending on the groundwater level. In the case of a lower groundwater level than the depth of the DIT sewer, it starts to infiltrate water into the soil, if there is water present in the pipe. This can either be pumped in or it can be stormwater, which enters the DIT sewer through the connected gullies or water from the surface water body the DIT sewer is connected to. If the amount of incoming water into the pipe exceeds the infiltration capacity of the DIT sewer, it starts to act as a regular stormwater sewer and starts to discharge on the connected surface water. If the groundwater level is higher than the depth on which the DIT sewer is constructed, it starts to act as a drain and the groundwater (and stormwater in case of

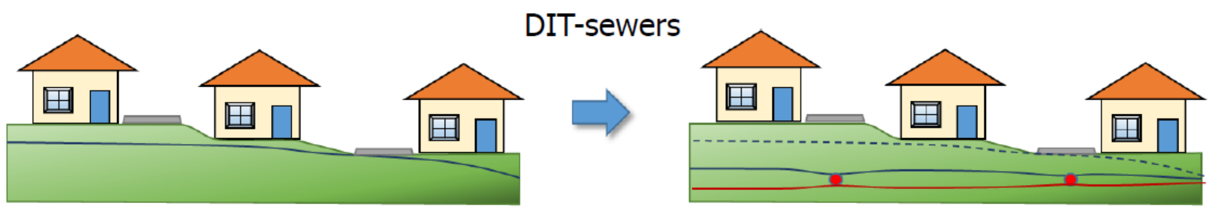

Figure 13 Groundwater level without (left image) and with (right) DIT sewers. Blue line is the intended GW level in case of high GW level and the red line is the intended GW level in case of a low GW level.

rainfall) will be discharged to the surface water (Kennisprogramma Bodemdaling, 2018). Infiltrated rainwater through the soil can also enter the DIT sewer and influence its operation depending on the permeability of the pavement.

Terms for a sewer with these functions are used interchangeably. Another type is the Infiltration-Transport (IT)-sewer. These drains are usually constructed above the groundwater level and connected to the stormwater system with gully pots. They infiltrate the stormwater in case of rainfall and transport the water to the surface water when the infiltration capacity is exceeded. Another type is the drainage-infiltration system. Major difference compared to a DIT or IT sewer is that these are not connected to gullies, hence the absence of the stormwater transport term.

A DIT sewer is placed in a gravel casing, surrounding with geotextile to prevent influx of sand and soil particles which block the inflow of water between the gravel particles. These gravel casings also act as a small water buffer.

2.2 Subsurface flow to drains

As mentioned, research on DIT sewers is scarce. Recently, TU Delft and Waternet did an investigation on the DIT sewer in the Argonautenstraat, Amsterdam (Abbas, 2017). In this area, a DIT sewer is constructed together with a high permeable gutter, called a Granudrain. An infiltration test and a drainage test were performed. These tests gave a value of 0,10 – 0,20 m^3/m/u per meter potential difference for infiltration capacity and a value of 0,18 m^3/m/u per meter potential difference for drainage capacity. From a groundwater analysis was concluded that the DIT sewer did not fulfill its infiltration purpose since the groundwater level lowered to 0,20 m beneath the desired level in the summer, which can cause a threat to the wooden pile foundations. Hypothesized cause was the transpiration of the trees along the road. In the winter when the groundwater level was higher than the surface water level, the DIT sewer was supposed to drain. Observations in this period showed that the groundwater level fluctuated in the same manner as the surface level.

2.2.1 Ernst equation

Principles of groundwater flow can describe the flow of groundwater towards field drains, mostly restricted to parallel drains. Several steady-state drainage equations exist. These equations are based on the assumption that the drain discharge equals the recharge to the groundwater and consequently the water table remains in the same position (Ritzeman, 1994). Further assumptions are:

- Two-dimensional flow, meaning the flow is identical in any cross-section perpendicular to the drains
- Uniform distribution of the recharge
- Homogenous and isotropic soils, so ignoring any spatial variation in the hydraulic conductivity within a soil layer

One of these equations is the Hooghoudt (or Donnan) equation, which describes the flow of groundwater according to Darcy's equation. This equation is applicable in homogenous soil profiles.

Another formula which can describe the flow of groundwater next to parallel drains is the equation of Ernst. It has the advantage over the Hooghoudt equation that the interface between the layers can be either above or below the drain level. The general principle underlying this equation is that this flow, and consequently, its available total hydraulic head can be divided into three components: a vertical (v), a horizontal (h) and a radial (r) or

$$h = h_v + h_h + h_r = qR_v + qR_h + qR_r$$

where q is the flow rate and R is the resistance.

Working out various resistance terms, we can write the Ernst equation as

$$h = q\frac{D_v}{K_v} + q\frac{L^2}{8KD} + q\frac{L}{\pi K_2} ln\frac{aD^2}{u}$$

where

h = the total hydraulic head (m)

q = flow rate (m/d)

L = drain spacing (m)

D_v= thickness of the layer over which vertical flow is considered; in most cases, this component is small and may be ignored (m)

K_v = hydraulic conductivity for vertical flow (m/day)

KD = the sum of the product of the permeability (K) and thickness (D) of the various layers for the horizontal flow component according to the hydraulic situation:

one pervious layer below drain depth: $KD = K_1D_1 + K_2D_2$ (Fig. 2a)

two pervious layers below drain depth: $KD = K_1D_1 + K_2D_2 + K_3D_3$ (Fig.2b)

a = geometry factor for radial flow depending on the hydraulic situation:

$KD = K_1D_1 + K_2D_2$, a = 1

$KD = K_1D_1 + K_2D_2 + K_3D_3$, the a-value depends on the K_2/K_3 and D_2/D_3 ratios

u = wetted section of the drain (m); for pipe drains $u = \pi r$

2.2.2 Entrance resistance

Water flowing into a drain radially converges with a secondary convergence at the drain openings. In Figure 14, the flow pattern for evenly distributed openings, as in plastic drains with geotextile and gravel envelopes, is shown. This drain is considered to be a hydraulically-ideal pipe which allows water to enter uniformly over its surface. A hydraulically-ideal drain is essentially a completely permeable drain without any appreciable entrance head loss or secondary convergence. In gravel envelopes, any secondary convergence takes place in the high permeability gravel and the pipe-envelope (Vlotman, 2000).

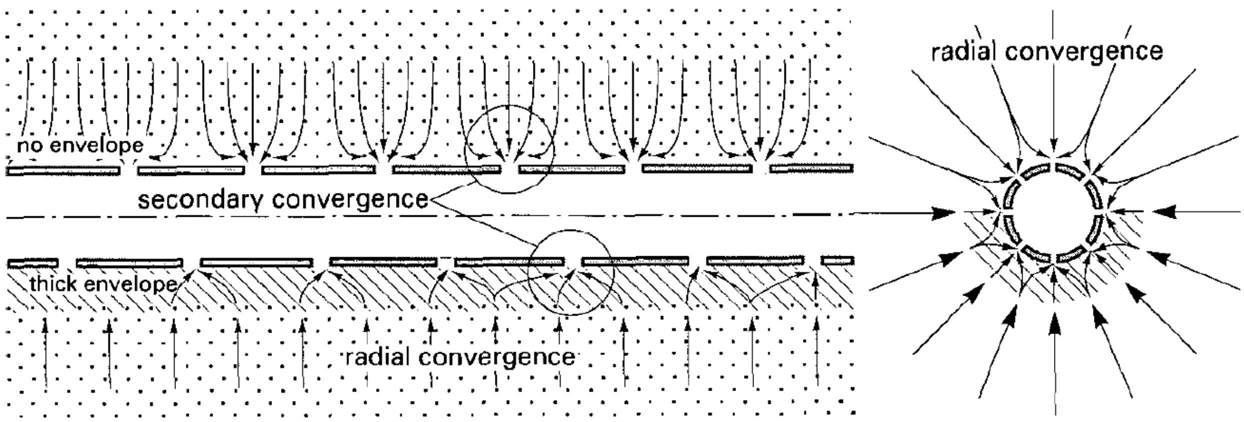

Figure 14 Flow pattern towards perforated plastic pipe drains (Vlotman, 2000)

In reality, entrance head loss is caused by factors as variability in the soil condition, application of a (gravel) envelope and converging streamlines towards the drain perforations. This is the sum of convergence head loss, and the combined radial head loss in the soil, trench, and envelope:

$$h_e = h_c + h_r$$

Measuring convergence or radial head loss separately is impossible in the field. Therefore, entrance resistance reported from field experiments is the total entrance head loss, expressed as:

$$w_e = \frac{h_e}{q} \qquad \text{or} \qquad h_e = q w_e$$

where,

h_e = the head loss determined as the difference between the water level in the observation well closest to the drain and in the drain pipe

q = the actual drainage coefficient in days, which can either be the design drainage coefficient or the actual discharge (Q) divided by the drain length (L) and spacing (S)

w_e = the total resistance w_r (radial entrance head loss) and w_c (contraction head loss) in days

This extra resistance term can be added to the Ernst equation, which becomes:

$$h = q\frac{D_v}{K_v} + q\frac{L^2}{8KD} + q\frac{L}{\pi K_2} ln\frac{aD^2}{u} + qLw_e$$

2.3 Clogging risks

Considering the construction of a DIT sewer, one can identify several risks on clogging:

1. The geotextile around the gravel casing
2. The gravel casing
3. The holes in the perforated pipe
4. The DIT sewer pipe

2.3.1 Geotextile

Geotextiles can provide enough in-plane liquid flow capacity and can, therefore, be used for drainage purposes. Nonwoven (heat-bonded or needle-punched) geotextiles have more void space in their structure, which makes them less vulnerable to clogging and are, therefore, more suitable as drainage material (Chai, 2016). These geotextiles are compressible, which can reduce the thickness and coefficient of permeability in case of confining stress. In general, the in-plane flow capacity of geotextile is equivalent to fine gravel at low normal stresses and can decrease to medium sand at high stresses (Koerner, 1984). The filter capacity of a geotextile is specified in the O90 value, meaning that 90% of all particles larger than the named O90 value will be filtered out.

It is stated that in soils with a high humus content (such as peat) washed-out degradation products can clog the geotextile. Also, soils with a high pH-value (meaning calcium-rich soils) have a potential for geotextile clogging. The calcium comes into contact with the stormwater and forms a low-permeable layer on the geotextile (Boogaard & Wentink, 2007). Clogging due to iron oxides are also common, which is discussed paragraph 2.3.3.

2.3.2 Gravel casing

Previous research has shown that voids within the granular leachate collection layer become filled with clog material as a result of the growth of biomass, bio-induced chemical precipitation of inorganic matter (as calcium carbonate) and accumulation of particulate matter (McIsaac, 2007). Further research shows that under unsaturated conditions there is much less clogging than when the gravel (in this research crushed limestone) layer is saturated. Also, coarse gravel is less vulnerable to biofilm growth than less coarse gravel (McIsaac, 2007).

In Gouda, the casing around the DIT sewer pipe consists of Argex expanded clay aggregates. These particles are light weighted, have good draining properties and are used in the Netherlands as a casing around sewers and other pipes (Argex, 2009). These expanded clay aggregates further have characteristics like non-biodegradable, moisture impermeable, do not damage or bind together in water (Rashad, 2018). They are implemented as broken aggregates (4-8 mm) to increase the tear resistance and avoid rutting of the parent pavement while still having good drainage properties. These properties, especially the low weight, makes them suitable for Gouda since less bearing load induces less subsidence.

Figure 15 Concrete infiltration pipe affected by oxidation of iron in groundwater (Boogaard & Wentink, 2007)

2.3.3 Blockage of perforations

The holes in the pipe wall can clog for several reasons. Already mentioned are bioactivity and inflow of particulate matter which can cause blockage of the perforations. An additional cause for blockage can be the formation of iron oxides. When bivalent iron in the groundwater comes into contact with oxygen it forms iron oxide flocs (Boogaard & Wentink, 2007). These flocs clog an infiltration facility. Especially DIT sewers can be vulnerable to this chemical reaction. When the groundwater is iron-rich and fluctuates, the conditions around the pipe are alternately oxygenated and low in oxygen, so iron oxide flocs are likely to be formed and can block the perforations of the sewer pipe as well as the geotextile. Figure 15 shows a concrete infiltration pipe which is affected by oxidation of iron.

2.3.4 Blockage of DIT sewer

Water entering a DIT sewer is mostly infiltrated stormwater. Looking at the major causes of blockages in a stormwater system, we can distinguish between:

1. Natural debris: Leaves and sediment which enter the pipe through connections and gullies.
2. Broken pipe: (Uneven) settling can fracture the pipe as well as roots growing into the pipe
3. Improper installation

Figure 16 IT-sewer with sediment buildup after one and a half year (Boogaard & Wentink, 2007)

In a DIT sewer, the inflow of sediment is prevented by the geotextile, casing, and the sand trap in the gullies, but still, a part of the sediment can enter the pipe via the perforations and connected gullies. In the thesis research of Chuan on an infiltration sewer in Eindhoven, the clogging in an IT sewer was investigated. He found an accumulation of 4 cm of sediment on the bottom of a 300 mm pipe in a period of 8 years (Chuan, 2011). When this is spread over the entire bottom, it could hamper the exfiltration property of the sewer. Since the velocity in a drain pipe is usually small, the sediment can settle and is likely to clog the system.

2.4 Cleaning options

Not much literature can be found on the cleaning of drain pipes. For different clogging mechanisms, different cleaning procedures are valid. The most common clogging cause is due to sediment, in drain pipes as well as in combined sewers. Cleaning options for these sewers are, therefore, also valid for drain pipes or stormwater sewers.

2.4.1 Sediment removal

Methods of sediment removal are most commonly employed as moving the sediment to a location for removal by mechanical or suction equipment. A number of these methods are discussed and can be either used in case of blockages or as preventive maintenance (Pisano, 2003).

A hydraulic cleaning method, called balling, is used to remove settled grit and grease inside a pipe. The pressure of a water head creates a high velocity water flow around an inflated rubber cleaning ball, which has an outside spiral thread and swivel connection that causes it to spin. Methods with a poly pig, kites and bags work similarly as balling. Water pressure moves these devices and by scouring the settled sediment is removed.

Figure 17 Cleaning of a drain pipe with high pressure water jetting (BudgetDrainCleaning, 2018)

A hydraulic cleaning method, most suitable for (corrugated) drain pipes is jetting. At various angles, a jet of water is directed against the pipe walls at high velocity with a spray nozzle. This is more effective compared to the other mentioned methods since the force is much larger due to the high intensity which the water is shot. Besides, this method is more capable of bending around curved or corrugated pipes. In case of drain pipes, the water with the sediment is removed with mechanical or suction equipment so it doesn't end up in the surface water.

To displace deposited solids, flushing can be used. With flushing, an unsteady waveform is induced by rapidly adding external water. The deposited solids come loose and transported with the 'flush'.

2.4.2 Ochre deposit removal

The formation of Fe(II) into Fe(III), which is precipitated as iron ochre, is a process common in drain pipes. This ochre can be precipitated both chemically and biologically (Vaughan, 1994; Ivarson, 1978). Literature on this subject is mostly from the 70s and 80s. The most logical removal of the ochre deposits is to create a microaerophilic or anaerobic condition by raising the groundwater level. Former research showed a complete removal of iron oxides in a clogging drainpipe 3 to 4 weeks after the drain pipe was completely submerged for this period (Abeliovich, 1985). However, research in the Netherlands showed that oxidation still occurred in these conditions (Scholten, 1989).

Research shows that precipitation of iron oxides can be delayed using Cu(II) ions. Incorporation and slow release of this chemical can be a method to control ochre problems (Vaughan, 1994). However, toxic properties of Cu(II) ions would render this approach environmentally unacceptable. Another suggested method to counteract the clogging of drains is the use of tannin, either as a layer on the surface of the drainpipe or as an added solution into the drain pipe (United States Patentnr. 3.917.530, 1974).

It is difficult to precisely estimate the risk of iron ochre clogging and the severity and duration of it. It may be removed by frequent flushing of the drains. In severe cases in the Netherlands, flushing once a year was necessary, while once in six years was adequate in other cases (Vlotman, 2000).

3 Methods

To get an insight into the performance of the DIT-sewer in the Kuiperstraat and the drainage-infiltration sewer in the H.J.A.M. Schaepmanstraat, three tests are executed:

- An inundation test: to find the permeability of the pavement above the DIT sewer and determine an infiltration capacity value. An estimation of the infiltrated water volume can be found which influences the operation of the DIT sewer.
- An infiltration test: to test the infiltration of water from the sewer into the surrounding soil
- A drainage test: to test the drainage of groundwater from the soil into the sewer in case of high groundwater levels

To monitor the reaction of the groundwater levels on the performed test, monitoring wells were placed at the tested areas.

In this chapter, the three tests are firstly explained in general form and subsequently for the specific locations in Gouda.

3.1 Tests

3.1.1 Inundation test

To determine the permeability of the pavement several tests can be performed, giving each a value for the infiltration into the soil. Most known are the single or double ring infiltrometers, either circular or square, as shown in Figure 18.

Figure 18 Modified Ring Infiltrometers used for Permeable Pavement Testing (a) Double Ring Infiltro Test (DRIT); (b) Square, Double Ring; (c) Double Ring Infiltro Test (Boogaard, 2015)

More accurate infiltration results can be acquired if the tested area is increased. By inundating an area of at least 50 m^2 spatial variations in infiltration capacity would be averaged-out, giving more reliable infiltration data (Boogaard, 2015). To restrict the area and contain the water within this area it is necessary to construct temporary dams at the end of the pavement test sections. Ideally, the area should have at least one speed-hump at one of the ends of the area to save setting-up time and minimize leakages during testing. Also, the amount of the drainage gullies should be minimized inside the area, since these need to be sealed. Soil- or sand-filled plastic bags are advised to create the dams, due to their ability to properly seal the test sections, the rapid filling and emptying of the bags, the opportunity to reuse the material and no necessity of heavy machinery.

For water supply, several options are available. The recommended method is to pump water from a nearby canal. This offers flexibility in different types of testing and offers an unlimited amount of water. Downside of this option is the quality of the water in the canal, which can be worse than ordinary stormwater. This can lead to clogging and consequently an unreliable outcome of the permeability value. Second recommended option is the water truck. Disadvantage of this method is the limited availability of water and the space it takes to park and maneuver the vehicle.

To measure the water height on the street it is recommended to locate pressure transducers at several places in the test area. The transducers are continuously monitoring the water pressure, whereafter the stored data can be retrieved and converted to an appropriate water depth on the pavement. Since these devices can be sensitive to external influences such as wind and changes in atmospheric pressure, the data retrieved needs to be calibrated. This can be done by taking water level measurements manually with a simple hand ruler at strategic locations on the pavement surface throughout the duration of the test.

3.1.2 Infiltration test

To determine the infiltration capacity of the DIT sewer, a test was performed whereby a high surface water level is simulated. A part of the DIT sewer was isolated by placing rubber balloons in the pipe ends at two manholes. By pumping water from the truck into one manhole, the water height was set on maximum level (street level). In this way, a pressure difference was created between the DIT sewer and the surrounding groundwater and water would infiltrate into the soil via the perforations in the pipe until the groundwater level was equal to the water level in the manhole. As soon as the water level in the manhole was stable, the discharge of water was equal to the infiltration discharge, which was measured by the measuring device. This discharge could be measured with a constant head, so be adding an equal amount of water as infiltrated. It could also be measured with a falling head. The manhole was filled until street level and the time was measured until the water level was equal to the initial water level. This method is preferred when the infiltration capacity is low. In this case, a constant head would require such a low added water amount per unit of time, that a test isn't feasible timewise.

3.1.3 Drainage test

To determine the drainage capacity of the DIT sewer, a test was performed whereby the DIT sewer was completely emptied. A part of the DIT sewer is isolated by placing rubber balloons in the pipe ends at two manholes. The first manhole was completely emptied with a pump on the tank truck and the pumped water volume was determined. Groundwater would start to drain into the sewer. Depending on the infiltration velocity, a constant head measurement was performed by keeping the water level at a constant height and measure the water inflow. If the infiltration capacity was lower than the minimal pump capacity, only a falling head measurement was performed. Otherwise, both a constant head and a falling head measurement were executed. The pump was switched off when the water level was at its highest possible level and the time was measured until the water level was at its original level.

3.2 Kuiperstraat

The Kuiperstraat is a street located in the southwest part of the inner city of Gouda and is a connection between the Peperstraat and the Raam, see Figure 19. Most houses in this street are built before 1900, some even before 1828, which can be noticed by the degree of subsidence. It is, therefore, a perfect location for a research in the context of the project “Slappe Bodem”. The pavement has been in service for over 2 years. A cross-section of the street at the tested location is given in Figure 21. The DIT sewer (Wavin DT Buis + PP450 GN DN250 SN8) beneath the pavement was installed in 2016 at a depth of -1,05 m NAP (invert pipe level) and has a diameter of 0,25 meter. It is installed in a 0,4 x 0,4 m casing of expanded clay aggregate (Argex AG4/8-320), which is wrapped in a woven geotextile (Geolon 80, O90 value:275 µm). The DIT sewer is connected to the gully pots and the rainwater drainage of the surrounding roofs. Test bores in the soil until 2 meters beneath ground level by Lievense in 2015 showed a soil composition of sand with an amount of debris varying from slightly to heavily.

Figure 19 Geographic location of the Kuiperstraat

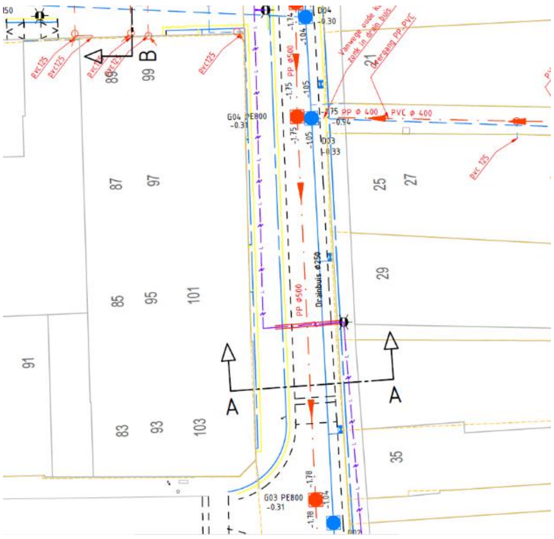

Figure 20 Overhaul plan of the Kuiperstraat at the tested area

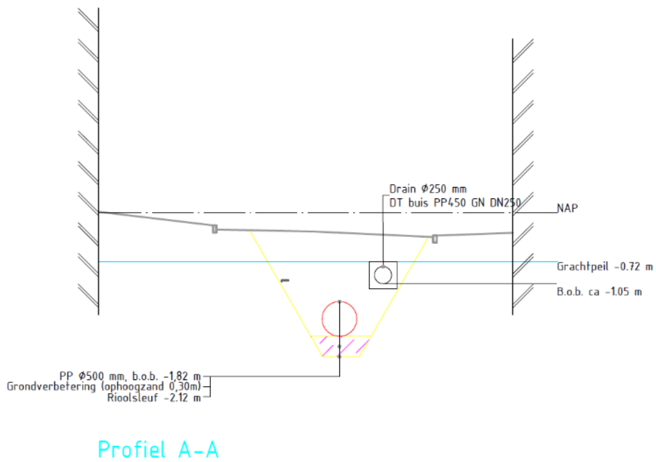

Figure 21 Cross-section A-A from overhaul plan

Figure 22 Impression of the Kuiperstraat seen from the Raam

Figure 23 The DIT sewer pipes used in the Kuiperstraat

To monitor the effects of the tests on the groundwater level around the DIT sewer monitoring wells were installed. Two rows with two monitoring wells (one next to DIT sewer and one on approximately 1 meter) were placed as depicted in Figure 24 and Figure 25. Pressure transducers were placed to monitor the groundwater level fluctuations during the period around the tests.

Figure 24 Top view of the Kuiperstraat with the locations of the monitoring wells

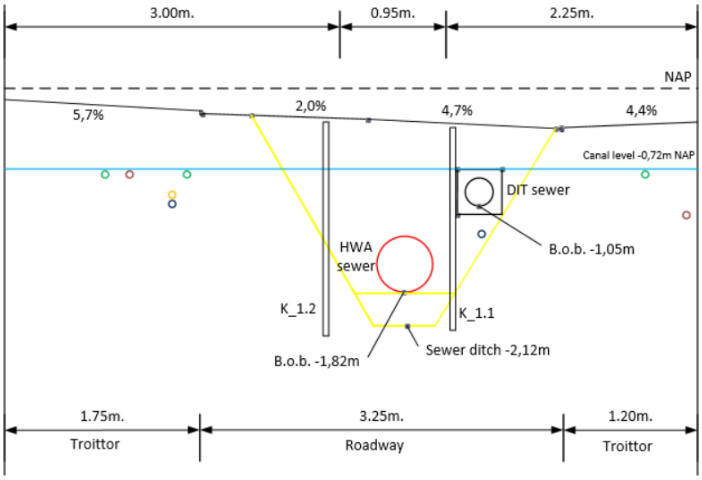

Figure 25 Cross-section Kuiperstraat with the first row of monitoring wells

3.2.1 Inundation test

To accurately determine the permeability of the pavement in the Kuiperstraat a part of this street has to be selected. The most suitable location, keeping in mind above constraints, is between 30 and 60 meters, which is between the crossing with the Keizerstraat and the alley to the Barbarahof. To minimize the number of gully pots and manholes, the part of the street from the speed hump until the second gully pot is taken as depicted in Figure 27. This area (yellow rectangle) has a surface of 52 m^2.

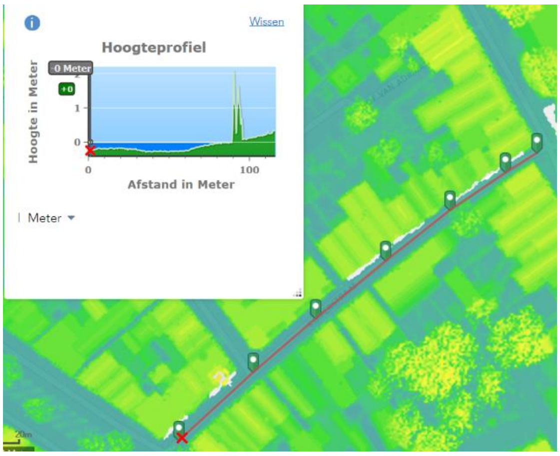

Figure 26 Relief map of the Kuiperstraat

Figure 27 Kuiperstraat with flooded area in yellow

For inundation of the area, water was supplied with a water truck. The speedhumps in this street are not significantly higher than the road itself, but more an optical slowing down device and can, therefore, not be used as a water barrier. This means that the total area has to be restricted by a dam. Chosen is to construct this barrier of sandy clay. The gully pot is covered with sandbags to prevent leakage. For an impression of the inundation test, see Figure 28.

The enclosed surface was inundated with the water from the truck until the highest point in the street has at least 1 cm of water column. This gives a water height of 11 cm on the lowest point in the street depicted in Figure 29. These heights were measured by two pressure transducers, both on the right side of the street on the lowest points. Manual water depths were also recorded at the transducer locations as well as on several other locations of the street over the duration of the test to enable calibration and verification of the transducer readings. The depicted height was held constant by supplying water at an even rate as the infiltration rate, according to the constant head full-scale method (CHFS). This turned out to be impossible since the infiltration rate was lower than the minimal pump capacity of the water truck. Therefore, only the falling head full-scale method (FHFS) was applied.

At a given water level, the supply of water was stopped and the time it took to drain the water on the lowest point in the street was measured according to this method. The time range of the FHFS method was 70 minutes.

Figure 28 Impression of the Kuiperstraat during the inundation test

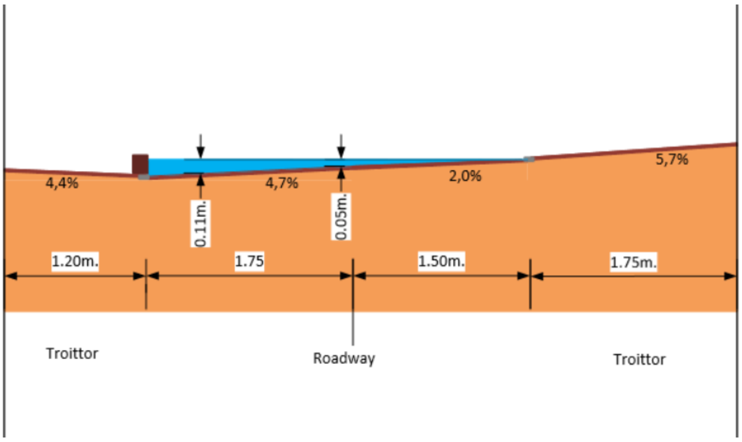

Figure 29 Cross-section of the Kuiperstraat with inundation

3.2.2 Infiltration test

To determine the infiltration capacity of the DIT sewer, a pipe segment between manhole D02 and D03 was isolated by placing rubber balloons as depicted in Figure 30. The total length of this segment is 24 meters. According to the drawings, the first 6 meter attached to each manhole is non-perforated, leaving an effective DIT sewer of about 12 meters.

The initial water level in the manhole was measured. A pressure transducer was placed in manhole 124133, whereafter the pipe was filled via this manhole until street level. This level was held constant, according to the constant head method. The infiltration capacity was lower than the supply capacity of the water truck, so only the falling head method was applied. The supply was stopped and the time measured until the water level was equal to the initial water level.

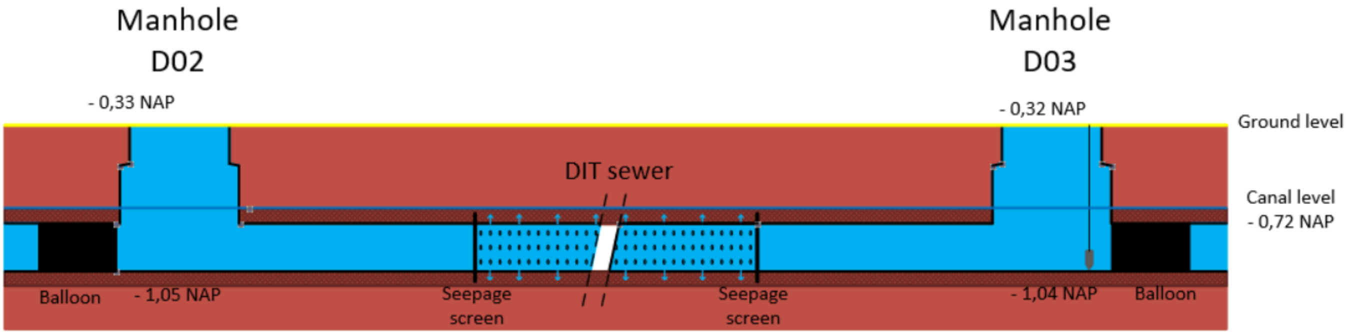

Figure 30 Longitudinal section of the tested DIT sewer segment during infiltration test

Then the DIT sewer was emptied and the pipe segment was cleaned by using a pressurized spray nozzle (see Figure 31). After this cleaning step, the test was performed again as depicted above, to check if there was any difference with the uncleaned test.

Figure 31 Spray nozzle

3.2.3 Drainage test

To determine the drainage capacity of the DIT sewer, the same pipe segment between manhole D02 and D03 was isolated by placing rubber balloons as depicted in Figure 32. Via manhole D03, the DIT sewer was completely emptied. The pressure transducer was placed in manhole D02. The drainage capacity was lower than the pumping capacity of the water truck, so only the falling head method (in this case a rising head method) was applied. The time was measured until the water level was equal to the initial water level.

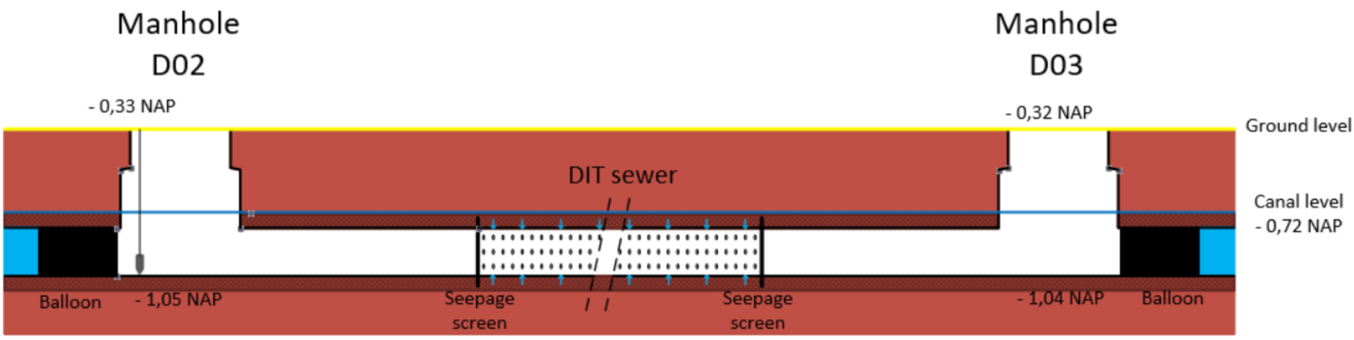

Figure 32 Longitudinal section of the tested DIT sewer segment during drainage test

3.3 Schaepmanstraat

The Schaepmanstraat is a street located in the south of Korte Akkeren, a neighborhood in Gouda (see Figure 33). The houses were all built between 1928 and 1949. The pavement has been in service for 9 years. A cross-section of the street at the tested location is given in Figure 35. The drainage pipe in this street is part of a drainage network in Korte Akkeren, which was constructed in 2009. In the Schaepmanstraat, a drainage pipe (Strabusil drainage, ø150) was installed at a depth of -2,61 m NAP (invert pipe level). It was installed in a 0,4 x 0,4 m casing of expanded clay aggregate (Argex AG4/8-320), which was wrapped in geotextile (brand unknown).

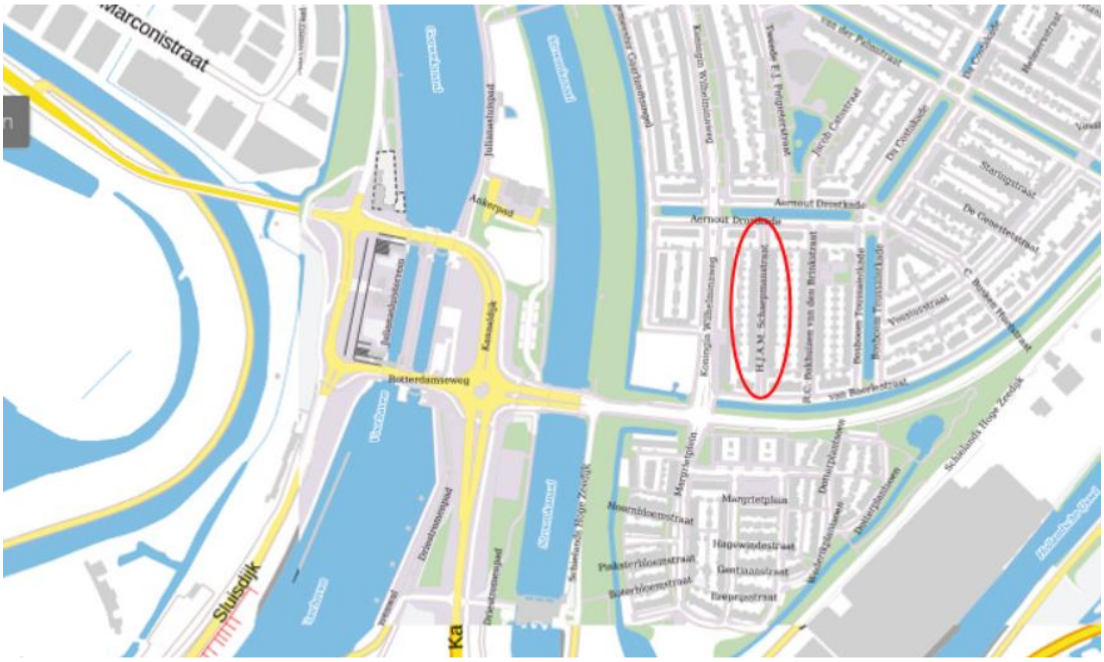

Figure 33 Geographic location of the Schaepmanstraat

To monitor the effects of the tests on the groundwater level around the DIT sewer monitoring wells were installed. Two rows with two monitoring wells (one next to DIT sewer and one on approximately 1 meter) were placed as depicted in Figure 34 and Figure 35. Pressure transducers were placed to monitor the groundwater level fluctuations during the test period.

Figure 34 Top view of the Schaepmanstraat with the locations of the monitoring wells

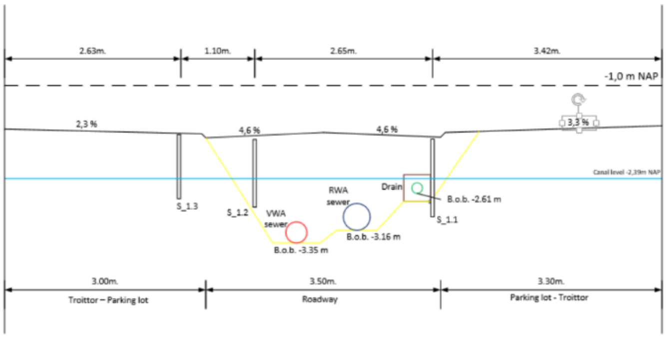

Figure 35 Cross-section Schaepmanstraat with the first row of monitoring wells

Figure 36 Impression of the Schaepmanstraat seen from the Aernout Drostkade

3.3.1 Inundation test

The same criteria as posed in the former paragraph are valid in this situation. From the relief map of the Schaepmanstraat (Figure 37) can be seen that the surface is not flat on any random part of the street. The street has several speedhumps and the curbs are approximately 10 centimeters high, which is beneficial for an inundation test. At the north side of the street, at the crossing with the Aernout Drostkade, the speedhumps have a spacing of 20 meters, which gives it an inundation surface of about 67 m^2. This part of the street (Figure 38) has also no entrée to any house and only four gully pots which gives it preference above other parts of the street.

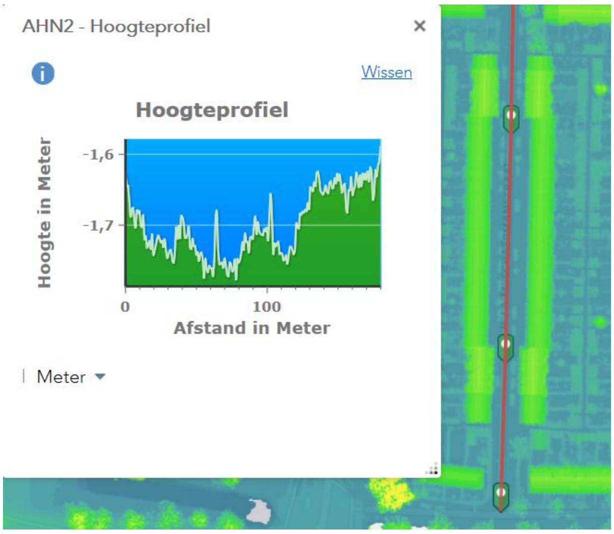

Figure 37 Relief map of the Schaepmanstraat

Figure 38 Schaepmanstraat with flooded area in yellow

The construction of a dam was not needed since the curbs and speedhumps are high enough for an inundation test. The street was flooded until the highest point, which is on the center of the street at the north end, has a water column of at least 1 cm. An impression of the inundation test is shown in Figure 39. For an average cross-section of the street with inundation, see Figure 40.

This water level was held constant for about 20 minutes according to CHFS method and the water supply was measured several times with a 50 L bucket. Then the water supply was stopped and the FHFS method was applied for approximately 100 minutes.

Figure 39 Impression of the Schaepmanstraat during the inundation test

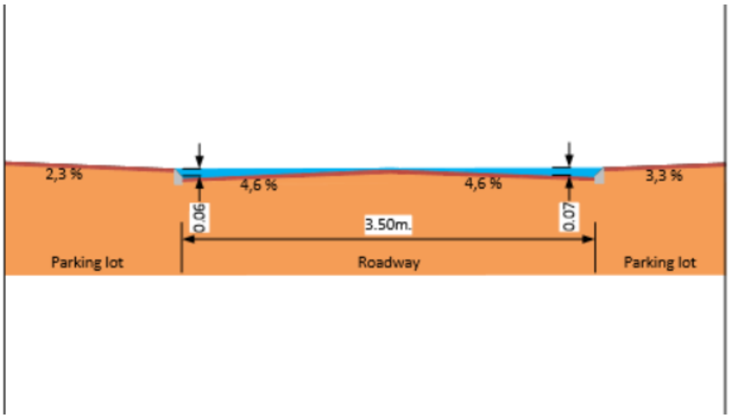

Figure 40 Cross-section of the Schaepmanstraat with inundation

3.3.2. Infiltration test

To determine the infiltration capacity of the drainage sewer, a pipe segment between manhole "Bestaand" and DO8 was isolated by placing rubber balloons as depicted in Figure 41. The total length of this segment is 49 meters. The initial water level in the manhole was measured. A pressure transducer was placed in manhole "Bestaand", whereafter the pipe was filled via this manhole until street level. This level was held constant, according to the constant head method. After several constant head measurements, the water supply was stopped and the time was measured until the water level was equal to the initial water level.

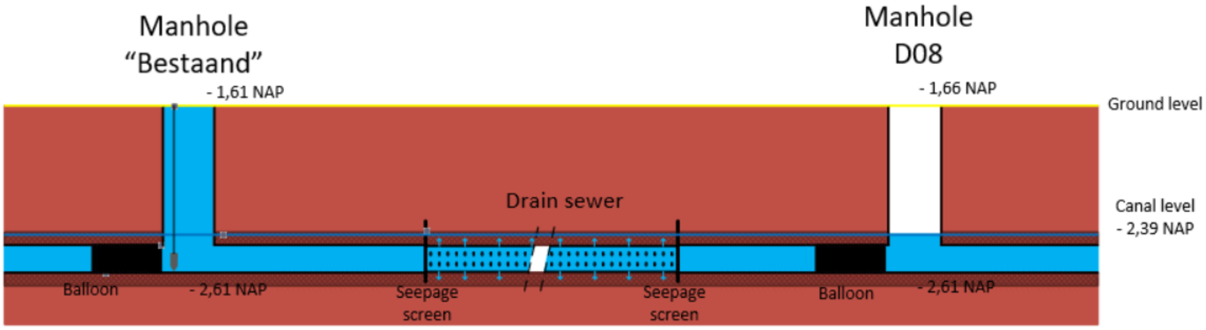

Figure 41 Longitudinal section of the tested drain sewer segment during infiltration test

3.3.3 Drainage test

To determine the drainage capacity of the DIT sewer, the same pipe segment between manhole "Bestaand" and DO8 was isolated by placing rubber balloons as depicted in Figure 42. Via manhole D08, the DIT sewer was completely emptied. The pressure transducer was placed in manhole "Bestaand". The drainage capacity was lower than the pumping capacity of the water truck, so only the falling head method (in this case a rising head method) was applied. The time was measured until the water level was equal to the initial water level.

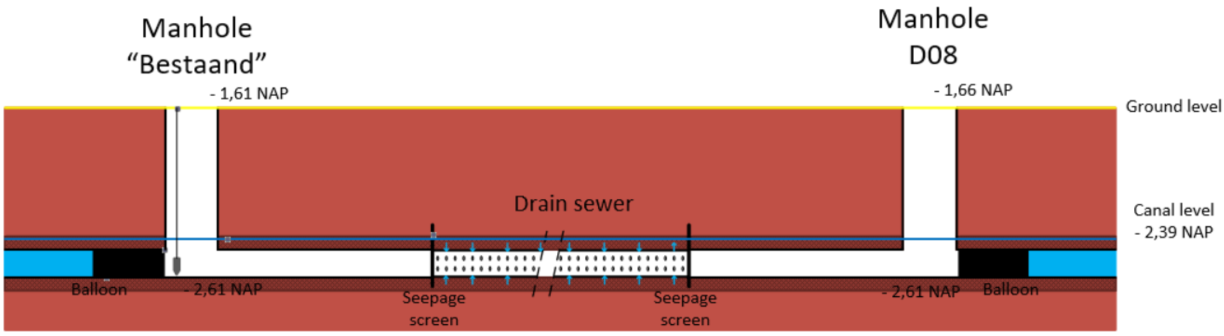

Figure 42 Longitudinal section of the tested drain sewer segment during drainage test

3.4 K-value

To compare values of different drainage systems, a parameter has to be found where relevant processes influencing the infiltration capacity are expressed. This parameter (further referred to as k) expresses a resistance dependent on the rate of infiltration (I) and the area through which the water infiltrates (A), at a given average potential difference between the nearest measured groundwater level and the water level in the manhole over the chosen period of time (dH):

$$\mathrm{k} = {}^{\mathrm{I}}\!/_{\mathrm{A}} \text{ at given dH}$$

This value k is expressed in L/m^2 pipe wall surface/h, which is basically the flux through the pipe wall.

3.5 Modeling in Hydrus 2D

To confirm our understanding of governing processes, the groundwater flow model Hydrus2D (version 2.05) is used to model the performed tests. The goal is to get a better understanding of the processes influencing the performance of the DIT sewer and the importance of the Argex drain envelope.

3.4.1 Introduction to Hydrus2D

Hydrus2D is a software program capable of simulating water flow in variable saturated porous media. In addition, simulating solute and heat flow is possible within the program. The program solves the Richards equation for saturated-unsaturated groundwater flow, for which for further details I refer to the Technical and User Manual (Šimůnek, 2012) (Šejna, 2012).

To let Hydrus achieve the defined goal, several parameters and constraints must be prescribed, including flow geometry, mesh generation, domain properties, initial conditions, and boundary conditions. The hydraulic model is solved with the van Genuchten-Mualem equations, which reads:

$$Soil\ water\ retention\ \theta(h) = \begin{cases} \theta_r + \frac{\theta_s - \theta_r}{[1+|\alpha h|^n]^m}, & h < 0 \\ \theta_s, & h \geq 0 \end{cases} \qquad \text{(Eq. 1)}$$

$$Hydraulic\ conductivity\ K(h) = K_s S_e^l\ [(1-(1-S_e^{l\frac{1}{m}})^m]^2 \qquad \text{(Eq. 2)}$$

$$m = 1 - \frac{1}{n}, \qquad n > 1$$

$$Effective\ water\ content\ S_e = \frac{\theta - \theta_r}{\theta_s - \theta_r} \qquad \text{(Eq. 3)}$$

h = pressure head (cm)
θ_r = residual volumetric water content [-]
θ_s = saturated volumetric water content [-]
K_s = saturated hydraulic conductivity (cm/min)
α = inverse of air entry pressure (cm^{-1})
m = coefficient related to n, the pore size distribution index [-]
n = pore size distribution index [-]
1 = pore connectivity, average value for soils is 0,5 [-]

Hereafter, these groundwater flow parameters are selected for the equations. The two-dimensional plane is drawn according to the geometry of the tests and a Finite Element mesh is created. The resulting generated mesh can be refined or stretched to the users' needs. Refinement is for instance needed along the flux boundaries (a drain) and near the water table. Within the mesh, observation points can be depicted from which the required information can be retrieved, which can either be pressure head or water content.

Within the domain, materials and their distribution are defined. The saturated and unsaturated properties of the materials include hydraulic conductivities, porosity, residual saturation, alpha, and n. The initial conditions provide the starting point for the equations to be solved and are defined for the pressure head. The water table is the zero plane with positive values below (which is the saturated zone) and negative values above (which is the unsaturated zone). Lastly, boundary conditions need to be defined for each boundary.

To calibrate the model with the field data, Hydrus offers a tool called Inverse Solution. Hydrus implements a Marquardt-Levenberg type parameter estimation technique for an inverse estimation of the soil hydraulics and/or solute transport from the measured flow and/or transport data. This nonlinear minimization method has proven to be very effective and has become a standard in nonlinear least-squares fitting among soil scientists and hydrologists (van Genuchten, 1981). In the inverse solution, Hydrus produces a correlation matrix which specifies the degree of correlation between fitted coefficients. This matrix quantifies changes in model predictions caused by small changes in the final estimate of a parameter, relative to similar changes as a result of changes in other parameters. Then it reflects the nonorthogonality between two parameter values: ± 1 means perfect linear correlation whereas 0 indicates no correlation at all. Based on these values, the program decides which parameters are best kept constant (or not) in the parameter estimation process. As a measure of fit, the r^2 value for regression is used. This is a measure of the relative magnitude of the total sum of squares of residuals associated with the fitted equation; a value of 1 indicates a perfect correlation between the fitted and observed values (Šimůnek, 2012). For further, more extensive, explanation of the inverse solution, I refer to the Technical Manual.

Possible problems in the inverse solution may arise and are related to convergence and parameter uniqueness. Therefore, it is important to check if the program does converge to the same global minimum in the objective function by rerunning the program with different initial parameter estimations. This is especially important with field data sets, which can often show significant scatter in the measurements or cover only a narrow range of the soil water contents or pressure head.

3.4.2 Methods per test

General

For the iteration criteria of the model simulations, only the pressure head tolerance was relevant and set on 1 cm, with the maximum number of iterations on 10, which is default. The time step control parameters were left on default and the initial conditions are in pressure head.

In the soil hydraulic model, the Van Genuchten – Mualem equation is used with an air-entry value of -2 cm. No hysteresis is applied.

Kuiperstraat – Infiltration test

An average cross-section of the Kuiperstraat over the tested part of the DIT sewer with the drain envelope of Argex granules was constructed. Chosen is to construct only half of the drain since conditions on both sides of the drain are expected to be the same. The model simulated 480 minutes from 11/10/2018 8:30 until 15:00. With the found values for the infiltration capacity of the two infiltration tests, the fluxes into the soil over the (half) drain perimeter were calculated. For the first test, the average flux from 8:52 until 9:49 was used as input and the fluxes after the end of this test were estimated by fitting them into the curve outcome. For the second test, the average flux from 10:22 until 11:28 was used as input and the fluxes after the end of this test were estimated.

The initial condition was assumed to be constant over the chosen cross-section and was -0,71 m NAP. The boundary condition of the drain was variable flux, negative indicating infiltration into the soil, with the calculated fluxes as input. The left boundary condition was constant head, with the same head value as the initial condition. All the other boundaries were defined as no flux.

The saturated volumetric water content θ_s and saturated hydraulic conductivity K_s were estimated and the calibrations were done. One calibration was performed with the inverse solution. The first 105 minutes of field data of the first observation node (K_1.1) and the second observation node (K_2.2) were used to get value for θ_s and K_s. The outcomes were implemented into the model and validated for minute 106 till 480. Hereafter, an anisotropy ($K_{s,v}/K_{s,h}$) of 1/5 was added into the model. The water flow parameters were further slightly adjusted by trial and error. Further model optimization was done by adding fluxes after the peaks of both infiltration tests, which were not measured during the test but exist nonetheless.

The complete detailed model structure can be found in Chapter 5.

Kuiperstraat – Drainage test

In this simulation, the same average cross-section of the Kuiperstraat was used as in the simulation of the infiltration test. Also, half the drain was modeled. The model simulated 1380 minutes from 17/10/2018 9:00 until 18/10/2018 8:00. With the found values for the drainage capacity of the drainage test, the fluxes into the pipe over the (half) drain perimeter were calculated. From 17/10/2018 09:25, the fluxes were calculated over every 2 hours until the end of the modeled period.

The initial condition was assumed to be constant over the cross-section and was -0,73 m NAP. The boundary conditions were the same as in the infiltration test. The variable flux in this simulation was positive, indicating a flux from the soil into the drain pipe.

Values for the relevant water flow parameters θ_s and K_s were initially taken from the outcomes of the infiltration test. Also, the same anisotropy was added as in the infiltration test simulation.

Two calibrations were done to get an indication of θ_s and K_s for this test. The first calibration was done over the first 10 hours with the field data of both K_1.1 and K_2.2. Then a second calibration was done with only the field data of K_2.2 with the first 12 hours of field data. From the first calibration, results for the drain envelope were obtained and from the second calibration, the water flow parameters of the soil itself is obtained. These values were validated for the remaining field data.

Schaepmanstraat – Infiltration test

A cross-section of the Schaepmanstraat at the location of monitoring well row 1 (with S_1.1, S_1.2, and S_1.3), with the drain envelope of Argex granules, was constructed. Chosen is to construct only half of the drain since conditions on both sides of the drain were expected to be the same. The model simulated 330 minutes from 11/10/2018 12:30 until 18:00. With the perceived value for the infiltration capacity of the infiltration test, the average flux into the soil over the (half) drain perimeter was calculated from 12:40 until 13:23. Fluxes after the end of the test were estimated by fitting them into the curve outcome.

The initial condition was assumed to be constant over the chosen cross-section and is -2,39 m NAP. The boundary condition of the drain was variable flux, with the before mentioned calculated fluxes as input. The left boundary condition was constant head, with the same head value as the initial condition. All the other boundaries were defined as no flux.

The saturated volumetric water content θ_s and saturated hydraulic conductivity K_s were estimated and the calibrations were done. Three calibrations were performed. A calibration was done with the field data of monitoring well S_1.1 over the entire monitored period of 330 minutes. Another calibration was done with the field data of monitoring well S_1.2 over the entire monitored period of 330 minutes. Another calibration was done with the field data of monitoring well S_1.3 over the entire monitored period of 330 minutes. Outcomes of the calibrations were checked on consistency. Since S_1.1 lies directly next to the drain envelope, values for θ_s and K_s were taken from this calibration. Since it is not sure if S_1.3 is located in the sewer trench, firstly the outcomes of the calibration of S_1.2 were used as input. Further optimization was done by trial and error and fitting the simulation outcomes into the test results.

The complete detailed model structure can be found in Chapter 5.

Schaepmanstraat – Drainage test
In this simulation, the same average cross-section of the Schaepmanstraat was used as in the simulation of the infiltration test. Also, half the drain was modeled. The model simulated 750 minutes from 17/10/2018 11:30 until 18/10/2018 0:00.

The initial condition was assumed to be constant over the cross-section and is -2,39 m NAP. The boundary conditions were the same as in the infiltration test. The variable flux in this simulation was positive, indicating a flux from the soil into the drain pipe.

In the simulation of the drainage test, a different approach was applied since in the performed test no values for the drainage capacity could be found. Instead of calibrating the model with the field data to find the water flow parameters, the water flow parameters of the infiltration test were used to calibrate the model for the fluxes into the drain. From the field data, two flux periods were defined, namely from 11:49 till 12:05 and from 12:06 till 0:00. Hydrus does not offer the inverse solution to calibrate for the time-variable fluxes, so two fluxes for these two periods were obtained by trial and error.

4. Results

In this chapter, the results of the performed tests, as discussed in Chapter 3, are shown and analyzed. Firstly, the tests in the Kuiperstraat will be presented and discussed, with the inundation test, infiltration test, and drainage test respectively. Secondly, the tests in the H.A.J.M. Schaepmanstraat are presented and discussed, with the inundation test, infiltration test, and drainage test respectively.

4.1 Calibration of results

The acquired data is retrieved from the divers and calibrated to the proper heights with help of the program Diver Office, which is provided by the manufacturer of the pressure transducers, van Essen B.V. Further calibration is done by hand measurements.

Some data show noise due to inaccuracy of the divers and need to be “smoothened”. This is done by the moving-average method, with n varying from 5 to 10. The raw data with the applied moving-average method can be found in the appendices, which are specifically depicted in each section.

4.2 Kuiperstraat

4.2.1 Inundation test

Due to some complications before starting the test, the test was not performed as intended, which will be elaborated on in the discussion part. Consequently, the total inundation area shrank from the intended 52 m^2 to approximately 41 m^2.

In Figure 43, the inundated area is shown with the location of the divers and the location of the hand measurements, which were used to calibrate the diver data.

The raw data calibrated with the hand measurements can be found in Appendix 1.

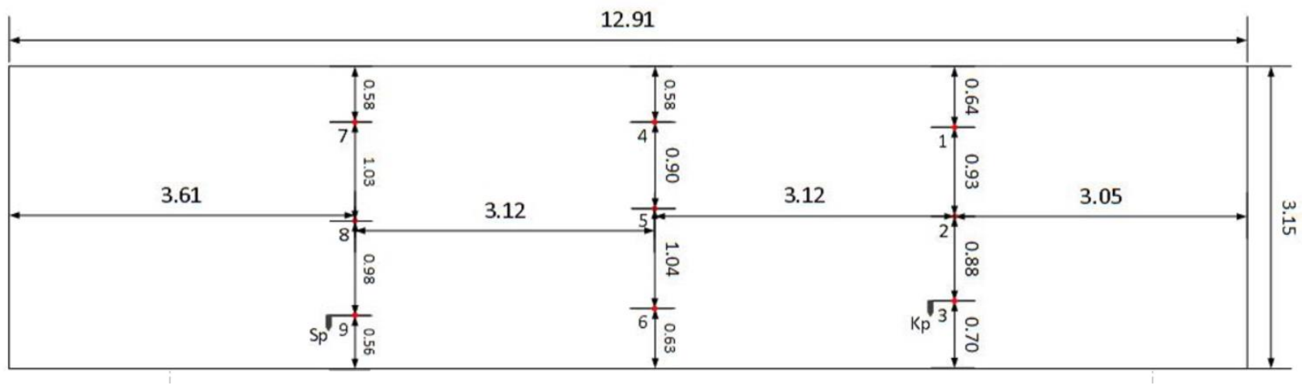

Figure 43 Inundated area in the Kuiperstraat with locations of rulers for hand measurements (red dots) and the divers Sp and Kp

From the graphs in Appendix 1 can be observed that the water pressure over time shows noise. This can be addressed to two causes, namely the accuracy of the divers and the windiness in the street. The typical accuracy of the divers is 0,5 cm, while the water level dropped less than this value in the measuring time step. So, after calibrating the raw data with the hand measurements, the next step was to only extract the values of every minute. This data is shown in the graph in Appendix 1, where the time lap of the test is set at 13:59 till 15:12. The blue line is the data set from every minute. This line still shows noise, so it is “smoothened” with the moving average method. The orange line is the moving average method with n=5, while the gray line shows n=8. These same steps are performed for the diver Sp which was on the other side of the test area. The graph of this diver is shown in Appendix 1.

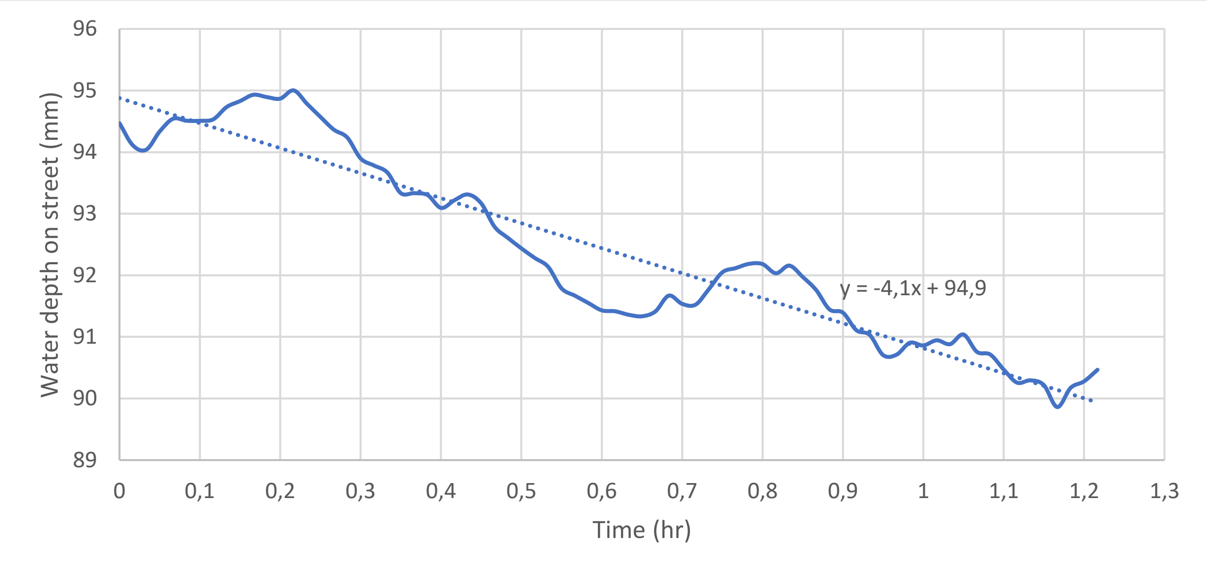

Figure 44 Water level drop in the Kuiperstraat during the inundation test at location of diver Kp with regression line

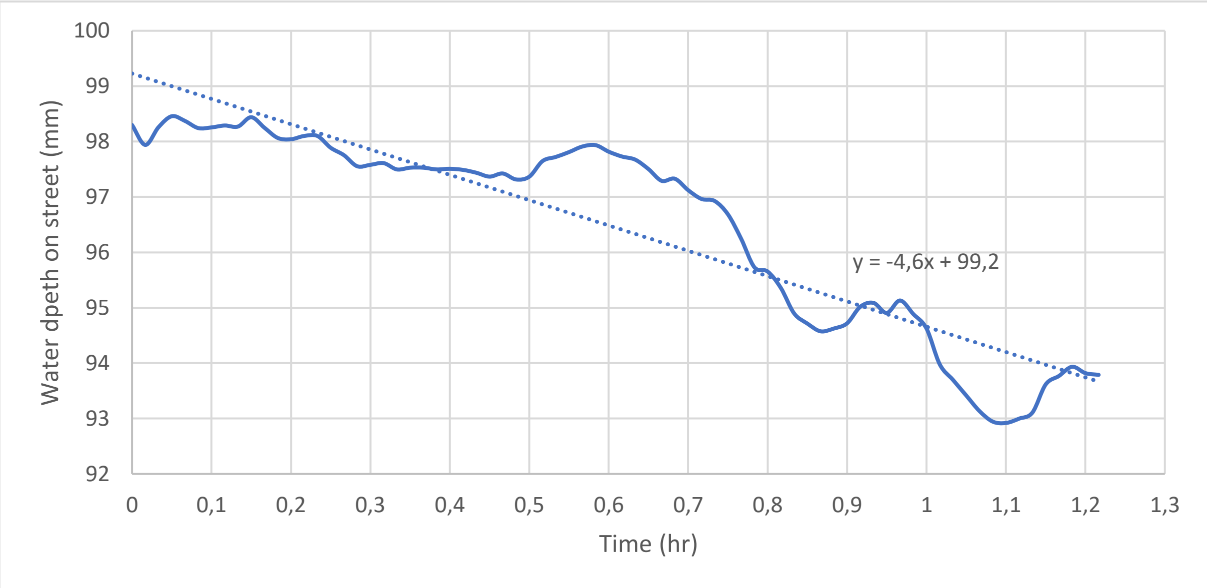

Figure 45 Water level drop in the Kuiperstraat during the inundation test at location of diver Sp with regression line

The graphs with the calibrated data of both divers are shown in Figure 44 and Figure 45, with the corresponding linear regression lines. Diver Kp shows an infiltration value of 4,1 mm/h. Diver Sp shows an infiltration value of 4,6 mm/h. While the line shows infiltration, it can be discussed if this infiltration is linear over time.

Taking a closer look at the regression lines of both divers, a phenomenon can be noticed. In diver Kp, a drop can be noticed from approximately 0,5 till 0,7 hr, while in diver Sp a hump is shown at the same time step. The same can be seen in the time steps 0,8 till 0,9 hr and 1,0 till 1.1 hr. This suggests a shift of water from Kp to Sp and back due to the before mentioned wind. Therefore, the water levels of diver Kp and Sp are added up and plotted, whereafter a new linear regression line is formed, which corresponds with an infiltration value of 4,3 mm/h, as shown in Figure 46.

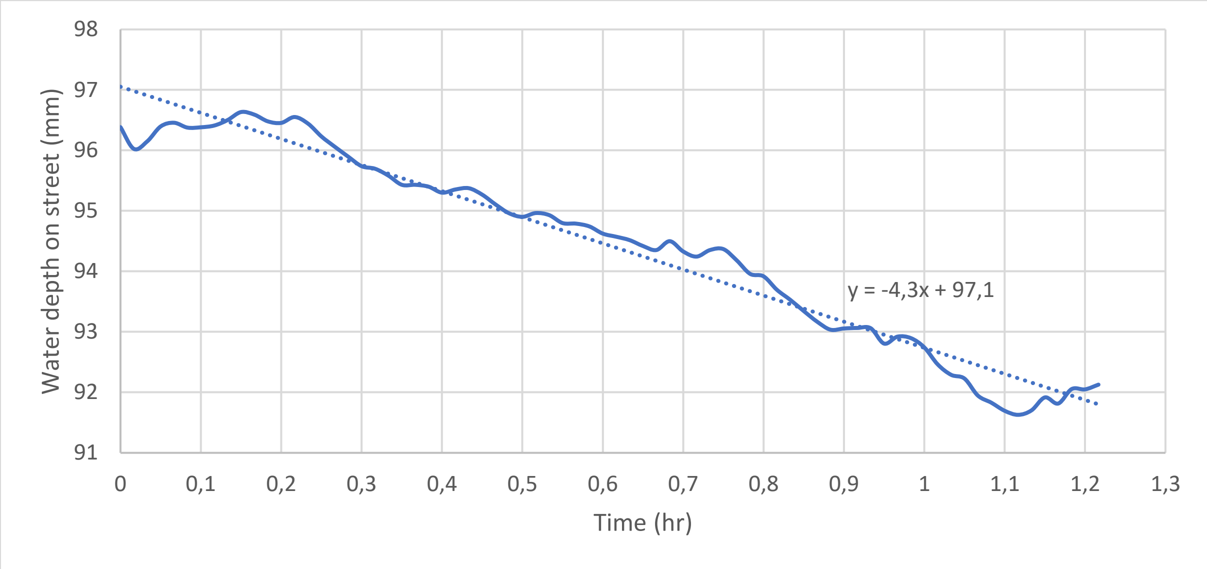

Figure 46 Average drop-down of diver Kp and Sp during the inundation test Kuiperstraat with regression line

This low infiltration velocity incorporates significant runoff in case of rainfall intensities higher than this value. The need for a stormwater transport sewer and the potential for infiltration is therefore founded. A DIT sewer directing the flow (partially) to the soil shows potential based on these numbers.

Discussion

The inundation of the area in the Kuiperstraat was a complex task, due to the location of the street, the narrowness of the street with the doorsteps directly at the pavement and the lateral steepness of the road. Starting the test, the first issue encountered was the lack of material for the construction of the dam, which was apparently miscommunicated with the contractor. Therefore, we had to improvise, which meant that infiltration into the dam was possible. When the dam was removed, it showed a water penetration depth of about 10 cm. With a wetted perimeter of 13 meters, a water height of about 10 cm and an assumed porosity of clay of 0,4, the water loss is estimated. The infiltration value into the clay dam is estimated at approximately 0,5 mm/h. The infiltration value of the pavement in the tested area in the Kuiperstraat is then set on 3,8 mm/h.

4.1.2 Infiltration test

The infiltration test was performed on the pipe segment between manhole D02 and D03, which has a length of 24 meters, with an infiltration area of 9,3 m^2, as described in chapter 3.

Test

The first falling head test lasted one hour. Then the water was sucked out of the pipe and the pipe was cleaned, whereafter a second falling head test was applied. The raw data can be found in Appendix 2. This graph shows two lines with a negative slope, which are the two infiltration tests. The second test shows some instability at the start which is due to the entrapped air inside the pipe after the second filling. After all the air was out, so the bubbling stopped out of the manholes and gully pots, the manhole was again filled until street level and the second test started.

Figure 47 shows the water level during the first infiltration test, which is slightly downward parabolic. Since the raw data showed noise, the moving average method is applied (n=8) to smoothen the line. Figure 48 shows the infiltrated water volume per time step. This is calculated by taking the water level each minute during the test and multiplying the water drop in this time step with the surface in which the water drops. This surface is composed of 2 manholes and 2 gully pots. The manholes are of the type Tegra 600. The diameter of this manhole changes in the water level drop interval, which is accounted for.

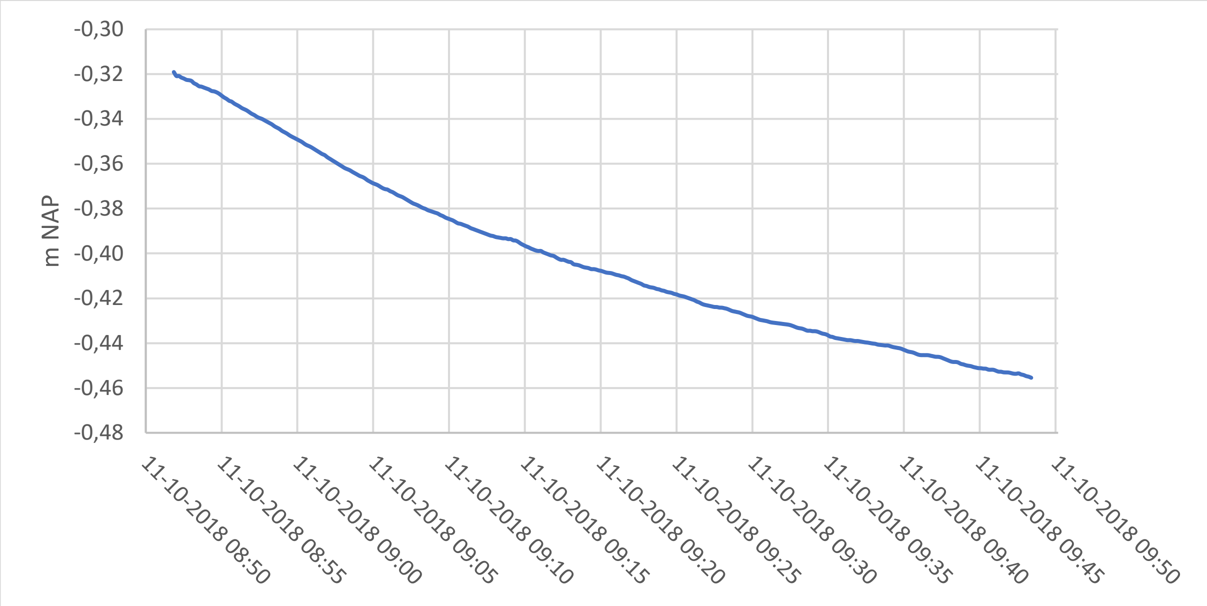

Figure 47 Water level drop in manhole D03 during infiltration test 1 in the Kuiperstraat

The infiltrated volume over time shows a slight parabolic function, indicating that the infiltrated volume is dependent on the pressure difference between the water level in the manhole and the groundwater table. During the water level drop, this pressure difference becomes less and the rate of infiltration becomes lower. The trendline shows the linear estimation of infiltration for this particular DIT sewer segment, which is 84,5 L/h. During the test, 6 hand measurements were taken, showing similar results with an infiltration value of 83,5 L/h. This graph can be found in Appendix 2.

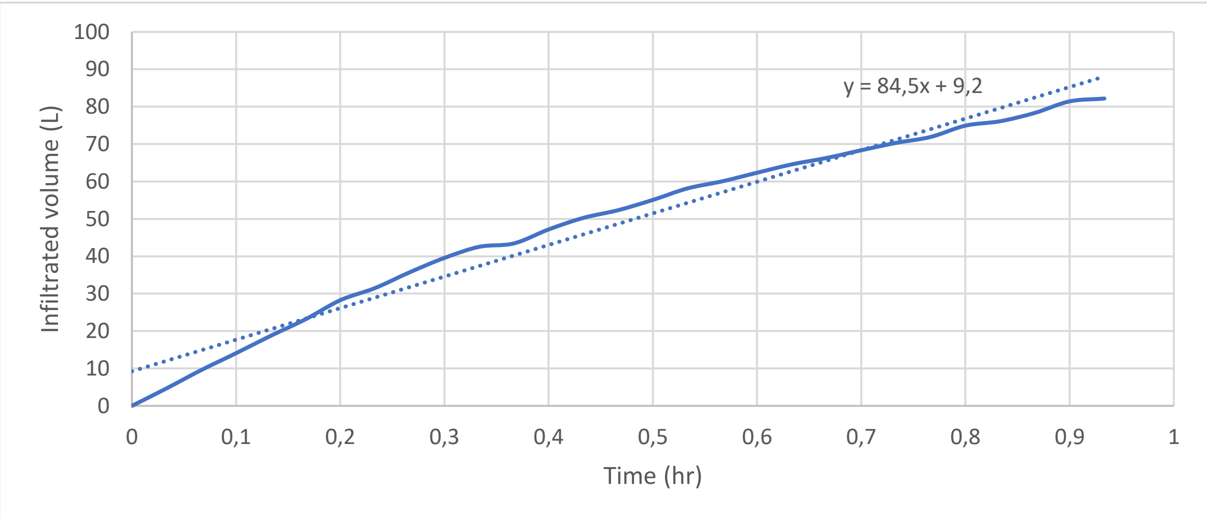

Figure 48 Cumulative infiltrated volume of water during infiltration test 1 in the Kuiperstraat over time

The infiltration area (A) is the length of the drain (which is 12 meter) times the perimeter of the drain (which is 0,79 m). dH is defined as the difference between the average groundwater level next to the drain during the test and the average water level in the manhole, which is 0,28 m. For this first infiltration test, the k-value is calculated on -9,0 L/m^2/h at dH = 0,28 m, with a negative value indicating water leaving the pipe.

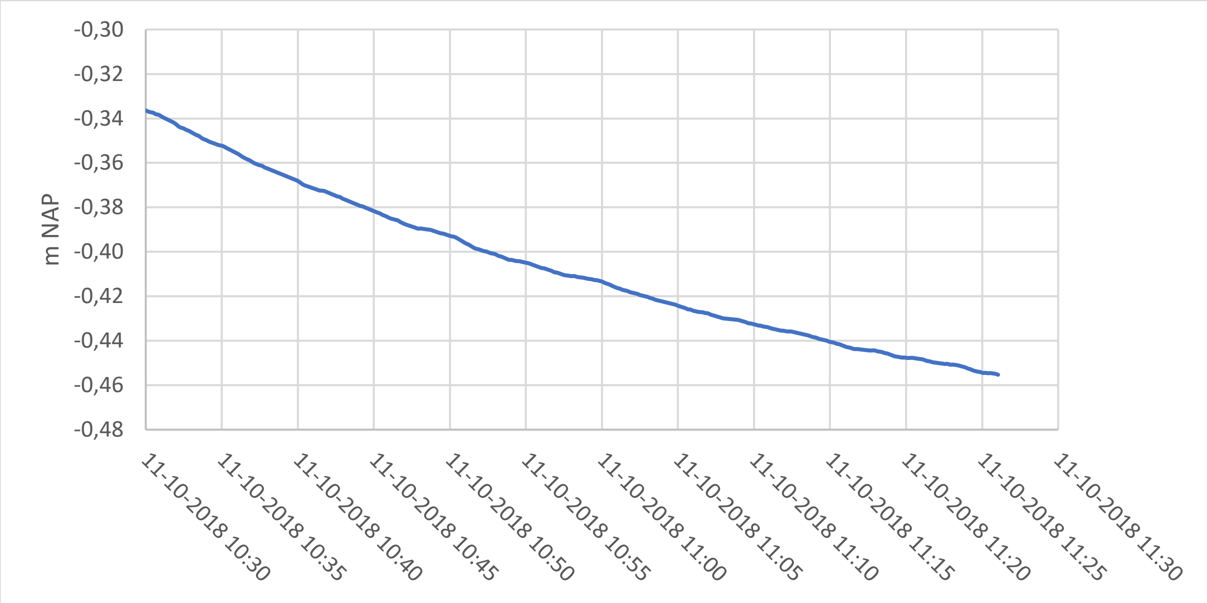

Figure 49 Water level drop in manhole D03 during infiltration test 2 in the Kuiperstraat

Figure 49 shows the water level during the second infiltration test. Since the raw data showed noise, the moving average method is applied (n=8) to smoothen the line. Figure 50 shows the infiltrated water volume per time step in this test. The trendline shows the linear estimation of infiltration for this particular DIT sewer segment, which is 73,5 L/h. This corresponds with a k-value of -7,8 L/m^2/h at dH = 0,24 m.

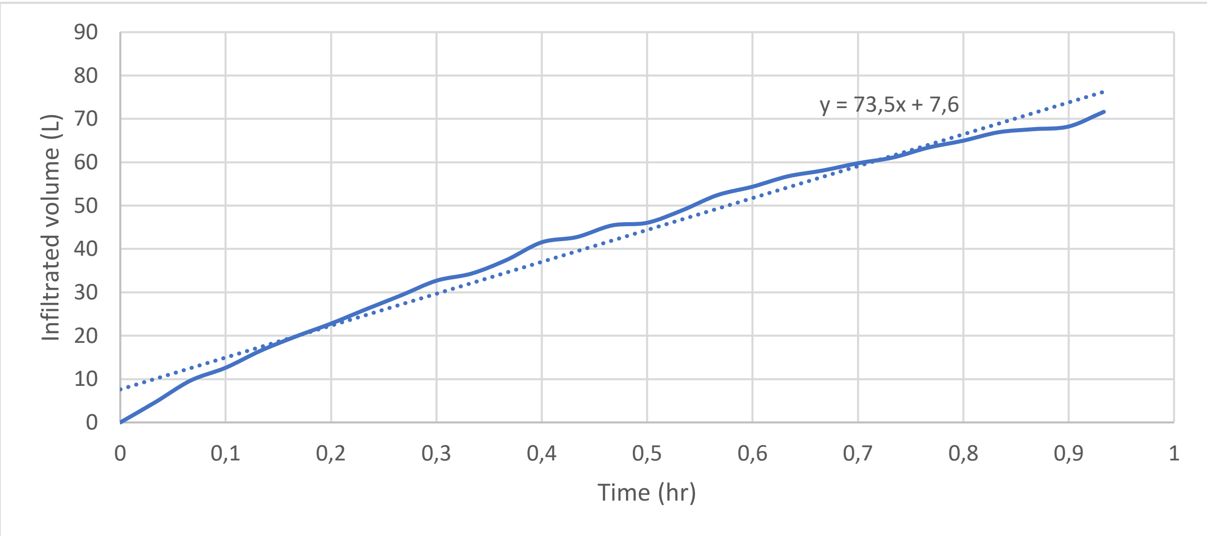

Figure 50 Cumulative Infiltrated volume of water during infiltration test 2 in the Kuiperstraat over time

Groundwater levels

The groundwater levels in the time frame of the infiltration test are shown in Figure 51. Also shown is the water level development in the sewer, as well as the location of the pipe and the ground level.

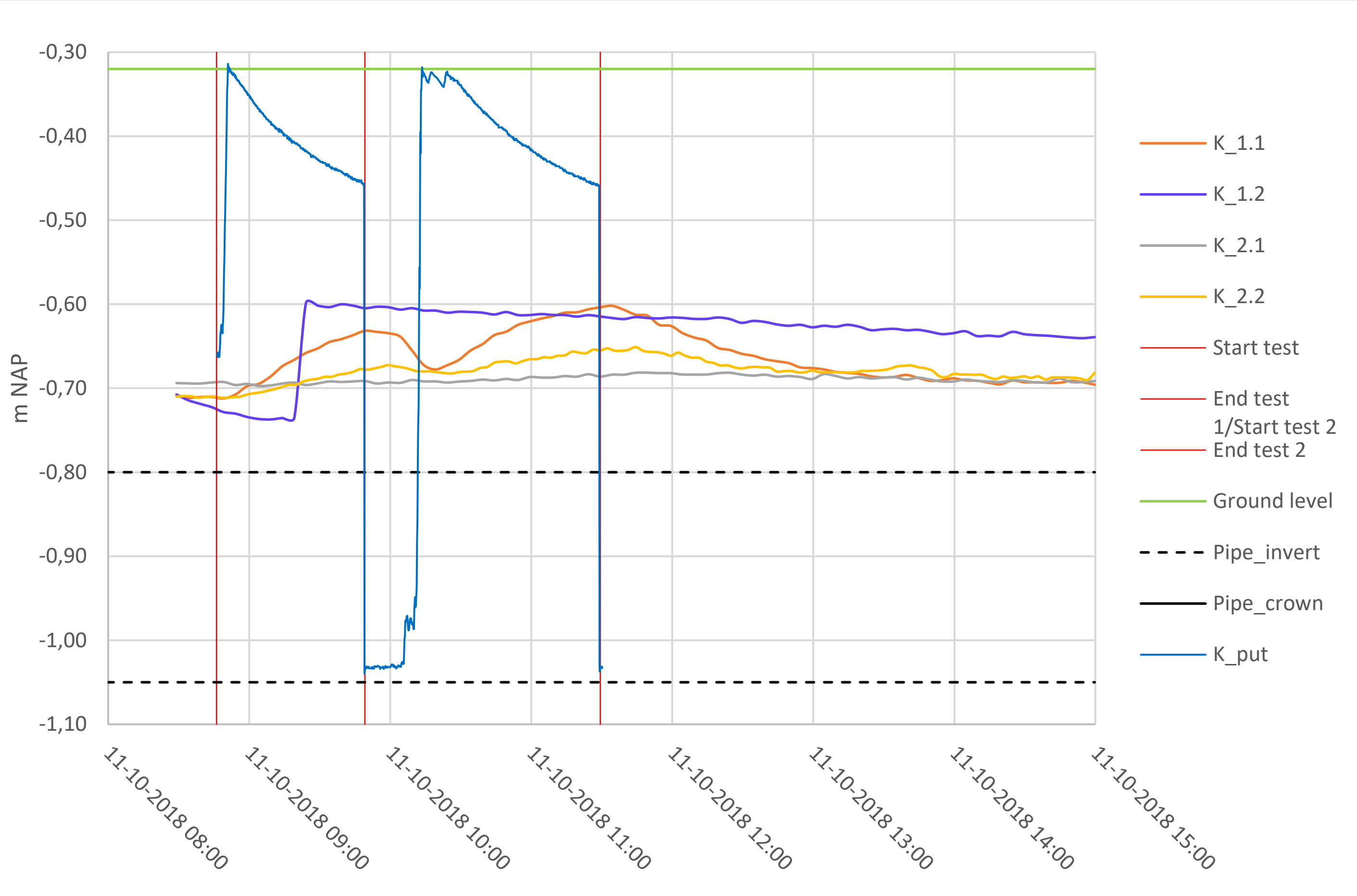

Figure 51 Infiltration test Kuiperstraat with water level in manhole DO3, groundwater levels in the monitoring wells K_1.1, K_1.2, K_2.1 and K_2.2 with location of pipe and streetlevel

K_1.1 is the monitoring well directly next to the drain pipe, at the side of the Raam and the orange line in the graph. The groundwater level responded very well and the two tests can be clearly distinguished. The test started at 08:47 and the manhole was full at 08:50. It takes approximately 7 minutes until the monitoring well responded after the water level was at street level. From a groundwater level of -0,71 m NAP, it went up to -0,63 m NAP. Immediately after the test was ended, the groundwater level dropped again. During the jet flushing, it dropped further. When the second test was started, the groundwater level was at -0,68 m NAP. During the test, it rose again and reached its maximum value just after the test was stopped, which was -0,60 m NAP. Both tests lasted one hour and in both tests, the groundwater level rose 8 cm. This indicates that the water infiltrated directly and homogeneously into the soil during these tests. In the graph can be noticed that at 15:00, which is 3,5 hours after the tests, the groundwater level was almost at its original level. From the data is retrieved that the exact original level was reached at 21:00.

K_1.2 was placed at the same location in the street as K_1.1 only at 1 meter from the DIT sewer. Expected is that this monitoring well shows the same behavior as K_1.1, but only has a weaker response. The data (purple line) shows, however, a slight decline of the groundwater level until 09:20, after which it suddenly rose instantly to -0,60 m NAP. This behavior indicates, that this monitoring well was clogged at that time and due to the building pressure during the test unclogged and the groundwater suddenly poured in. It shows zero reaction to the second test and this data is therefore not useful for this test.

K_2.1 is the monitoring well directly next to the drain, only at a distance of 12 meters from the first row of monitoring wells. The grey line does not show a distinction between the two tests but shows a slight rise of 1,6 cm until the maximum was reached 25 minutes after the test. The reason can be a local tightly packed soil around the monitoring well which limited the water to flow in and was therefore not susceptible to sudden changes in the groundwater level.

K_2.2 was placed at the same location in the street as K_2.1 only at 1 meter from the DIT sewer. The expectation of a weaker but distinct reaction compared to the groundwater level next to the drain is fulfilled at this location. The yellow line shows a rise in the groundwater level during the first test, which reached its top 10:00 with a value of -0,67 m NAP. Note that this top arrived slightly later compared to K_1.1, which makes sense considering the larger distance away from the drain. The second test is also clearly visible in this data. The groundwater level reached its maximum at 11:48 with a value of -0,65 m NAP. In returning to its original groundwater level, it followed the same path as described in analyzing K_1.1.

Although two of the monitoring wells give poor results, conclusions can be drawn from the other two locations. The groundwater level next to the drain pipe and on 1 meter of the drain pipe responded to the test, indicating that the DIT sewer fulfills its infiltration purpose in case of a high surface water level.

Discussion

Ideally, an infiltration test should be performed in several steps with for each step a different water height in the manhole. As such, a more accurate relation can be found for the pressure difference between the groundwater level and the simulated surface water level. In Gouda, the difference between the groundwater table and the street level is very small, so performing the test in different steps is a time-consuming procedure with little result.

Due to problems during the placement of the monitoring wells, the monitoring well at 3 meters of the DIT sewers is absent at both rows. Information on the groundwater table variations at these locations is lacking, while this data could give more insight into the behavior of the system. Since two monitoring wells were compromised, the groundwater level data in this test is not as desired. Data analysis is, however, still possible since the data from K_1.1 and K_2.2 give a clear response.

Performing a second test directly after the first test gave a lower initial infiltration value, which was expected considering the higher groundwater level in the second test in comparison to the first test. The resulting lower potential difference of 4 cm gave logically a lower infiltrated volume. An intended conclusion on clogging of the pipe cannot be drawn from this second experiment.

4.1.3 Drainage test

The drainage test was also performed on the pipe segment between manhole D02 and D03, as described in chapter 3. The main difference in setup between the two tests is the place of the diver which is in this test in manhole DO2 instead of D03 since it is not possible to place the diver in a manhole where also water is sucked out by the pump.

Test

After the sewer segment was isolated, the water in the pipe was sucked out. When there was (almost) no inflow into the pump anymore, it was switched off and the test started. The data from the diver can be found in Appendix 3. Since it was possible in this test to leave the sewer segment isolated for a longer period, the total test has a duration of 56 hours. Then the balloons were removed, hence the peaks on 19/10, 14:40. Hereafter, the diver is left in the manhole for another day.

Figure 52 shows the water level in manhole DO2 during the drainage test. This graph is calibrated with several hand measurements taken during the test. The water level rise consists based on the development of this line of three stages. The first stage lasts 23,5 hours until it reaches 0,88 m NAP. This stage demonstrates a linear water level rise. Figure 53 shows this process translated to drained volume. An estimation of the drainage capacity in this stage is 33,0 L/h for this particular DIT sewer pipe. This corresponds to a k-value of 3,5 L/m^2/h at a dH value of 0,10 m. In the second stage, a remarkable phenomenon occurs. The last part of the pipe fills rapidly until the pipe is full and the rate of drainage becomes constant again. Hereafter, the water level in the manhole equilibrates with the groundwater level until it reaches the original groundwater level, which is the third stage.

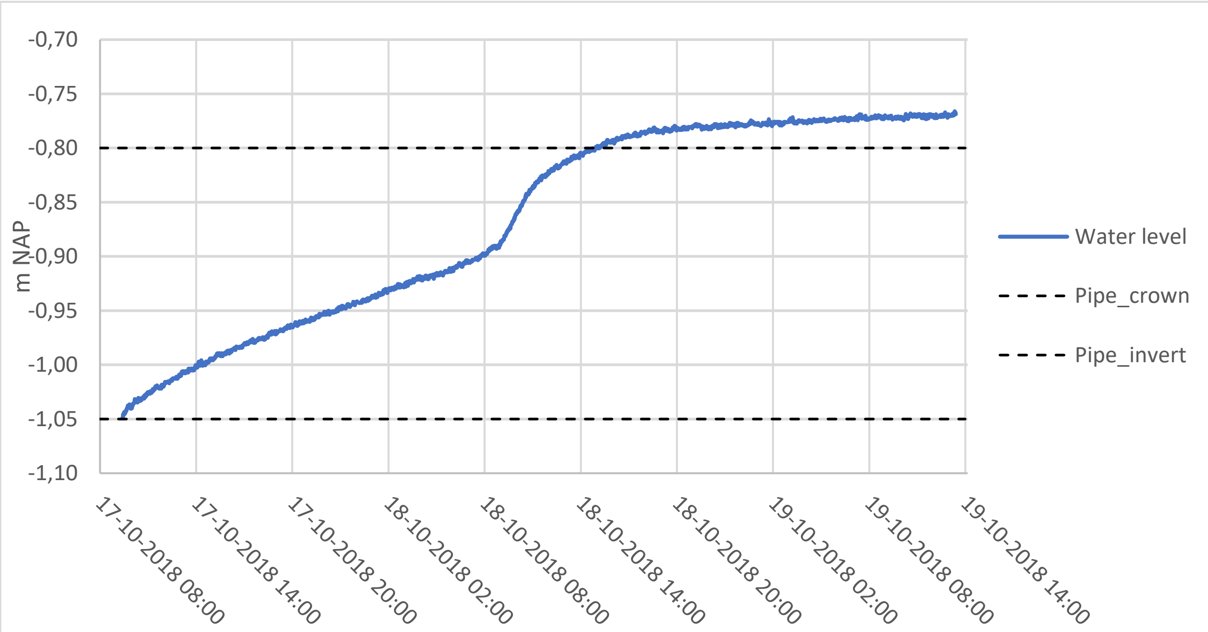

Figure 52 Water level in manhole D02 Kuiperstraat during in drainage test with location of the pipe crown and invert level

What happens in this second stage of filling is unsure. The same phenomenon is noticed in the drainage test performed by Abbas in the Argonautenstraat in Amsterdam, see Appendix 3. In the next paragraph on groundwater levels is some elaboration on this matter.

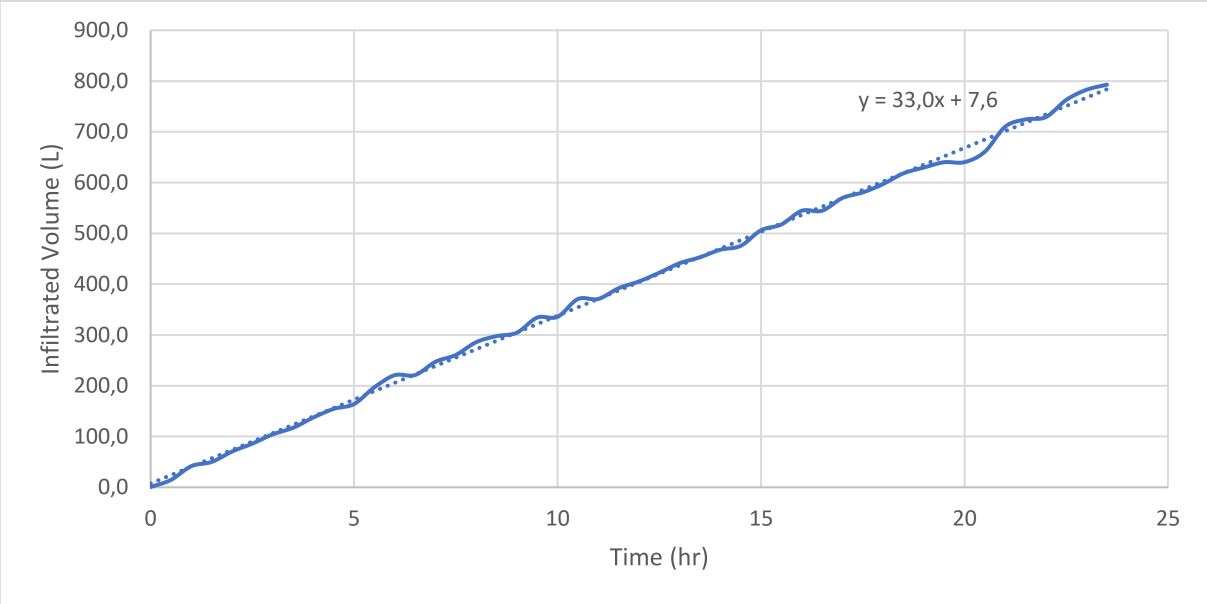

Figure 53 Cumulative water volume inflow into DIT sewer of the Kuiperstraat during the first 23,5 hours of the drainage test with regression line

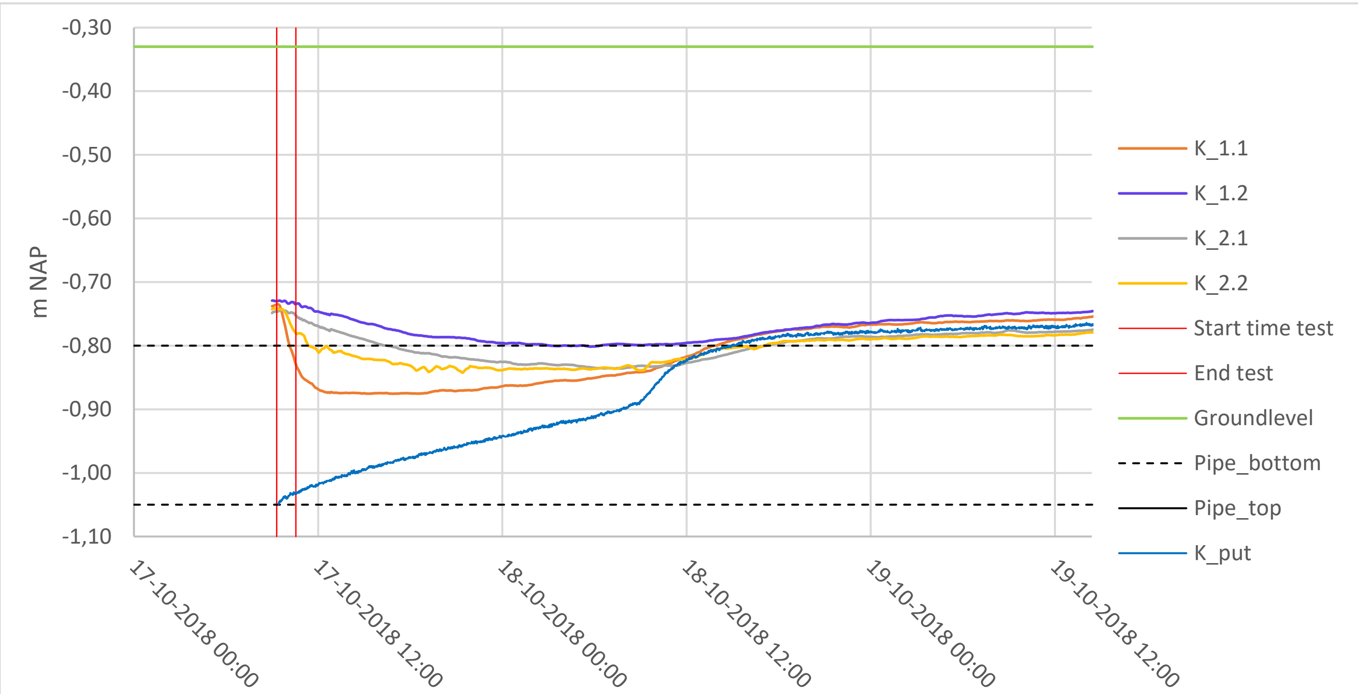

Figure 54 Drainage test Kuiperstraat with water level in manhole DO3, groundwater levels in the monitoring wells K_1.1, K_1.2, K_2.1 and K_2.2 with location of pipe and street level

Groundwater levels

The groundwater levels in the time frame of the drainage test are shown in Figure 54. Also shown is the water level development in the sewer, as well as the location of the pipe and the ground level.

As in the infiltration test, monitoring well K_1.1 responded very well to the test. When the test was started, the groundwater level dropped immediately from -0,73 m NAP to -0,87 m NAP in 3 hours. This clearly indicates a responding flow into the drainage pipe. This level stayed more or less constant for 10 hours after which it started to rise again. So, groundwater was supplied from further away. This claim is supported by the behavior of monitoring well K_1.2

Unlike the response in the infiltration test, K_1.2 reacted well in this test. Since the distance is further away from the pipe, the reaction was later as can be noticed in the graph. The response was also weaker, from -0,73 m NAP it took 13,5 hours to get to the minimum of -0,80 m NAP. This level remained constant for approximately 12 hours, whereafter the groundwater level started to rise again to the original water level.

Monitoring well K_2.1 gave a sensible reaction. Although, when compared to the reaction of K_2.1 the premise is again confirmed that this monitoring well responded less to sudden changes in the groundwater level. K_2.2 gave 7 centimeters in a timeframe of 3 hours. In these 3 hours, K_2.1 gave approximately the same drop in water level, but as can be observed from the development of the groundwater level, this drop was more gradual, so sudden changes (< 3 min) were less presented in this monitoring well. Both groundwater levels restored to its original level in the same manner as K_1.1 and K_1.2.

Resulting from the groundwater level reactions and the inflow into the drain pipe, the DIT sewer in the Kuiperstraat is working well. The reaction to the test was immediately in both the water level rise in the pipe as in the groundwater levels.

Discussion
During the first stage of filling, the k-value is calculated on 3,5 L/m^2/h at a dH value of 0,10 m. This value is in the same order of magnitude as the estimated values in the infiltration test. However, since the groundwater level next to the drain pipe dropped below the crown of the pipe, not the entire surface of the pipe was contributing to the drainage capacity in contrast to the infiltration test. This implicates that the k-value of 3,5 L/m^2/h could be an underestimation.

A specific point of discussion is the behavior of the second stage of pipe filling. The groundwater levels in the monitoring wells were not decreasing during this stage. This indicates that there is no extra water volume infiltrating into the pipe. The reason can, therefore, be searched in less available storage. This could be due to air entrapment in the pipe during the last centimeters of pipe filling, so less storage for infiltrating water. Another reason can be subsidence of the DIT sewer itself. If the middle of this pipe segment was a few centimeters lower than both invert levels at the manholes, this means that the middle of the pipe is full, while both ends of the pipe were not. Consequently, in the last stage of pipe filling, there was less storage, leading to a higher rate of water level rise, assuming a constant infiltration volume. Another reason could be a shift in the water flow regime inside the pipe. If at one location the pipe was full due to a higher local drainage value, the flow could become pressurized instead of free flow and the water flow was behaving differently.

4.2 H.A.J.M Schaepmanstraat

4.2.1 Inundation test

In comparison with the Kuiperstraat, the inundation test in the H.A.J.M. Schaepmanstraat was much easier to perform. As described in chapter 3, the street has an ordinary layout with a high crown in the middle and sloping downwards to the gutters. More importantly, we choose the tested area to be enclosed by two speedbumps, so the construction of a temporary dam was not needed. Other than expected from the drawings, the crown of the north side of the tested street was higher than the curbs at the south side, leading to flooding of the adjacent parking lot. Unfortunately, the one monitoring well that was not in the testing area flooded too as a consequence. Therefore, no groundwater levels are available during this test.

In Figure 55, the inundated area is shown with the location of the divers, which are on the lowest locations of the tested area, and the location of the hand measurements, which were used to calibrate the diver data. The pavement turned out to be well permeable, so a constant head test could be performed. The street was flooded until the water height on the crown of at the north side of the street (the highest point) was at least 1 centimeter. When this level was constant, the discharge of the inflow was measured twice. Measurement 1 gave a value of 0,55 L/s and measurement 2 gave a value of 0,54 L/s. This corresponds with an infiltration value of 25,4 mm/h and 24,9 mm/h respectively.

After this test, the falling head method was applied. The raw data of this test, calibrated with the hand measurements, can be found in Appendix 4.

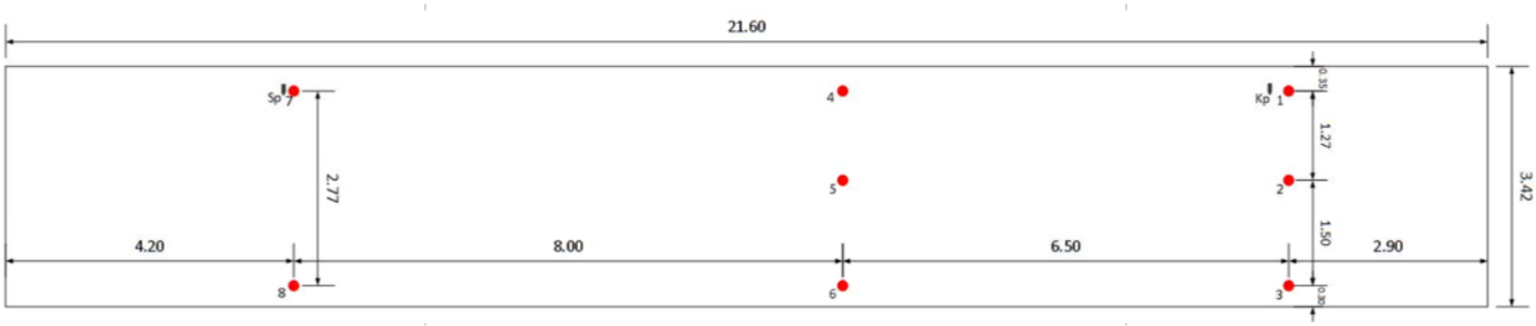

Figure 55 Inundated area in the Schaepmanstraat with locations of rulers for hand measurements (red dots) and the divers Sp and Kp

The data during the test showed noise, due to the before mentioned inaccuracy of the divers. To smoothen the line the moving average method with n=8 is applied. Figure 56 shows the infiltration during the test at the location of diver Kp. This graph shows a linear relationship between the dropping water level and time. The slope of this regression line, so the infiltration velocity, is 26,0 mm/h.

Figure 57 shows the infiltration during the test at the location of diver Sp. This graph shows also a linear relationship between dropping water level and time. The slope of this regression line, so the infiltration velocity, is 29,1 mm/h.

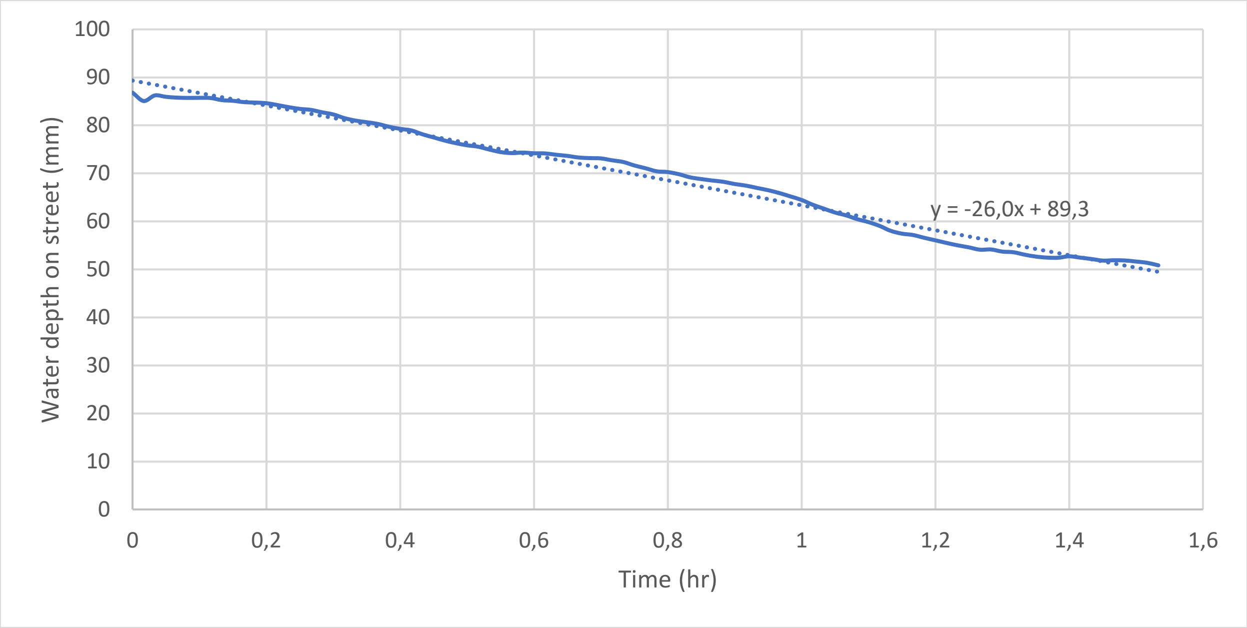

Figure 56 Water level drop in the Schaepmanstraat during the inundation test at the location of diver Kp with regression line

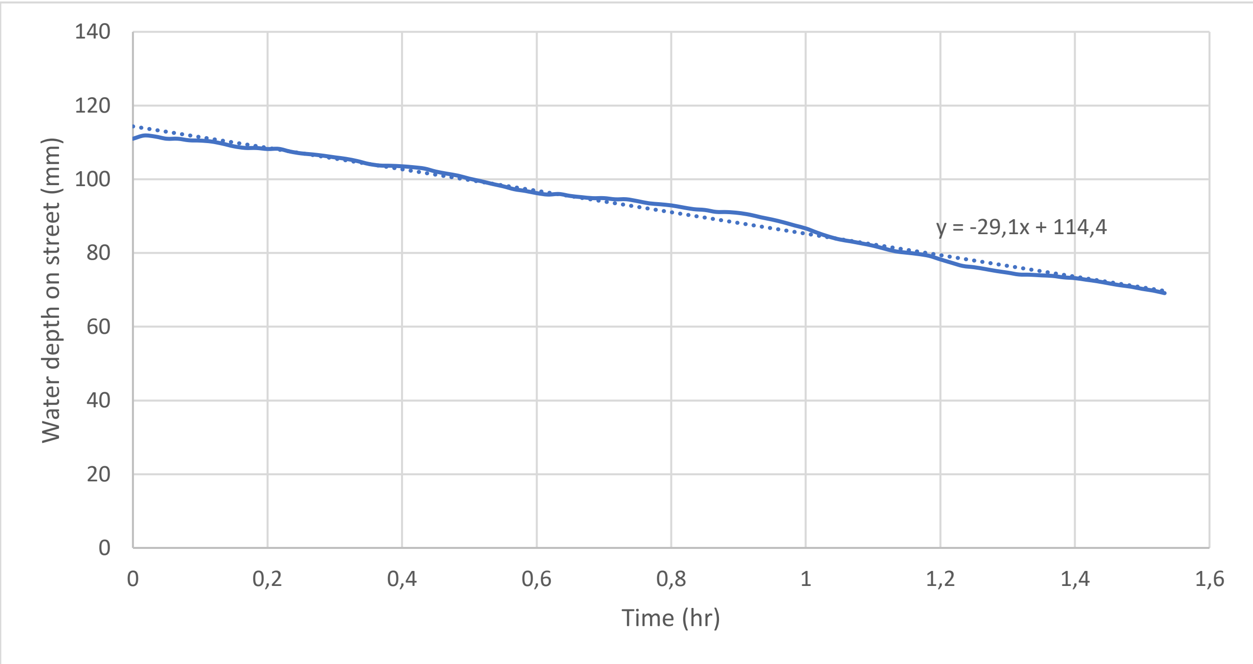

Figure 57 Water level drop in the Schaepmanstraat during the inundation test at the location of diver Sp with regression line

Discussion

The results from both the constant head as the falling head test show infiltration velocities in the same order of magnitude. The reason the value on location 7 is somewhat higher could be due to the higher water level on the street at this location, so the resulting higher pressure at the surface causes a higher infiltration rate.

These kinds of pavements do not have a required infiltration rate, mostly they are assumed to be impermeable. Commonly used permeable pavement guidelines in the Netherlands recommend that maintenance is undertaken on permeable pavements when the infiltration falls below 20,8 mm/h. This street is not documented as permeable pavement, however, this value is exceeded for this specific street. Spacers on the side of the BSS bricks give infiltration opportunity, and therefore a higher infiltration value as compared to "normal" bricks as in the Kuiperstraat.

4.2.2 Infiltration test

The infiltration test was performed on the pipe segment between manhole "Bestaand" (further referred to as manhole B) and D08, which has a length of 49 meters, corresponding to an infiltration area of 18,6 m^2, as described in chapter 3.

Test

Via manhole B, the pipe was filled and it turned out that water flew into the soil very rapidly. The filling lasted for almost 44 minutes until the water tank was empty. A constant head measurement was performed by filling a 100 L bucket. This gave an infiltration value of 2,4 L/s, corresponding with a k-value of 465 $L/m^2/h$ at dH = 0,11 m. The tank of the water truck has a volume of about 6600 L. This gives an infiltration value of 2,5 L/s, corresponding well with the constant head measurement. Figure 58 gives the water level in manhole B. The raw data can be found in Appendix 5. It is calibrated with hand measurements during the test, whereafter the line is smoothened with the moving average method (n is up to 8). The fluctuations were due to the rapid infiltration. Several times the pump was switched off or set on a lower pumping rate, so the water level drops up to 40 centimeters in a few minutes as the graph shows. Infiltration values of up to 0,82 L/s are observed during these drops.

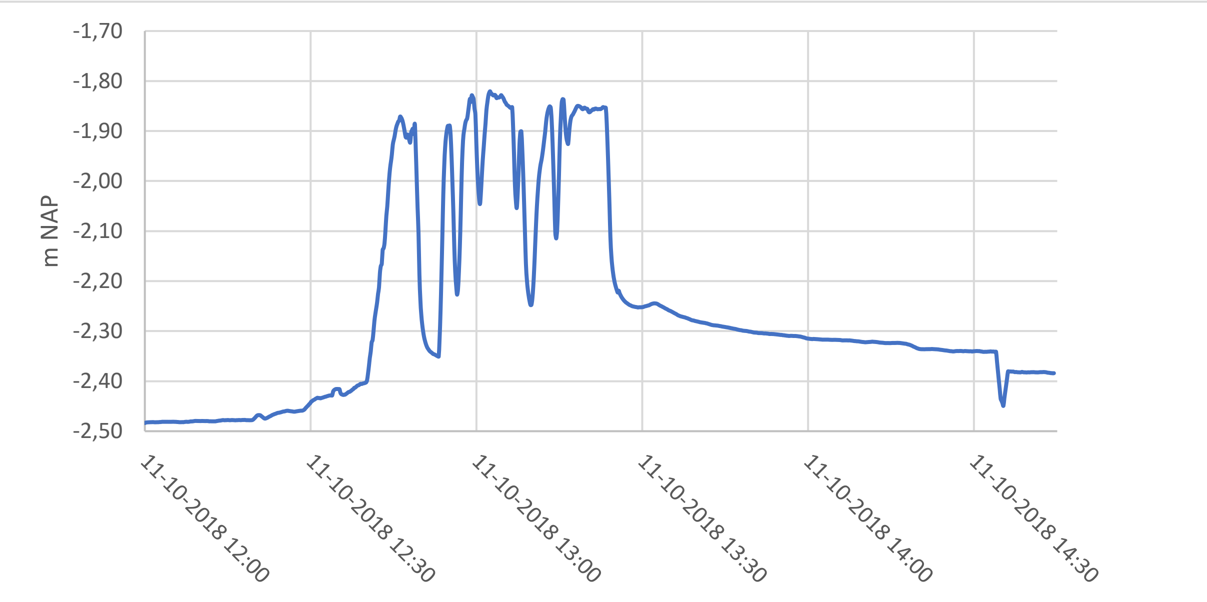

Figure 58 Water level development in manhole "Bestaand" during the infiltration test in the Schaepmanstraat

After the water tank was empty, the falling head measurement was performed at 13:23. The water level dropped until -2,25 m NAP at an infiltration rate of 0,62 L/s. Then an equilibrium was reached, after which the water level in the manhole dropped at a significantly slower rate. In Figure 59, the development of the cumulative infiltrated volume is shown from 13:32 until 14:26. The regression line shows in this stage an infiltration rate of 6,63 L/h. The infiltrated volume becomes less as the water level in the manhole approaches the groundwater level.

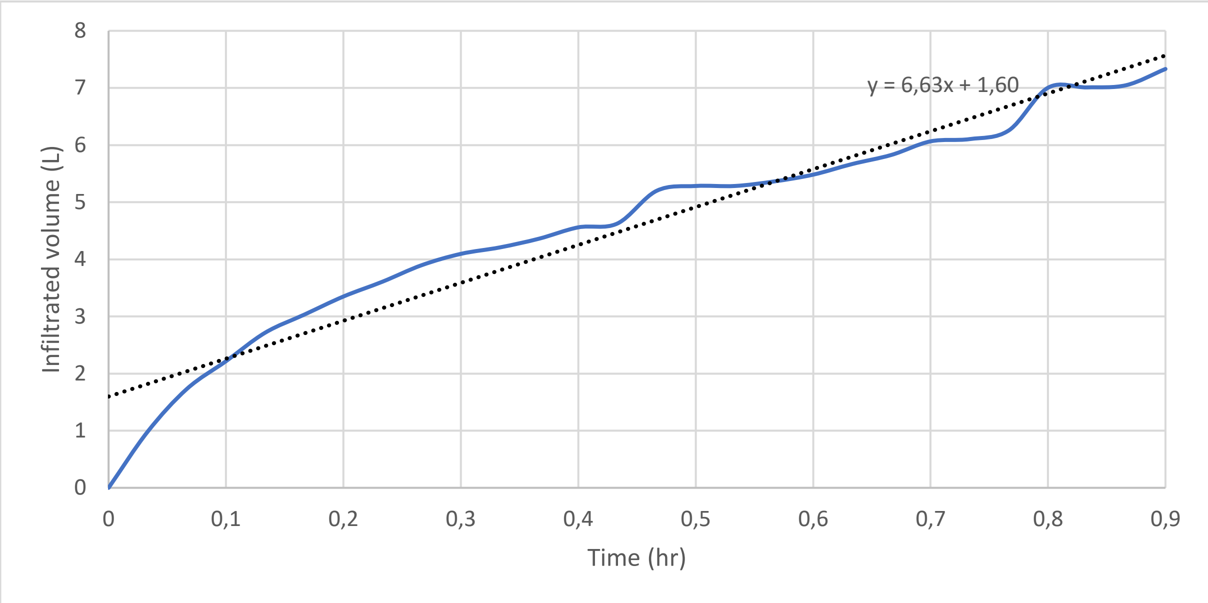

Figure 59 Cumulative infiltrated volume of water during the infiltration test in the Schaepmanstraat per time step after equilibrium point

Groundwater levels

The groundwater levels of row 1 in the time frame of the infiltration test are shown in Figure 60. Also shown, is the water level development in the sewer, as well as the location of the pipe and the ground level.

The water level in the manhole showed a similar development as monitoring well S_1.1. When the CH test was ended, the level immediately returned the value of the groundwater level of S_1.1. From this moment, it simply dropped together with this level until the original level was reached. The grey line also showed some minor drops during the CH test, which corresponded with the fluctuations in the manhole, as described above.

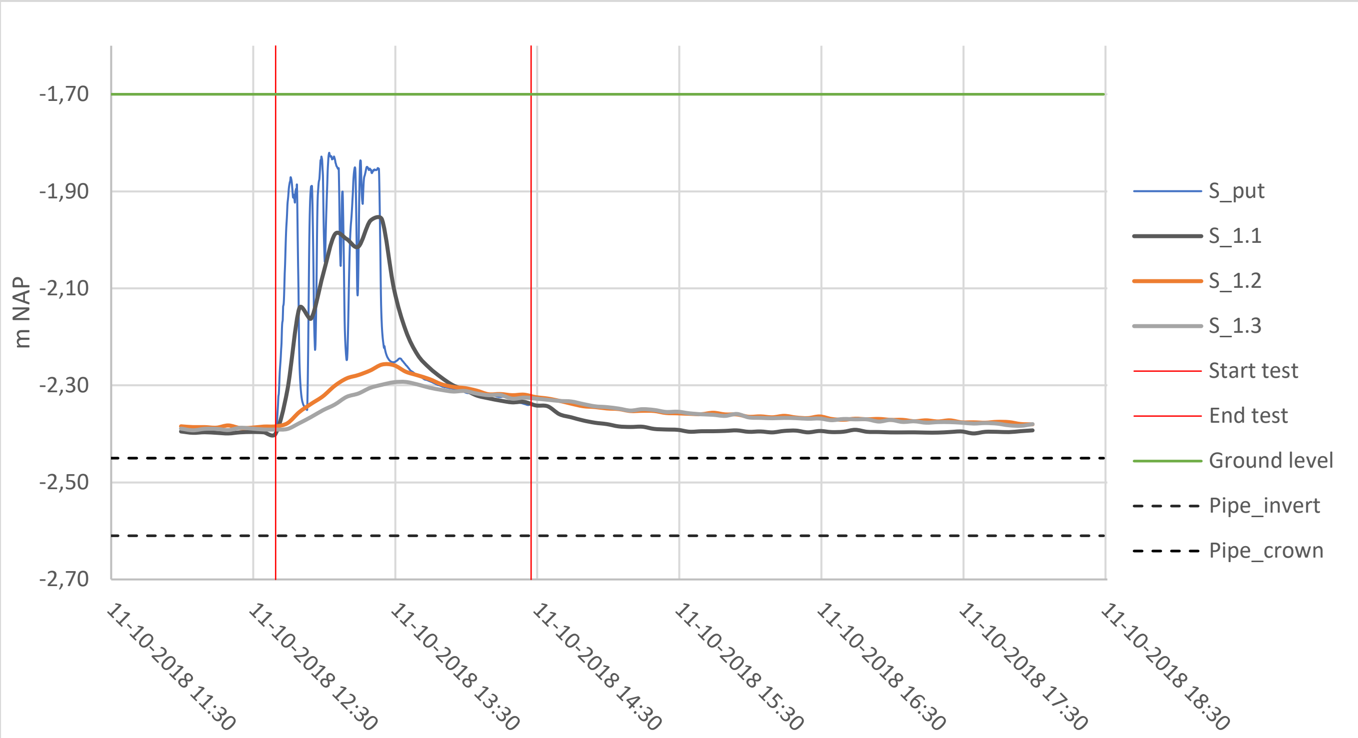

Figure 60 Infiltration test Schaepmanstraat with water level in manhole "Bestaand", groundwater levels in the monitoring wells S_1.1, S_1.2, S_1.3 with level of pipe and street level

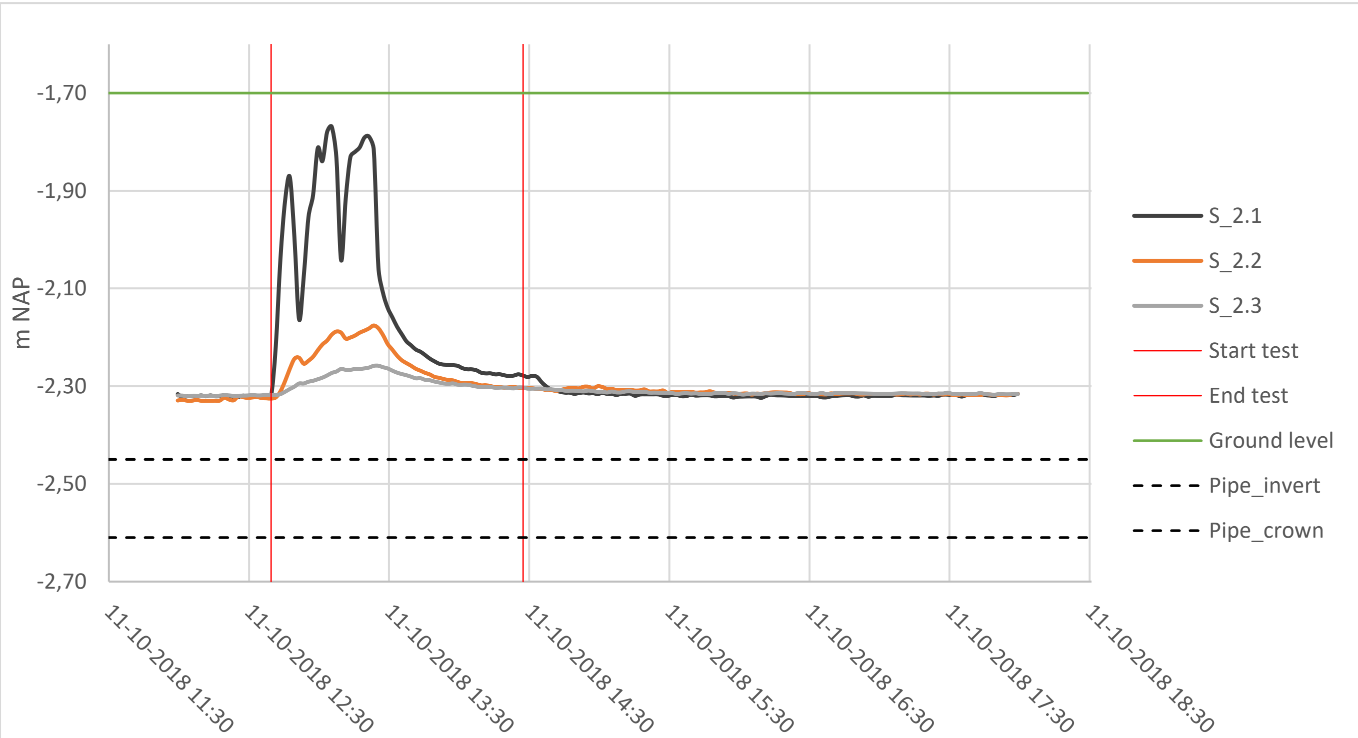

Figure 61 Infiltration test Schaepmanstraat with groundwater levels in the monitoring wells S_2.1, S_2.2, S_2.3 with level of pipe and street level

Monitoring well S_1.2 (orange line) responded to the test in the expected manner. The water level rose from -2,39 m NAP to -2,25 m NAP. When the constant head test ended, it returned to its original level. S_1.3 (light grey) reacted the same, except its maximum was logically lower, on -2,29 m NAP

In Figure 61, the groundwater levels of the second row of monitoring wells during and after the infiltration test are presented. The initial groundwater level at this location was higher than at the first location, namely at -2,31 m NAP. This corresponded with the monitoring well of Wareco, which is located about 12 meters away from these monitoring wells. The reaction of the groundwater level at the Wareco monitoring well and its location can be found in Appendix 5. At the time of the infiltration test, a peak of about 2 centimeters can be seen in the groundwater level of this monitoring well.

Monitoring well S_2.1 follows very clearly the path of the water level in manhole B. The two distinct dips during the test were on exactly the same moment as the dips in the water level in the drain. It is curious why S_2.1, which is located on about 42 meters of the inlet manhole, reacted better than S_1.1, which is on 12 meters of the inlet manhole. One reason can be a higher permeability of the soil at S_2.1, so the main flow was directed to this location. Another reason can be that monitoring well S_1.1 was less perceptive to sudden changes (< 1 min) in the groundwater level compared to S_2.1, so S_1.1 didn't display the real groundwater level fluctuations at that moment. Noted is that the maximum value of S_2.1 displayed in the graph is the top of the monitoring well, which could also be observed by the wet sand surrounding the well pipe. So, the groundwater level could be even higher and have reached the street level. When the FH test was started, the groundwater level next to the drain dropped immediately to -2,28 m NAP within half an hour after which it returned to its original value.

Monitoring well S_2.2 had a weaker reaction to the infiltration test but responded in the expected manner. The water level rose to -2,17 m NAP after which it almost directly after the end of the CH test returned to its original level. Monitoring well S_2.3 reacted in the expected manner and rose until -2,25 m NAP after which it immediately returned to its original level after the end of the CH test.

It can be concluded that the drain in the H.A.J.M. Schaepmanstraat fulfills its infiltration purpose. When the pressure difference between the water level in manhole B and the groundwater level was high, the infiltration value was also high. This indicates a high permeability of the surrounding soil. The flow in the soil was also high which can be concluded from the groundwater level developments after the CH test. Groundwater levels next to the drain, and on some distance of the drain almost immediately dropped to the original levels, indicating high groundwater velocities and high soil permeability. This is also the reason for the drop in infiltration rate after the end of the CH test. The pressure difference between the water level in manhole B and the groundwater level dropped in a matter of minutes, so the infiltration rate became also smaller.

Discussion

The sewer segment was isolated without incorporation of manhole D08. Since a PK- manhole isn't constructed until street level, but the height is at a random level under the street, the height in manhole D08 was lower as in manhole B. Therefore it would overflow, when the infiltration test was performed in manhole B. Performing the test in manhole B is preferred, since at this location the highest simulated surface water level could be achieved.

Because of the fluctuations during the test and rapid discharge of the water in the soil, it is hard to assign a distinct k-value to this test. An average pressure difference between the groundwater level and water level in the pipe can be taken from the last 15 minutes of the falling head test. During this time both levels were constant, namely -1,96 m NAP at S_1.1 and -1,85 m NAP in the manhole. Together with the before mentioned infiltration rate of 2,4 L/s, the k-value becomes 465 L/m^2/h at dH = 0,11 m. When the test was ended, the groundwater level and the water level in the pipe approached each other very fast and the pressure difference was about 4 millimeters. Consequently, the infiltration rate dropped two orders of magnitude.

For the values of k, the first row of monitoring wells was used since the groundwater level here is comparable to the values at the measuring location. The second row of monitoring wells showed a higher groundwater level because it is further away from the surface water level and has, therefore, a higher steady-state groundwater level midway the two surface water bodies. This hydraulic phenomenon influences also the rate infiltration of groundwater at each part of the drainage pipe. At the second row of monitoring wells, the unsaturated zone height was less compared to the first row. If the entrance resistance of the pipe is the same at each location, the same amount of water enters the soil and groundwater levels at the second row become higher and reach the street level. This was also noticed at monitoring well S_2.1, which overflowed during the infiltration test.

The difference in groundwater level along the tested pipe segment also means a difference in the pressure difference dH along the pipe. Infiltration values at different points in the pipe are consequently not the same and therefore an average for the whole pipe.

4.2.3 Drainage test

The infiltration test was performed on the same pipe segment as the infiltration test. The difference is that the pipe was emptied from manhole D08 and the diver was placed in manhole B, due to practical reasons at that moment.

Test

The sewer segment was isolated with the incorporation of both manholes. The manholes and drain pipe were emptied. From the infiltration test is already concluded that the drain is capable of high infiltration rates. This turned out to be not different for the drainage rates. It was impossible to completely empty the pipe and manhole B, with the pumping capacity available. This is can also be seen in the graph in Appendix 6. The initial water level drops from -2,39 m NAP to -2,50 NAP, which is not the pipe invert level. Since at this point we were just pumping out drained groundwater, the pump was switched off and the falling head test was started at 11:49. The same graph in Appendix 6 shows a rise in the water level of 7 centimeters without 10 seconds, indicating high drainage velocities. Figure 62 shows the development of

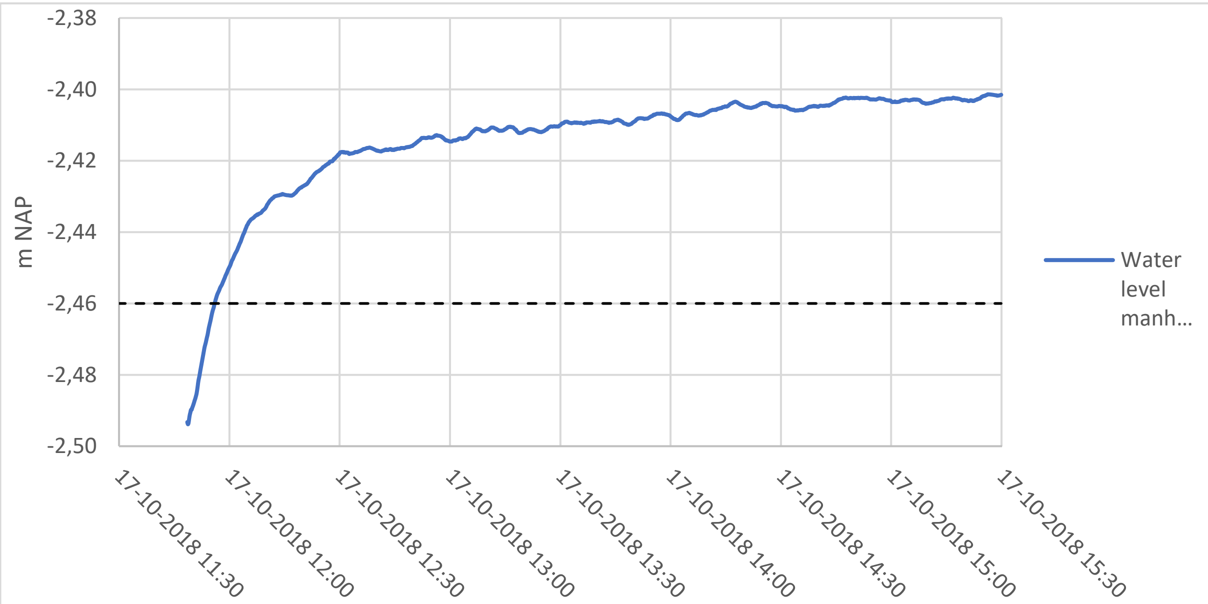

Figure 62 Water level in manhole "Bestaand" during the drainage test in the Schaepmanstraat

the water level in manhole B during this test. The raw data can be found in Appendix 6. This data is edited with the moving average method (n=15) to the data in the figure below.

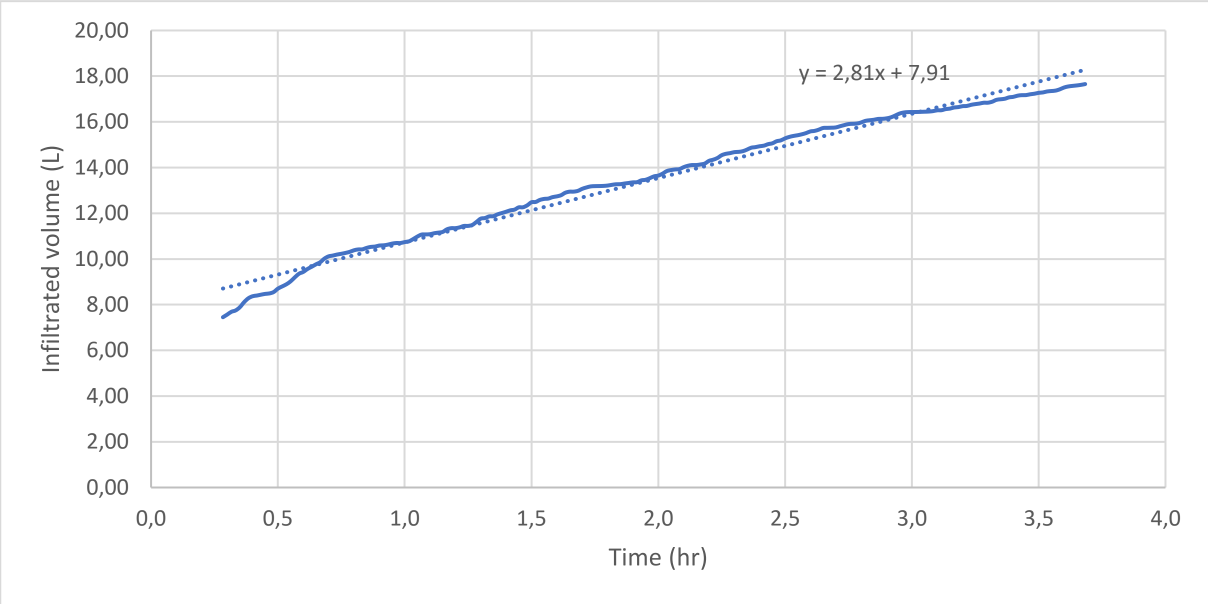

Figure 63 Cumulative water volume inflow into drain pipe during and after drainage test in the Schaepmanstraat with regression line

In Appendix 6, the graph with the water level translated to the drained volume can be found. Since only the manhole filling is incorporated, this graph follows the same path as the water level. In Figure 63, the linear filling of manhole B is shown, which starts at approximately 12:06. At this time, the pipe is full for sure and only the manholes are filling. An estimation of the drainage rate is 2,81 L/h.

Groundwater levels

The groundwater levels in the time frame of the drainage test are shown in Figure 64 and Figure 65. Also shown, is the water level development in the sewer, as well as the location of the pipe crown.

In Figure 64, the reactions of the groundwater levels of the first row of monitoring wells can be found. The initial level was at all locations at -2,39 m NAP. S_1.1, which was placed next to the drain pipe, has logically the strongest reaction. Immediately after the start of the test at 11:40, the groundwater level dropped to -2,44 m NAP, indicating inflow into the drain pipe. During the test, the groundwater level restored itself and was at -2,40 m NAP at 16:30 on the same day. This level was constant until the balloons were removed on 19/10 at 14:40. Then it restored fully to the groundwater level of -2,40 m NAP. This was also the level of monitoring wells S_1.2 and S_1.3 at that time.

Monitoring wells S_1.2 and S_1.3 reacted in the same manner. They both dropped from -2,39 m NAP to -2,40 m NAP during the test. They remained on this level during the time shown in the graph. Measuring data is available until 25/10/2018, showing fluctuations of the groundwater level between -2,39 and -2,41 m NAP for the entire period.

In Figure 65, the reactions of the groundwater levels of the second row of monitoring wells can be found. The initial level was at all locations at -2,32 m NAP. S_2.1, which was placed next to the drain pipe, has, as in the first row, the strongest reaction. The groundwater level dropped at the start of the test to -2,39 m NAP. This is a higher drop than S_1.1, which can have two reasons. The difference between the pipe crown and the initial groundwater level was higher, so the potential was higher and more water was forced to flow in. Another reason can be the before mentioned emptying of the pipe. At the side of manhole B, the pipe was not completely empty, while at the side of manhole D08 the pipe was emptied. Consequently, to this presumed water gradient in the pipe, there was more flow into the drain at this side than at the other end. During the test, the groundwater level of S_2.1 restored to a stable level of -2,33 m NAP at 15:30. In accordance with S_1.1, this level was constant until the balloons were removed on 19/10 at 14:40. Then it restored fully to the initial water level of -2,39 m NAP.

Monitoring well S_2.2 dropped to -2,34 m NAP during the test and restored to -2,33 m NAP after which it fluctuated between -2,33 and -2,34 m NAP during the time period till the removal of the balloons. S_2.3 dropped to -2,33 m NAP and reacted further the same as S_2.3.

From the reaction of both rows of monitoring wells, two characteristics stand out. Firstly, both S_1.1 and S_2.1 did not restore to the original groundwater level and gave a sudden rise of 1 cm after the balloons were removed. This could be because the Argex aggregates surrounding the drain were less capable of holding the water giving a local dip along the drain. Since it rises after removal of the balloons, the system acts as a whole again after which this phenomenon is equilibrated out.

Secondly, all monitoring wells did not give a restoration of the initial groundwater level during the measuring period. Measurements, taken until 25/10/2018 14:30 show fluctuations between given intervals, where the groundwater level returned to the initial levels from time to time. Probably it took some time (days) to supply water from the surrounding soil since quite a lot of water is drained during the test.

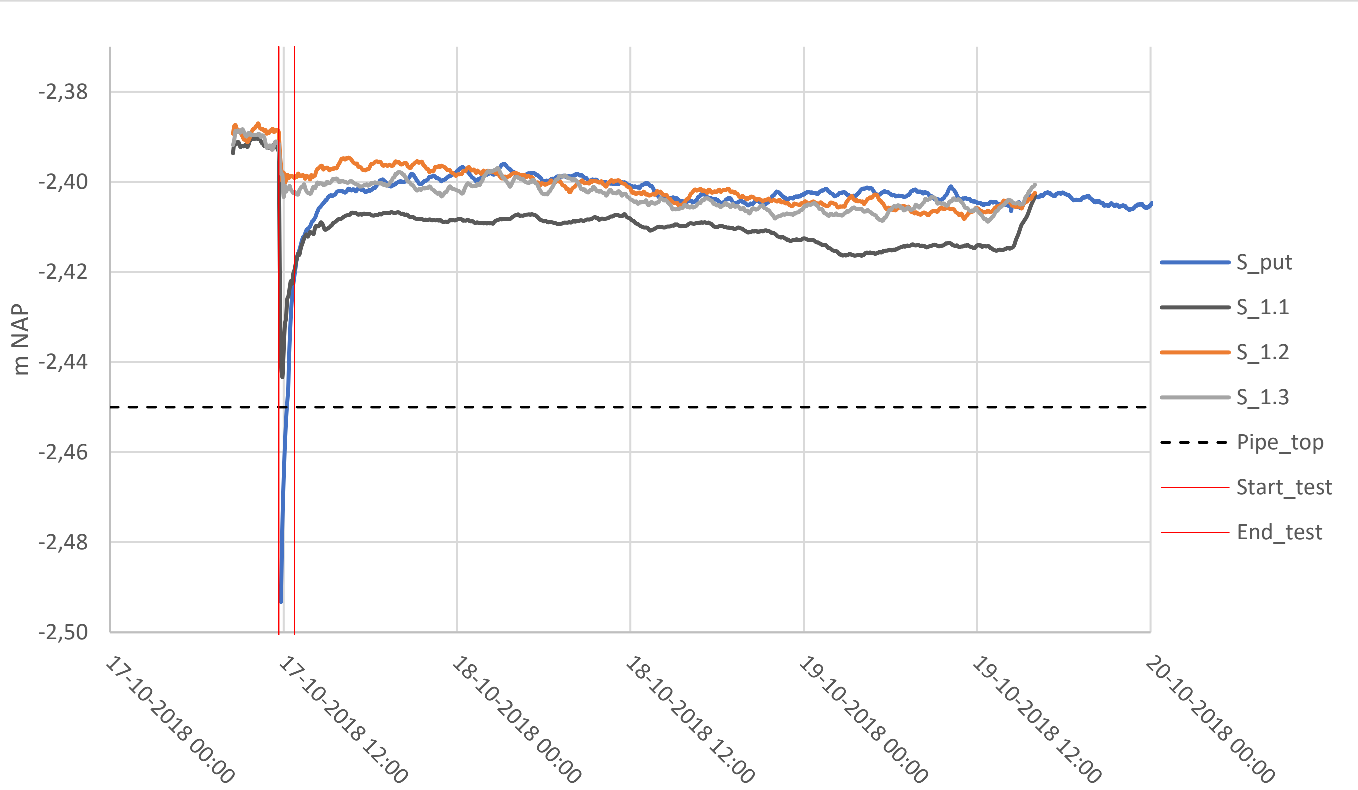

Figure 64 Drainage test Schaepmanstraat with water level in manhole "Bestaand", groundwater levels in the monitoring wells S_1.1, S_1.2 and S_1.3 with location of pipe crown

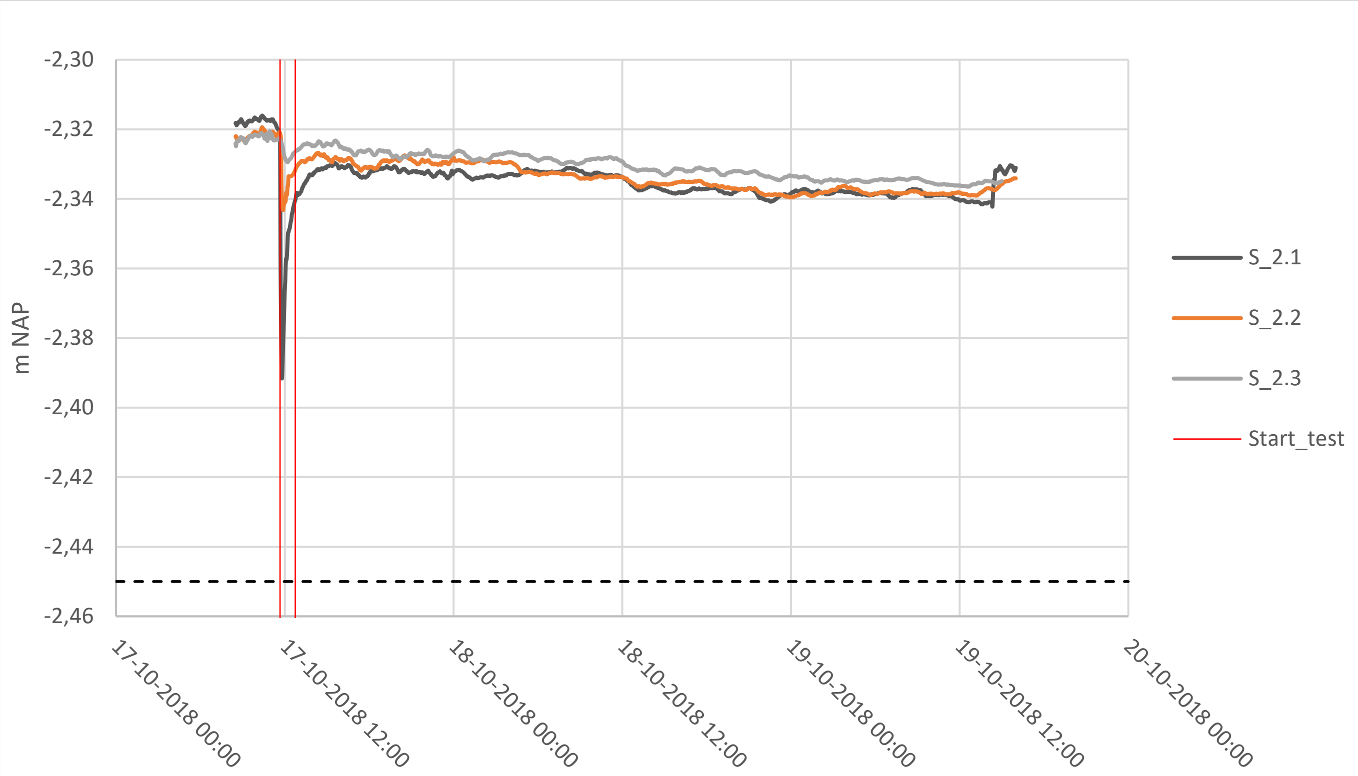

Figure 65 Drainage test Schaepmanstraat with groundwater levels in the monitoring wells S_2.1, S_2.2 and S_2.3 with location of pipe crown

Discussion

The first point of discussion is the emptying of the drain pipe. At the sucking side, the pipe was completely emptied, but the data on the other side showed a water level of -2,50 m NAP, while the invert level of the pipe is at -2,61 m NAP. This suggests a gradient between the two manholes and, consequently, different drainage rates along the pipe. Since the infiltration test showed a fast infiltration rate, there is no reason to believe this is different for this test. When emptying the system, it took long before there was no water flowing into the sucking device anymore. Therefore, the test was ended since we were probably sucking up inflowing groundwater. The data showed an incomplete emptying of the pipe at manhole B, so obtaining a distinct drainage capacity from this test is hard.

The groundwater level goes up very quickly but did not reach the initial groundwater level. Especially, S_1.1 and S_2.1 stayed 1 cm under the groundwater levels of the other monitoring wells. After taking out the plugs, S_1.1 and S_2.1 joined with the other levels. This indicates a water inflow into this area. It is curious where this inflow comes from since the water level in the pipe does not change at that moment.

Since the groundwater levels at S_1.1 and S_2.1 were lower than the water level in the pipe, there is theoretically no inflow possible. Calculating the k-value for this test is not possible since it would be a positive value, which indicates infiltration.

Looking at the groundwater levels after the test, which were measured until 24/10/2018, it shows fluctuations between -2,41 and 2,40 m NAP for the first row of monitoring wells, but do not return to the initial water levels. The same can be noticed in the other row of monitoring wells. Appendix 7 shows the groundwater levels of S_1.2 and S_2.2 where this process can be observed. This process is also observable in the Wareco monitoring well shown in Appendix 5. So it seems that either the total area is affected by the withdrawal of water from the pipe during the test, or the entire area is affected by another cause, which can be weather conditions, a change in surface water level or otherwise. It is however not clear what happened during this period. Since the groundwater level in the whole area has changed compared to the period before the test, it is hard to contribute this change to the test or another cause.

4.3 Comparison Kuiperstraat and H.A.J.M. Schaepmanstraat

Each experiment is performed on the same day in both the Kuiperstraat and the Schaepmanstraat, so the circumstances were the same. This gives a possibility to compare the two locations without a significant difference in weather conditions to account for. This comparison is done for all three tests.

Inundation test

The pavements and layout of both streets were different. Both were paved according to a herringbone pattern with the points in the latitudinal direction of the street. The pavement of the Schaepmanstraat consists of BSS paving bricks which have spacers on the side, giving more infiltration opportunity. The Kuiperstraat paving bricks have no spacers, so less infiltration is possible. This is also visible in the infiltration value estimations, which are 3,8 mm/h and 29 mm/h for the Kuiperstraat and Schaepmanstraat respectively. The Kuiperstraat had a higher runoff relative to the precipitation in case of rainfall than the Schaepmanstraat, so transporting this runoff to the soil via gully pots and a DIT sewer, or to the surface water via this sewer in the Kuiperstraat is a proper solution.

Infiltration test

Values for infiltration in the Kuiperstraat and Schaepmanstraat are not easy to compare because of the different development of the water level in both tests and other dimensions of the drain pipes themselves. In the Kuiperstraat, the rate of filling of the pipe is two orders of magnitude higher compared to the Schaepmanstraat, while in the Schaepmanstraat the water level in the manhole approached the groundwater levels way faster. It is clear from both tests, that the groundwater velocities in the Kuiperstraat were slower than in the Schaepmanstraat. This is a result of a different composition of the soil beneath the pavement. The Kuiperstraat has polluted silty soil and the municipality choose to put the same soil after the construction of the street and sewer pipes rather than replace it with clean sandy soil. A drainage advice report for the sewer reconstruction in 2015, performed by Lankelma and commissioned by Royal Haskoning, characterized the soil as moderately fine, silty sand with a k-value of 2,5 m/d until a depth of -1,5 m NAP. From the drawings, it is not clear what soil is beneath the pavement in the Schaepmanstraat, but from conversations with construction workers which had worked in that area became clear that about two meters of pumice (Dutch: bims or puimsteen) were used beneath the street. This material is characterized by its high permeability, which supports the outcomes of the infiltration test in the Schaepmanstraat. The lower values in the Kuiperstraat are the result of the soil, which has a lower permeability due to already inflicted compaction of the soil over the years before the construction of the street.

Drainage test

What is concluded above in the infiltration test is not different in this test. The compacted soil in the Kuiperstraat results in a lower drainage value compared to the permeable soil of the Schaepmanstraat.

5 Modeling with Hydrus2D

This chapter gives the settings and results of the simulations of the performed tests in Hydrus2D. The first two paragraphs give the outcomes of the Kuiperstraat with the infiltration test and drainage test respectively. Paragraph 5.3 and 5.4 give the outcomes of the Schaepmanstraat with the infiltration test and drainage tests respectively. After each result, the results are substantively discussed.

5.1 Kuiperstraat - Infiltration test

5.1.1 Model structure

Geometry

A 2D- general geometry domain in the x-z plane, is chosen to represent a cross-section of the Kuiperstraat. Since the conditions on both sides of the drain are the same, it is chosen to only model one side. Consequently, only half of the drain is represented in the domain and is modeled as an irrigation dripper with a radius of 12,5 cm. A plot was made according to the cross-section of the Kuiperstraat, with a width of 4,27 meters and a height of 1,86 meters, with the bottom boundary at – 2 m NAP. The drain is placed in the middle of an envelope of Argex clay granules of 40x40 cm.

Soil hydraulic parameters

As a process, only the groundwater flow is relevant for this simulation. Units of time are in minutes with a time discretization from 0 to 480 min, with an initial time step of 5 seconds and a minimum timestep of 0,5 seconds.

Soil hydraulic parameters are firstly chosen according to the before mentioned drainage advice report of Lankelma and are shown in Table 1. Values for the drain envelope are first chosen as sand. For this simulation, only the saturated volumetric water content θ_s and the saturated hydraulic conductivity K_s are valid.

	θ_s [-]	K_s [cm/min]
Soil	0,43	0,09
Envelope	0,43	0,50

Table 1 Initial theoretical water flow parameters

FE Mesh

To create a Finite Element Mesh, for each boundary a targeted FE size has to be chosen. It is chosen to dense the finite element sizes closer to the drain since this is the domain of interest and larger changes are expected over short distances. The targeted FE size at the drain boundary was set on 1,5 cm, at the edge of the drain envelope on 2,5 cm and every other boundary on 5 cm. The resulting FE mesh is shown in Appendix 8.

Domain properties

The material distribution is shown in Appendix 8. The drain envelope is light blue and the soil is dark blue. An anisotropy is added on the soil only. The anisotropy is defined as $K_{s,v}/K_{s,h}$ and estimated as 1/5. Observation nodes are added on the initial groundwater table at the location of the monitoring wells K_1.1 and K_2.2 and are also visible in Appendix 8.

Conditions

To let the model run, both the initial conditions (on each defined planar surface) as the boundary conditions (on every defined boundary) have to be given. The initial condition is defined as the bottom pressure head from the lowest located nodal point. For the infiltration test, this implies a value for h of 129 cm corresponding with a groundwater table of -0,71 m NAP.

Boundary conditions are defined as follows: the left boundary is a constant head boundary with the value of the initial condition. The (half) drain has a variable flux boundary and the infiltration fluxes found in the performed test are used as input. This flux is defined as the total infiltration capacity of the drain pipe divided by its infiltration surface (perimeter x length). These time-variable boundary conditions are shown in Table 2. The first test starts at minute 22, corresponding with 10/10/2018 08:52, the second test at minute 112. All the other boundaries have no flux.

Time [min]	Variable flux [cm/min]
22	0
79	-0,029
85	-0,02
91	-0,015
97	-0,01
101	-0,005
112	0
174	-0,026
178	-0,023
184	-0,02
190	-0,017
196	-0,013
202	-0,009
208	-0,007
214	-0,005
220	-0,003
480	0

Table 2 Time variable fluxes infiltration test, Kuiperstraat, for the chosen modeled period

5.1.2 Model optimization

After running the model with these fluxes, there are basically three parameters to optimize for the best fit with the results of the performed tests. These parameters are θ_s, K_s and the anisotropy ($K_{s,v}/K_{s,h}$). Hydrus offers for the water flow parameters an inverse solution option. The boundary values for the parameters θ_s and K_s are defined, and Hydrus runs an iteration process to fit the results. The solution of this process is not necessarily the best solution but gives a good indication for these parameters.

With the found parameters, further optimization is done with trial and error. The final parameters for this model are shown in Table 3.

	θ_s [-]	K_s [cm/min]	$K_{s,v}/K_{s,h}$
Soil	0,28	0,17	1/5
Envelope	0,38	0,20	1/1

Table 3 Water flow parameters and anisotropy used in the final simulation of the infiltration test, Kuiperstraat

5.1.3 Model results and discussion

In Figure 66 and Figure 67, the model outcome is plotted against the data obtained from the infiltration test for a period of 480 minutes.

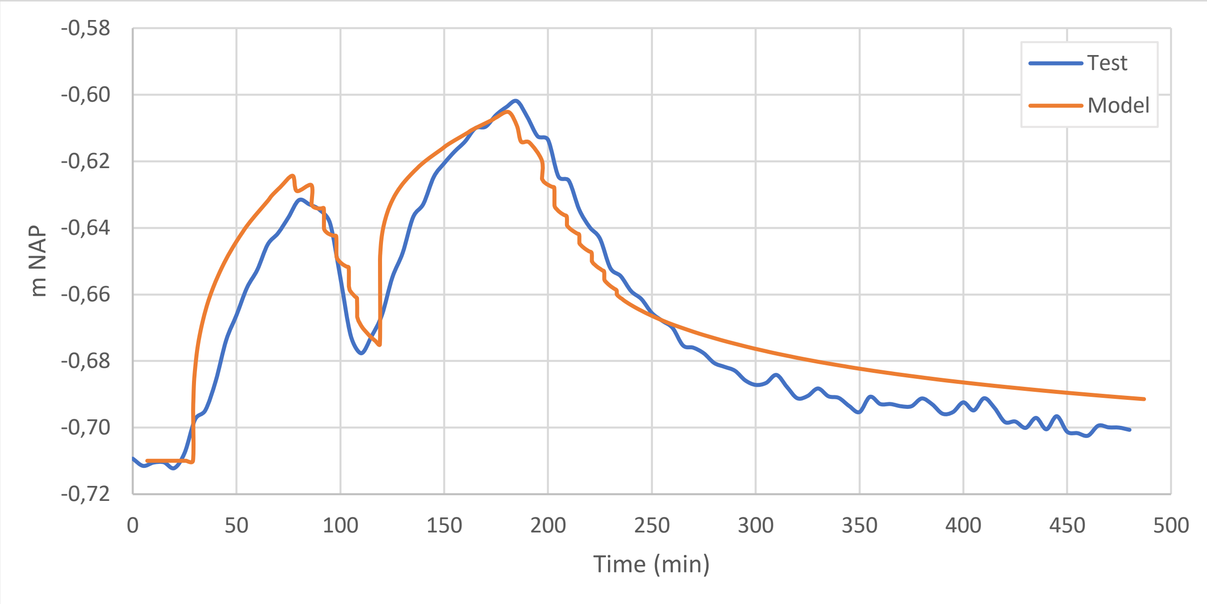

Figure 66 Model results versus test results of monitoring well K_1.1 of the infiltration test Kuiperstraat

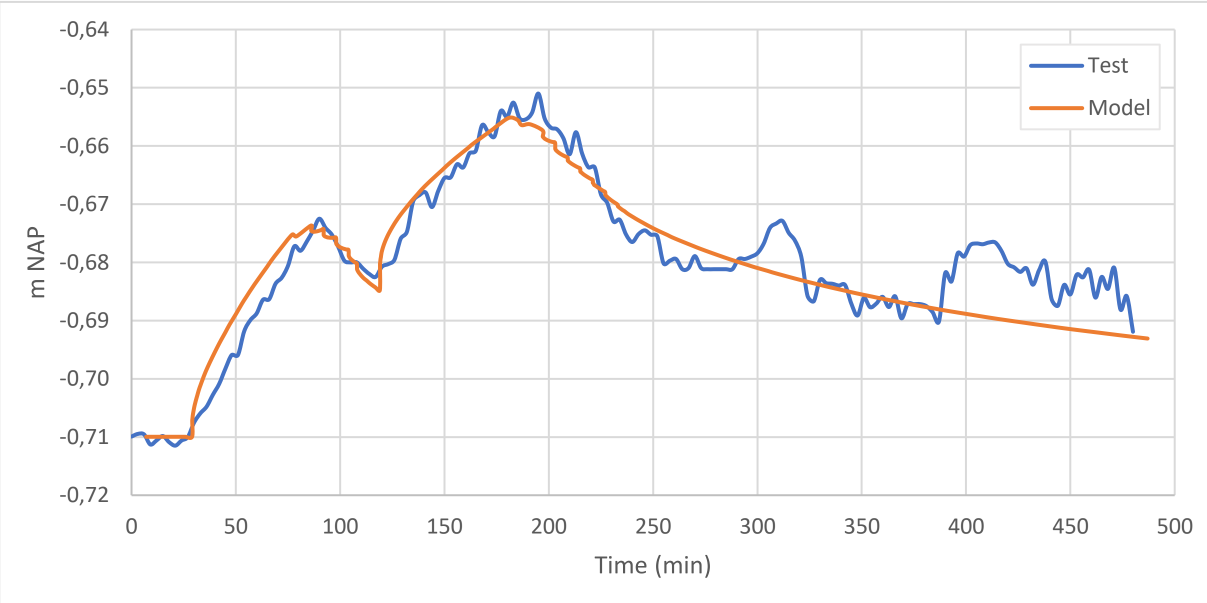

Figure 67 Model results versus test results of monitoring well K_2.2 of the infiltration test Kuiperstraat

When analyzing the model results of K_1.1, there are some misfits noticeable. Firstly, the groundwater level of the model rises faster than the groundwater level in both tests initially. Secondly, the first peak is slightly overestimated, with 0,9 cm. And lastly, from minute 260 the groundwater level drop is underestimated, with a difference in groundwater level at minute 480 of 1 cm.

Analyzing the model results of K_2.2, it can be noticed that the model fits better than K_1.1. The peaks are at the same level and time and the groundwater level at 480 minutes are at the same level. The short-time fluctuations in the groundwater level of this monitoring well cannot be modeled and chosen is to model the bottom values of this test.

Discussion

The major point of discussion is the locations of the monitoring wells and the observation points in the model. The model assumes K_1.1 and K_2.2 in one row, while ideally K_1.1 and K_1.2 (or K_2.1 and K_2.2) are to be modeled. Since the monitoring wells are not located in one row, it is questionable if they show the real results. Therefore, it is, for instance, possible that the mentioned difference in the first peak in K_1.1 is the actual result of monitoring well K_2.1. Another reason for the peak being too high is an overestimation of the flux of the first test.

It is chosen to add average fluxes for both tests. In this manner, the same amount of water is added in the timespan of both tests. If fluxes are added in time steps of 5 minutes, the first fluxes would be higher than the average flux, leading to high initial groundwater levels for this first period. This is presumably caused by the drain envelope of Argex granules, which buffers or drains the first high flux.

Some curve fitting is done in this test. To fit the peaks and drops in the data, a time shift of 7 minutes is applied. In paragraph 4.1.2 was mentioned that the monitoring wells responded 7 minutes after the test was started. Since the modeling showed also a difference of this exact 7 minutes the field data is thought to be slightly compromised and a time shift of 7 minutes seems legit.

Fluxes after the peaks are not measured but exist nonetheless. They are, therefore, estimated by fitting them into the test results. The peaks and drops in the model are slightly out of sync with the data by approximately 7 minutes. In paragraph 4.1.2 was mentioned that the monitoring wells responded 7 minutes after the test was started. Since the modeling showed also a difference of this exact 7 minutes the field data is thought to be slightly compromised and a time shift of 7 minutes seems legit. The location of the monitoring wells can also be the cause, as described above. Another reason for the peak being reached too early is the operation of the Argex drain envelope. The model directs the flow only in 2D, while in reality also 3D effects can be present, for instance, a discharge along the pipe in the Argex casing. Then, the flux into the soil is not equal to the flux measured in the test. This would cause an overestimation of the current soil hydraulic parameters, leading to higher and out of sync peaks in this model result.

Initially, the period after the two tests was not modeled well. The groundwater level did not drop to the initial level. To counter this, an anisotropy was introduced. With anisotropy of 1/5, the model fitted better. A higher ratio of anisotropy did not improve the results significantly. The cause of this anisotropy can be found in the presence of silt layers in the soil, which is backed up by the research in the before mentioned report of Lankelma. These silt layers cause a lower hydraulic conductivity in the vertical direction compared to the horizontal direction. The results of the model simulations can be found in Appendix 9.

The parameters of the drain envelope are not changed from the outcomes of the inverse solution. It turned out that the model outcome was less sensitive to these parameters compared to changes in the parameters of the soil.

5.2 Kuiperstraat - Drainage test

5.2.1 Model structure

Geometry

The same 2D – geometry domain as used in the former simulation of the infiltration test is used in this simulation.

Soil hydraulic parameters

Also, the only process modeled is water flow. Units of time are in hours with a time discretization from 0 to 23 hours, with an initial time step of 1 minute and a minimum timestep of 6 seconds. The first 23 hours were chosen since the data from the performed drainage test was steady and most trustworthy until that time.

The soil hydraulic parameters were firstly set on the found values in the simulation of the infiltration test.

FE Mesh

The FE mesh was the same as in the simulation of the infiltration test.

Domain properties

The material distribution is the same as in the simulation of the infiltration test. Also, anisotropy is added on the soil only. Observation nodes are added on the initial groundwater table at the location of the monitoring wells K_1.1 and K_2.2. Although monitoring wells K_1.2 and K_2.1 gave more reasonable results than in the infiltration test, it is chosen to work with the same monitoring wells as in the infiltration test to make the comparison more reliable. Also, K_2.1 gave still higher groundwater levels as K_2.2, which does not make sense.

Conditions

To let the model run both the initial conditions (on each defined planar surface) as the boundary conditions (on every defined boundary) have to be given. The initial condition is defined as the bottom pressure head from the lowest located nodal point. For the drainage test, this implies a value of 126 cm corresponding with a groundwater table of -0,74 m NAP.

Boundary conditions are defined as follows: the left boundary is a constant head boundary with the value of the initial condition. The (half) drain has a variable flux boundary and the drainage capacity found in the performed test are used as input. These time-variable boundary conditions are shown in Table 4. The test starts at minute 25, corresponding with 17/10/2018 09:25. All the other boundaries have no flux.

Time [min]	Variable flux [cm/min]
25	0
145	0,0123
265	0,0121
385	0,013
505	0,0127
625	0,0117
745	0,0109
865	0,0107
985	0,0123
1105	0,0113
1225	0,0102
1345	0,0109
1405	0,0122

Table 4 Time variable fluxes drainage test, Kuiperstraat, for the chosen modeled period

5.2.2 Model optimization

The model is optimized by performing the inverse solution option. The found values for the parameters are θ_s, K_s and the anisotropy ($K_{s,v}/K_{s,h}$). Further optimization is done by trial and error.

The final parameters for this model are shown in Table 5.

	θ_s [-]	K_s [cm/min]	$K_{s,v}/K_{s,h}$
Soil	0,11	0,12	1/5
Envelope	0,60	2,0	1/1

Table 5 Water flow parameters and anisotropy used in the final simulation of the drainage test, Kuiperstraat

5.3.3 Model results and discussion

In Figure 68 and Figure 69, the model outcome is plotted against the data obtained from the infiltration test for a period of 23 hours.

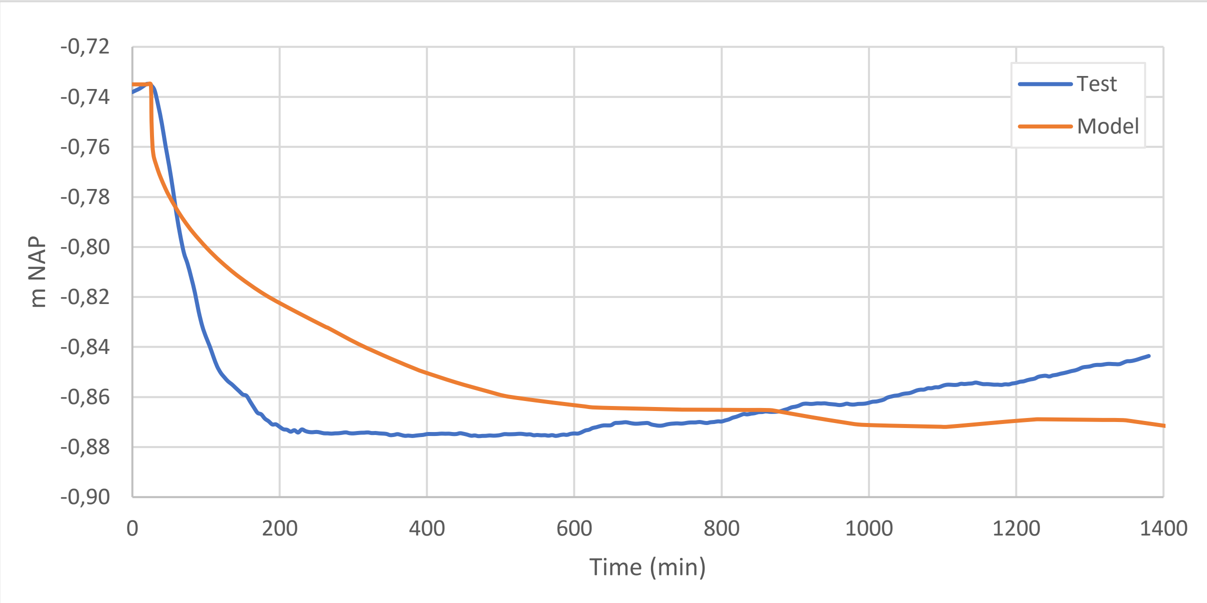

Figure 68 Model results versus test results of monitoring well K_1.1 of the drainage test Kuiperstraat

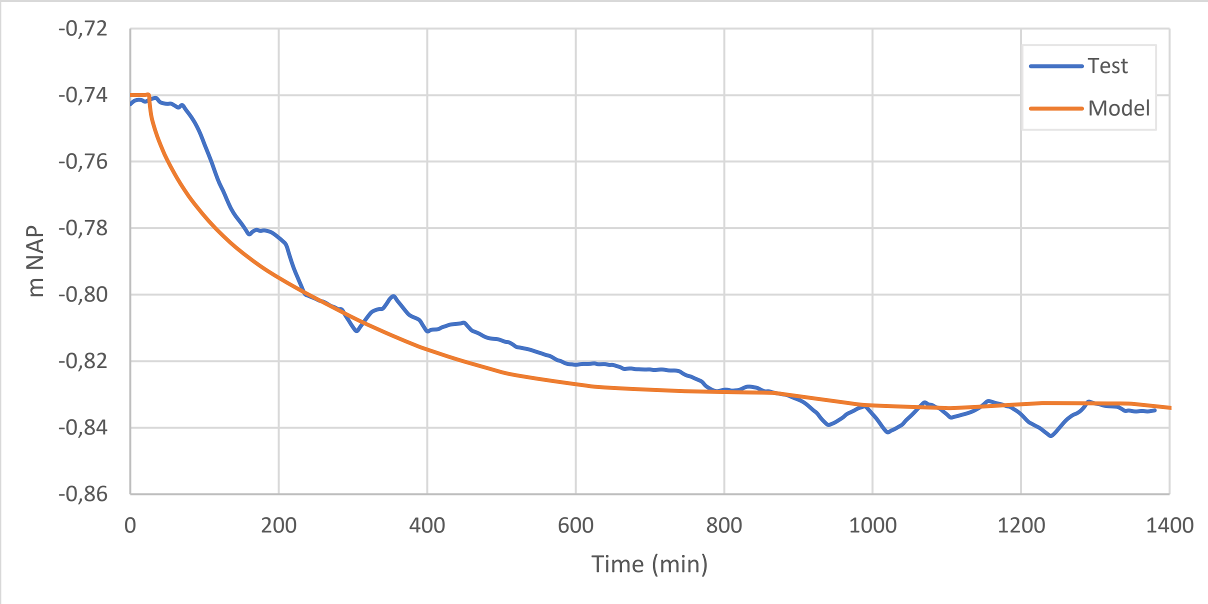

Figure 69 Model results versus test results of monitoring well K_2.2 of the drainage test Kuiperstraat

When analyzing the plot of K_1.1, two processes stand out. Initially, the drop in groundwater level fits until minute 80. Then, the model follows a smooth drop until the end of the modeled period, while the test results show a drop until -0,87 m NAP at 200 minutes (difference is 4,7 cm). The model and test have the same values around minute 900. From then on, the groundwater levels obtained from the test start to rise again. The model result continues to drop however and the difference at 23 hours is 2,8 cm.

Analyzing the model results of K_2.2, it can again be noticed that the model fits better than K_1.1. The model results drop faster at first, but from minute 200 it follows the same trajectory as the test result and end at 23 hours on the same groundwater level.

Discussion

It is chosen to compare the model results again with the monitoring wells K_1.1 and K_2.2, for the reasons before mentioned. This approach is discussed in the former paragraph and is not different for this simulation.

The modeled K_1.1 does not fit the test values. The data showed an initial drop which could not be modeled, without altering, for instance, the fluxes. Altering the flow parameters resulted in a wrongly modeled K_2.2, so this is chosen to be the best fit. Reason for the difference could be the incorporation of the drain envelope of Argex granules. When the pipe was empty, firstly the water in the envelope was drained in a faster rate as the soil itself. Then this empty space is again filled with the groundwater and the water could also be flowing along the drain in the horizontal direction, which is not modeled. Therefore, the modeled groundwater levels are higher at first. Since K_1.1 is placed very close to the drain envelope and is thus representing the groundwater levels in the Argex granules.

The modeled last 500 minutes of K_1.1 also shows a difference. The test data shows a rise in the groundwater level, while the model still shows a small drop. This again can be attributed to the horizontal flow along the drain which is not modeled but exists nonetheless.

The simulation of K_2.2 shows an initial drop faster than the test result. This could be caused by a delay in the reaction of the monitoring well.

Comparing the water flow parameters from the infiltration and drainage models shows some interesting differences. Firstly, the saturated hydraulic conductivity K_s are 0,170 and 0,124 cm/min for the infiltration and drainage simulation respectively. This difference is nothing to be concerned about but indicates a good estimation of these parameters since it does not differ significantly. The saturated volumetric water content θ_s is 0,28 and 0,105 for the infiltration and drainage simulation respectively. This difference is significant since θ_s can only vary from 0 to 1. Hysteresis can explain this difference. The soil moisture characteristic or pF-curve gives the relation between de log of the under pressure in the soil (ψ) and the soil moisture content (θ). Different soil types have different pF-curves. This pF-curve is also dependent on under which conditions this moisture content is reached. During an increase of the soil moisture content (adsorption) the curve has a different development compared to a decrease of the soil moisture content (desorption), see Figure 70, which is called hysteresis. This is (partly) caused by the capillary forces of the soil which fill the small pores during an increase in the soil moisture content, while these forces during a decrease slow-down this process. During the infiltration test, the unsaturated zone was filled, so more volume was available for the infiltrated water leading to a higher θ_s, while during the drainage test less volume was available leading to a lower θ_s.

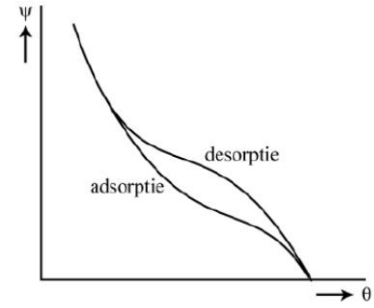

Figure 70 Hysteresis (Savenije, 2014)

The relationship between the water content of the soil and the corresponding water potential are different under saturated and unsaturated conditions. During infiltration, the unsaturated zone is filled with water and immediately emptied when the test was stopped. The hysteresis effect causes a higher θ_s. During drainage, the water content of the drained part is not at its unsaturated value, due to vapor tension of the soil. This leads to a lower θ_s in the model.

Last part of discussion is the importance of the drain envelope in the drainage test. The simulation of the drainage test is more sensitive to the change of water flow parameters in the envelope. Therefore, the values of these parameters are higher and more distinct for this test. This indicates a higher importance of the Argex granules in the drain function of the DIT sewer.

5.3 Schaepmanstraat - Infiltration test

5.3.1 Model structure

Geometry

A 2D- general geometry domain in the x-z plane, is chosen to represent a cross-section of the Schaepmanstraat. Since the conditions on both sides of the drain are the same, it is chosen to only model one side. Consequently, only half of the drain is represented in the domain and is modeled as an irrigation dripper, with a radius of 7,5 cm. A plot was made according to the cross-section of the Kuiperstraat, with a width of 4,90 meters and a height of 1,61 meters, with the bottom boundary at – 3,35 m NAP. The drain is placed in the middle of an envelope of Argex clay granules of 40x40 cm.

Flow parameters

As a process, only the water flow is relevant for this simulation. Units of time are in minutes with a time discretization from 0 to 330 min, with an initial time step of 0,5 seconds and a minimum timestep of 0,05 seconds.

Water flow parameters are estimated and shown in Table 6. Values for the drain envelope are first chosen according to the outcome of the tests in the Kuiperstraat. For this simulation, only the saturated volumetric water content θ_s and the saturated K_s are valid.

Name	θ_s [-]	K_s [cm/min]
Soil	0,55	1,0
Envelope	0,60	2,0

Table 6 Initial theoretical water flow parameters drainage test Schaepmanstraat

FE Mesh

To create an FE Mesh, for each boundary a targeted FE size has to be chosen. It is chosen to dense the finite element sizes closer to the drain since this is the domain of interest. The targeted FE size at the drain boundary was set on 1,0 cm, at the edge of the drain envelope on 2,0 cm and every other boundary on 5,0 cm. The resulting FE mesh is shown in Appendix 10.

Domain properties

The material distribution consists of the soil and the drain envelope. The drain envelope consists again of Argex clay granules. It is unclear what the soil under the Schaepmanstraat is composed of. Assumed is pumice, which has a high porosity and saturated hydraulic conductivity. The material distribution is shown in Appendix 10. The drain envelope is light blue, and the soil is dark blue. No anisotropy is added. Observation nodes are added on the initial groundwater table at the location of the monitoring wells S_1.1, S_1.2 and S_1.3 and are also visible in Appendix 10.

Conditions

To let the model run both the initial conditions (on each defined planar surface) as the boundary conditions (on every defined boundary) have to be given. The initial condition is defined as the bottom pressure head from the lowest located nodal point. For the infiltration test, this implies a value of 96 cm corresponding with a groundwater table of -2,39 m NAP.

Boundary conditions are defined as follows: the left boundary is a constant head boundary with the value of the initial condition. The (half) drain has a variable flux boundary and the infiltration capacity found in the performed test are used as input. These time-variable boundary conditions are shown in Table 7. All the other boundaries have no flux.

Time (min)	Variable flux [cm/min]
10	0
53	-1,498
330	0

Table 7 Time variable fluxes infiltration test, Schaepmanstraat, for the chosen modeled period

5.4.2 Model optimization

After running the model with these fluxes, the parameters θ_s and K_s are optimized. Four calibrations are performed with the inverse solution option in Hydrus. Calibrations are done with the outcomes of the monitoring wells S_1.1, S_1.2 and S_1.3 and the last one is done with S_1.2 and S_1.3 together. Eventually, all parameters are taken from this last calibration.

The final parameters for this model are shown in Table 8.

	θ_s [-]	K_s [cm/min]	$K_{s,v}/K_{s,h}$
Soil	0,59	1,80	1/1
Envelope	0,3	0,03	1/1

Table 8 Water flow parameters used in the final simulation of the infiltration test, Schaepmanstraat

5.4.3 Model results and discussion

In Figure 71, Figure 72, and Figure 73, the model outcome is plotted against the data obtained from the infiltration test for a period of 330 minutes.

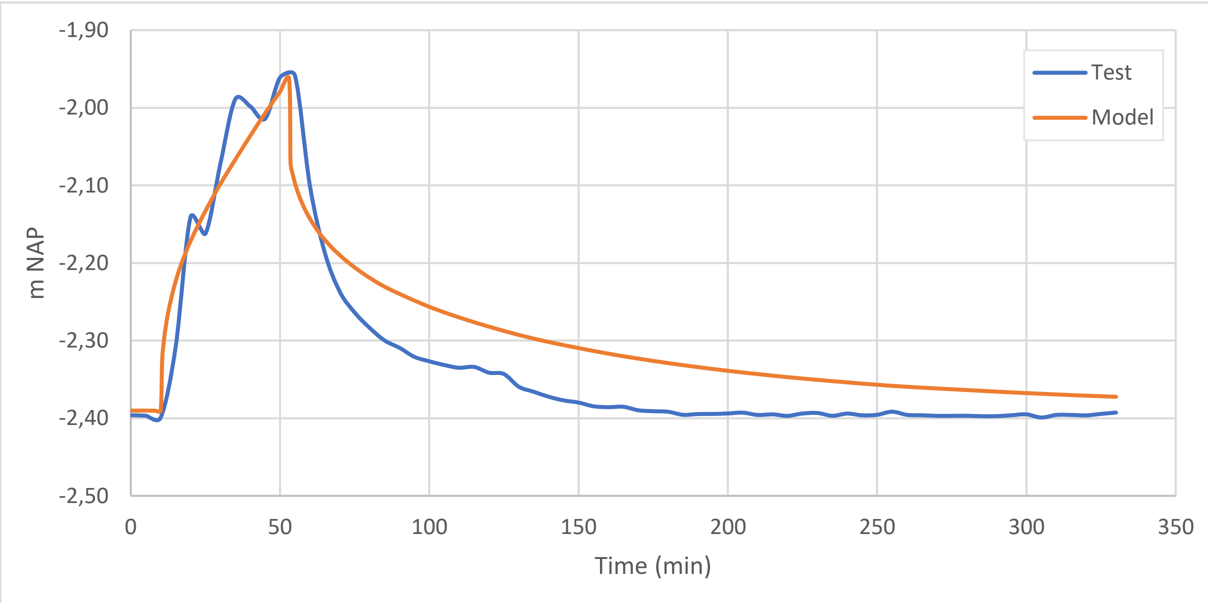

Figure 71 Model results versus test results of monitoring well S_1.1 of the infiltration test, Schaepmanstraat

The model gives the same peak as the test results at -1,96 m NAP at 55 minutes. During the 10 minutes after the end of the test, the model drops slightly faster than the test results until 64 minutes where the model has a slower return of the groundwater level to the initial level. At approximately 180 minutes, the groundwater level was at its original level, while the model does not return completely to this level in the simulated time. At 330 minutes, the difference is 1,9 cm.

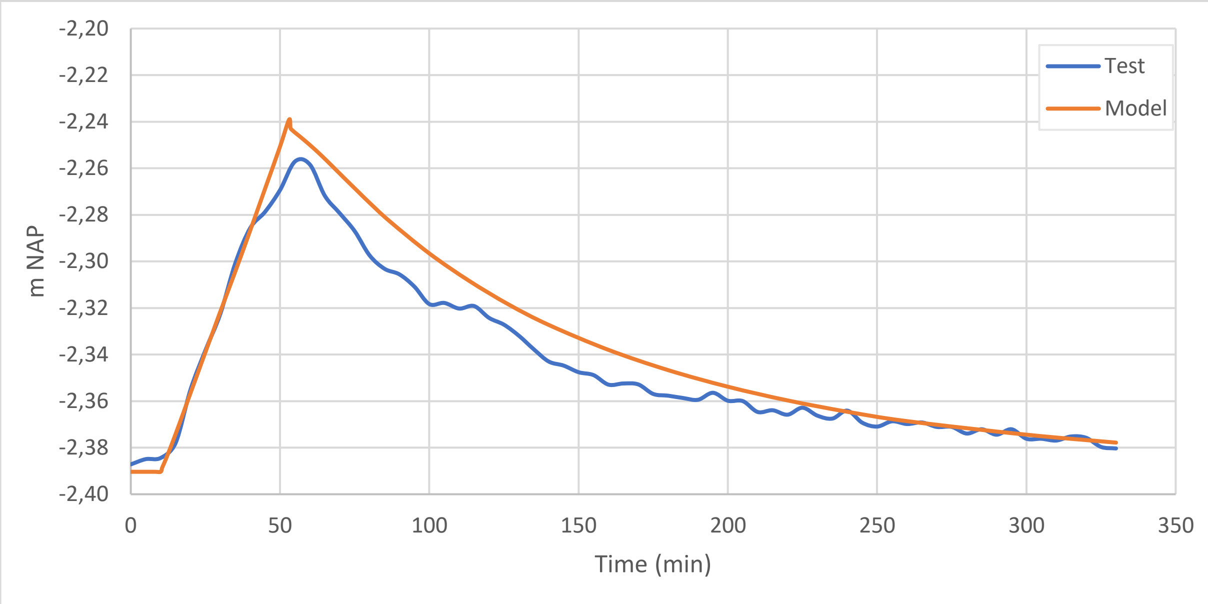

Figure 72 Model results versus test results of monitoring well S_1.2 of the infiltration test, Schaepmanstraat

The model simulation of S_1.2 follows during the test the same path as the test results showed. At the peak, the model has an overestimation of the groundwater table of 1,8 cm. After the end of the test, the model shows the same behavior as the test results and at t = 330 minutes, the model ends at the same groundwater level as the measured test result.

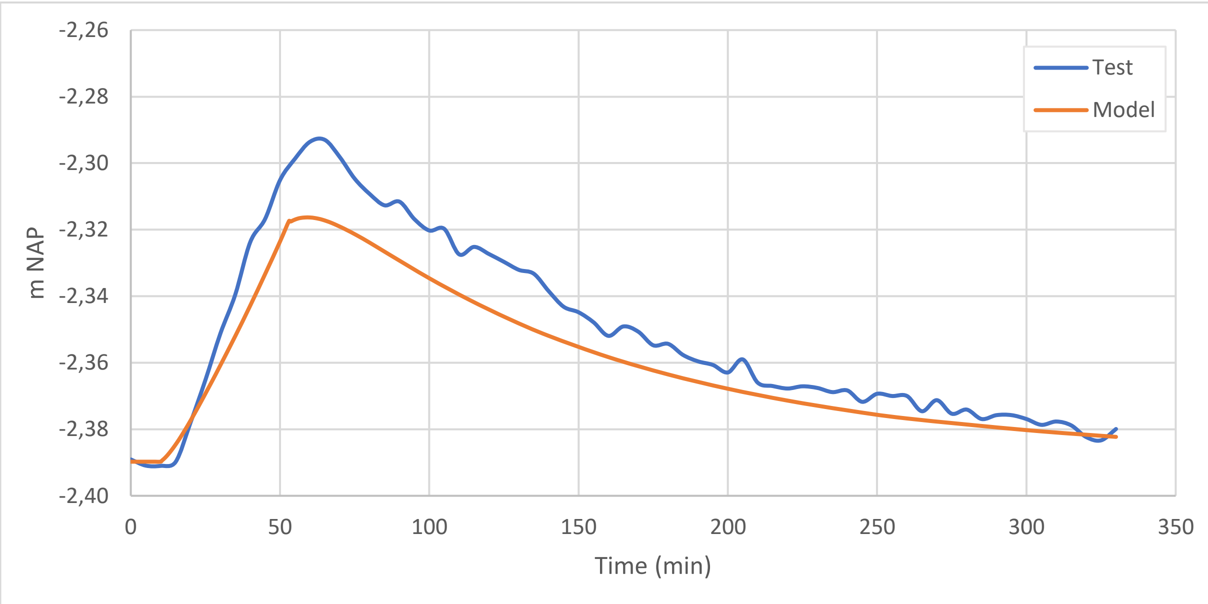

Figure 73 Model results versus test results of monitoring well S_1.3 of the infiltration test, Schaepmanstraat

The model results of S_1.3 are showing the same behavior as the test results. The initial rate of groundwater level rise at this location is slightly underestimated, leading also to an underestimation of the peak, of 1,8 cm. The return to the original groundwater level follows the same path, and the model and the test show the same groundwater level at t = 330 minutes.

Discussion

Comparing the model results with the test results of S_1.1, there are two main differences. Firstly, due to practical constraints, the test shows multiple peaks in the water level, in contrast to the model. Secondly, the model shows a slower return to the initial water level which is not reached in the simulated time. For instance, while being at the same groundwater level at 64 minutes, at minute 160, the model shows a groundwater level of -2,32 m NAP, while the test result gave a value of -2,38 m NAP. At the end of the simulation time, this difference narrowed down to 1,9 cm. It could be that the Argex envelope plays a role in the discharge of the water along the total drain in the street. Another reason could be the averaging of flux out of the drain. This flux is assumed to be constant over the entire length of the drain, while this can differ from place to place giving other groundwater levels after the test.

Comparing the model results with the test results of S_1.2, it shows a good initial rise of the groundwater level. However, the peak is slightly too high. This is the result of the executed calibration process. Chosen is to take the water flow parameters from the calibration of both S_1.2 and S_1.3. Since the peak of S_1.3 is slightly too low, these values give the best fit for both. Besides, these values give for both S_1.2 and S_1.3 an accurate return to the initial groundwater level and are therefore also the best fit.

As mentioned, S_1.3 gives a lower peak as the test results. It can be discussed if this monitoring well lies in the same soil as S_1.2. It is not clear how wide the sewer trench is. The drawings of the soil composition are outdated and current knowledge is absent, except from oral conversation with construction workers at place.

The water flow parameters θ_s and K_s are high. The saturated hydraulic conductivity of 1,80 corresponds to 25,9 m/d which is not uncommon in highly permeable soil as gravel or in this case pumice. Literature research shows a value for the volumetric water content of pumice of 0,55. This corresponds well with the found value of 0,58.

5.4 Schaepmanstraat – Drainage test

The approach to model the drainage test in the Schaepmanstraat is different from the infiltration test. Due to uncertainties in the degree of emptying of the drain pipe and, consequently, in the amount of water which is drained, a distinct drainage capacity could be not be found as mentioned in Chapter 4. Additionally, the groundwater level next to the drain gave a level lower than the level in the drain itself, so modeling the drainage test is not possible. Therefore, it is chosen to take the water flow parameter values from the infiltration test and use them as input in this model. By adjusting the inflowing fluxes, these fluxes can be estimated by fitting the results of the model to the test results. Hence, a drainage capacity can be estimated for this test, and, consequently, a k-value can be obtained.

5.4.1 Model structure

Geometry

The same 2D – geometry domain as used in the former simulation of the infiltration test is used in this simulation.

Flow parameters

As a process, only the water flow is relevant for this simulation. Units of time are in minutes with a time discretization from 0 to 750 min, with an initial time step of 0,5 seconds and a minimum timestep of 0,05 seconds.

Water flow parameters are initially taken from the results of the infiltration test in the Schaepmanstraat.

FE Mesh/Domain properties/Conditions

The FE mesh and the domain properties were the same as in the simulation of the infiltration test.

Conditions
The initial condition is the same as in the infiltration test, implying a value of 96 cm corresponding with a groundwater table of -2,39 m NAP.

Boundary conditions are defined as follows: the left boundary is a constant head boundary with the value of the initial condition. The (half) drain has a variable flux boundary, with values which are to be estimated. Three flux periods are defined, namely from 0-18 min, from 19 till 35 min and from 36 till 750 min. The first flux is zero, the second flux is firstly estimated as one third from the infiltration test (as the results of the Kuiperstraat showed) and the last flux is the value found in the results from the drainage test. All initially estimated fluxes can be found in Table 9. All the other boundaries have no flux.

Time [min]	Variable flux [cm/min]
19	0
35	0,50
750	0,01

Table 9 Initially estimated fluxes drainage test Schaepmanstraat

5.4.2 Model optimization
After running the model with these fluxes, the results were not satisfactory. Since Hydrus does not offer an inverse solution option for calculating the fluxes, this has to be done by trial and error. The value for K_s is changed for convergence purposes from 0,03 to 0,15 cm/min. In Chapter 6 is discussed if the use of the results of the water flow parameters from the infiltration test is justified.

The final estimated fluxes for this model are shown in Table 10.

Time [min]	Variable flux [cm/min]
19	0
35	0,31
750	0,022

Table 10 Estimated fluxes used in drainage test simulation of the Schaepmanstraat

5.4.3 Model results and discussion

In Figure 74, Figure 75, and Figure 76 the model outcomes are plotted against the data obtained from the drainage test for a period of 750 minutes.

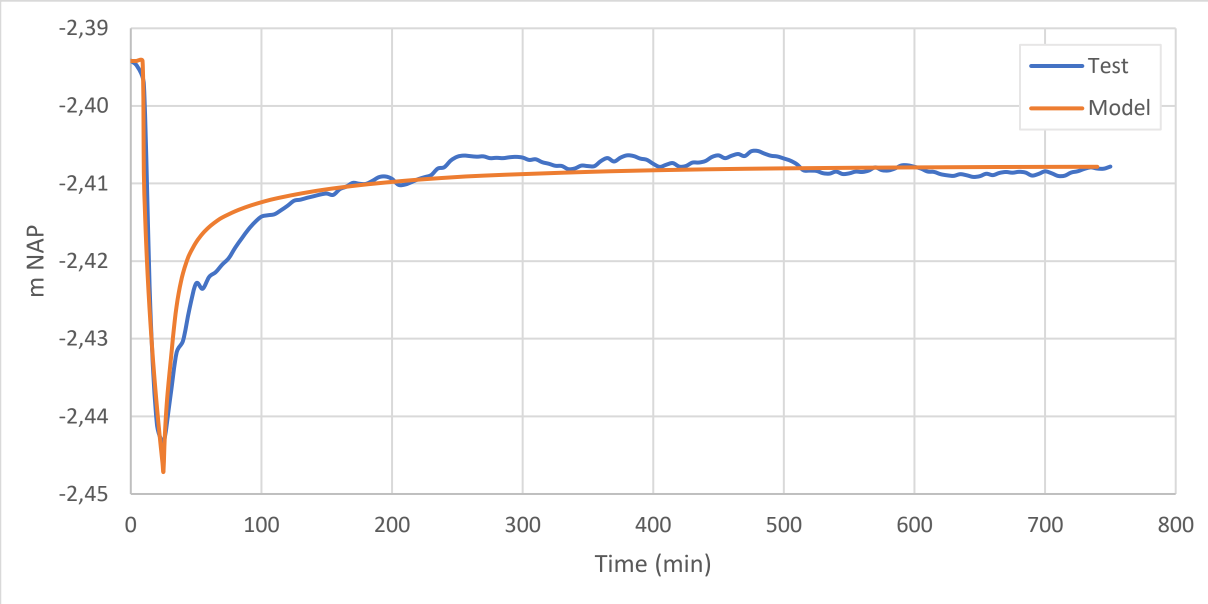

Figure 74 Model results versus test results of monitoring well S_1.1 of the drainage test Schaepmanstraat

The model result of S_1.1 has the same drop as the field data showed, with the same peak. The return to the equilibrium groundwater level is in the model faster than in the test. The model reaches -2,415 m NAP at 65 minutes while the groundwater level in the test gives this value at 95 minutes. The groundwater level reaches -2,41 m NAP at approximately the same time and follows from there on the same path until minute 750.

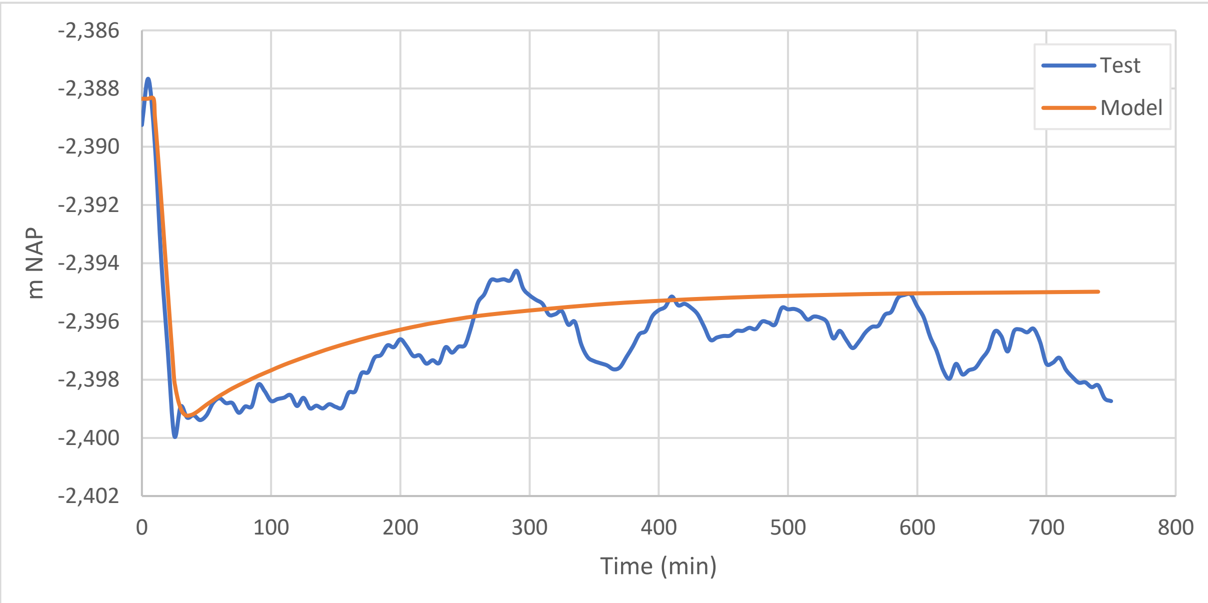

Figure 75 Model results versus test results of monitoring well S_1.2 of the drainage test Schaepmanstraat

The model result of S_1.2 shows again the same drop and peak as in the test. It follows the same path after the test and at minute 750 the difference is 4 mm. This is logical since it is chosen to calibrate on the upper side of the fluctuations of the test results.

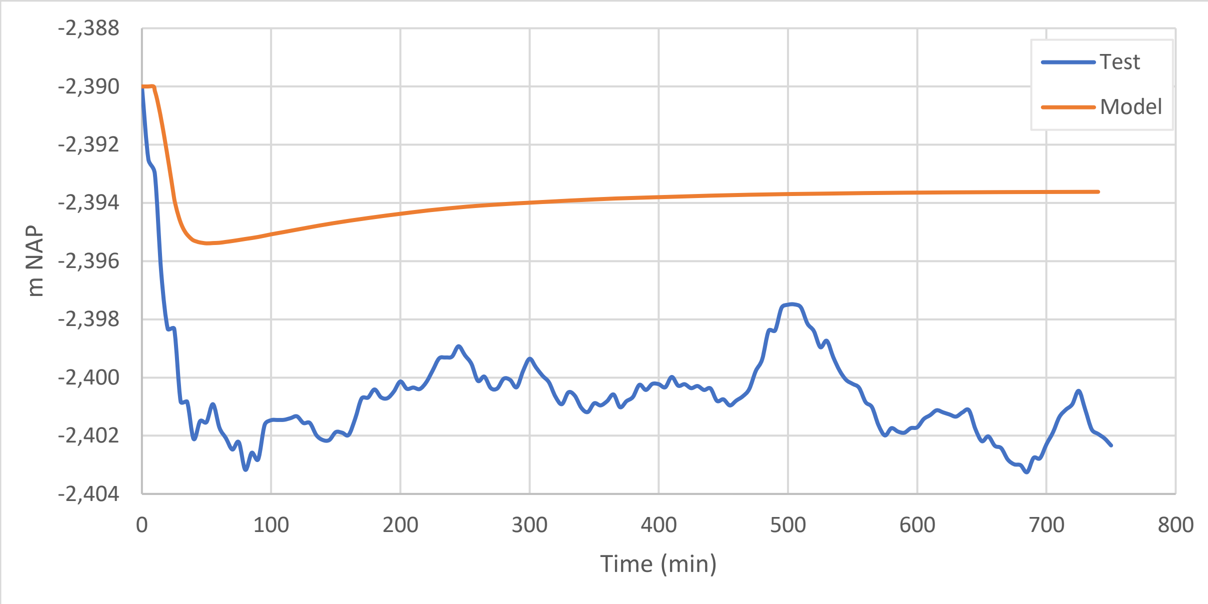

Figure 76 Model results versus test results of monitoring well S_1.3 of the drainage test Schaepmanstraat

The model results of S_1.3 shows a drop which is 15 minutes later than the test result shows. It drops until -2,395 m NAP, while the field data showed a drop until -2,402 m NAP. The difference varies between 3 and 8 mm, which can be attributed to the precision of the diver.

Discussion

The model does in its entirely not differ much from the field data, meaning in all three monitoring wells, the simulations show the same development of the groundwater level. With adapting the fluxes to the test results, way more accurate results can be achieved. It is chosen to limit the number of time-variable fluxes as much as possible since with more fluxes the results can be more accurate but less trustworthy. Adding more fluxes per time step would be curve-fitting which would only be acceptable if every choice for a chosen flux is explained. Since the flux is assumed to be linear, as less explanatory fluxes as possible are chosen.

Questionable is the adding of a flux of 0,022 cm/min (corresponding to 134 L/h) from minute 36. Firstly, the jump from 0,31 to 0,022 cm/min is not instant but gradually. Secondly, it is not sure whether the groundwater level of -2,41 m NAP is a new equilibrium of the groundwater level or if the drain is still draining that amount of water.

The only discussion point about S_1.1 is the part between minute 30 and 90. The slightly faster rise of the model can be the cause of the chosen flux value. It can also be the result of the Argex envelope which buffers the water so the return to the groundwater level is slower. However, the difference is not that significant to draw distinct conclusions.

The same is the case for the simulation of S_1.3. The graph shows a difference after minute 36 of 4 mm. Since this lies in the accuracy of the diver itself (5 mm) no conclusions can be drawn from this result.

6 Discussion

In this chapter, all relevant decisions made in the performed tests and the model simulations are discussed and criticized or approved.

6.1 Tests

The tests did not always go as planned. Looking back, decisions could have been taken differently and preparations would be done differently in some situations.

Inundation test Kuiperstraat

The infiltration into the dam is already discussed in the analyzing part and accounted for. Adding to this situation is the mixing of the sandy clay from the dam into the standing water. Especially, near the dam the standing water became turbid and the soil settled between the joints of the bricks. This can clog these joints preventing infiltration into the soil resulting in an underestimation of the infiltration capacity of this pavement.

The accuracy of the divers is typically 0,5 cm according to the manufacturer. Since in the Kuiperstraat the water column dropped less than this value during the test, these devices are not well suited for this purpose. Divers with a higher accuracy were not available. Microdivers have a smaller diameter, but not necessarily a higher accuracy and were too costly to purchase just for this one test. This pavement was expected to have a higher infiltration capacity and a higher accuracy would not be needed. To prevent this, ring infiltrometer tests could have been performed to get an estimation of the infiltration capacity before the actual infiltration test. Following from this low infiltration value the constant head test could not be applied. Ideally, a pump with a lower, more accurately adjustable capacity should be used. In this manner, both a constant head and a falling head test could be applied and compared giving more trustworthy results

Inundation test Schaepmanstraat

The discussion above about the accuracy of the used divers is also relevant for this test. However, since the infiltration rate was higher in this test the devices were more suitable. During the test, it became clear that the last part of the street could not be inundated without going over the curbs adding approximately 2,0 m^2 to the test. After 45 minutes, this area was empty. This added area can have had a minor influence on the infiltration rate since these extra square meters are paved with different bricks.

Two constant head tests were performed during the test with the 50 L bucket. This method can be inaccurate since it depends on the sight of the executor who stops the stopwatch at the moment the 50 L line is reached. Other research mentions this method to be ± 3,0% accurate with similar parameters (Lucke, 2014). With this method, the values of the infiltration velocity of the constant head test and falling head test are in the same range.

Infiltration/drainage test Kuiperstraat

During the infiltration test, the diver was placed at the bottom of the manhole and a rope secured it in place. For just one test it seemed unnecessary to fix a pipe to the manhole edge and place the diver in this pipe to protect it from possible impacts. This meant however for the drainage test that the diver had to be placed in the opposite manhole, otherwise, it would be sucked out with the water. Looking back, it would be handier and possibly more accurate to place the diver in a pipe.

The height of the street was measured just before the tests were executed. It is, however, unknown if the pipe itself had subsided in the time from the last height measurements of the pipe invert level. It is also unknown if the pipe had bent somewhere over the length. If the slopes within the pipe segment are known, a more accurate and unambiguous conclusion can be drawn about the infiltration/drainage capacity of the DIT sewer.

The manholes of the Kuiperstraat are constructed as Tegra 600 manholes. These have a changing diameter over the height, making it not easy to exactly calculate the amount of infiltrated or drained water. This implies a small inaccuracy, especially in the drainage test since the bottom of the manhole is shaped in certain bends.

Another inaccuracy in both the infiltration test and the drainage test in the Kuiperstraat is the incorporation of the gully pots which are connected to the DIT sewer. The gully pot itself is incorporated in the calculation but the connection from the gully pot to the drainpipe is not. Worst case scenario would be a total length of 2 meters of connection pipe with a standard size of 160 mm, adding 0,04 m^3 to the total drained volume. This is 5 times in order of magnitude lower than the total DIT sewer volume and therefore negligible.

The k-values of both tests come from an average groundwater level and water level in the manhole over the period of testing. Since both levels behaved linear and values of both tests were in the same range, these values are reliable for a comparison with other tested drainage systems.

The methods used in both the infiltration test and drainage test can be and are already applied in other DIT sewers (or IT sewers). The values found in these test are indicative for the inner city of Gouda since the soil in the Kuiperstraat is fill sand from the latest filling, as mentioned in the literature research in Chapter 1. It is not known how transferable these results are for other cities since the soil composition can be totally different. Besides, the amount of clogging due to for instance iron oxides is not known in this research and is a highly influential factor for the proper performance of drainage systems. Values from other research to compare with are absent.

Infiltration/drainage test Schaepmanstraat
The first assumption made in the calculations in the tests in the Kuiperstraat is the length of the non-perforated pipe at the manholes. The length of these pipes is unknown and assumed to be 6 meters on both sides, as in the Kuiperstraat. This seems justified since this length is a standard length used in the construction of drainpipes but is nowhere mentioned in the drawings.

As in the Kuiperstraat, it is not known if the invert levels of the pipe ends are according to the drawings and if the pipe had bent between the manholes. Calibrations are done with the heights that were known and from those, the height of manhole B is estimated. Especially in the drainage test, this is important. With a pipe diameter of 160 mm, a few centimeters offset leads to high uncertainties. Together with the before mentioned uncertainty in the degree of emptying of the pipe, giving a distinct value for the drainage capacity of this pipe is not possible.

Following from these uncertainties, it would be of great help to know how much water was sucked out of the pipe. Since we had no possession of a discharge meter, this pumped out water volume is not known.

The values found in these test are indicative for other drainage systems which are constructed in pumice. Since the amount of clogging is not known, it is not known how transferable these values are. Values from other research for comparison is absent.

Linking inundation test to infiltration/drainage test
In the Kuiperstraat, linking the inundation test to the infiltration/drainage test is possible with the achieved flux through the pavement into the soil and the infiltration/drainage capacities from both tests. The total area above the pipe between the manholes is 78 m^2, corresponding with a maximum infiltrated amount of water of 296 L/h through the pavement. This has to be discharged via the DIT sewer with an infiltration surface of 9,3 m^2. The potential difference in this situation is the street level minus the crown of the pipe, which is 0,48 m, assuming a filled unsaturated zone. The achieved fluxes from the infiltration and drainage test are 9,0 $L/m^2/h$ at dH = 0,28 m and 3,5 $L/m^2/h$ at dH = 0,10 m, respectively. Linearizing these values to dH = 0,48 m gives fluxes of 15,4 and 16,8 $L/m^2/h$ for the infiltration and drainage test respectively. To know the volume of water the DIT sewer can coop with, these values are multiplied with

the infiltration surface of 9,3 m^2, giving 145 and 168 L/h for the infiltration and drainage test respectively. For an overview, see Table 11. The DIT sewer is not capable of immediately discharging the maximum amount of infiltrated rainfall which can flow through the pavement.

	Achieved flux at dH = 0,28 m (inf) and 0,10 m (drain) (L/m^2/h)	Achieved flux at dH = 0,48 m (L/m^2/h)	Infiltrated volume (L/h)
Infiltration test	9,0	15,6	145
Drainage test	3,5	16,8	168
Inundation test			296

Table 11 Kuiperstraat comparison fluxes through the pavement from the inundation test and achieved fluxes in infiltration and drainage test

In the Schaepmanstraat, the same comparison can be made. The total area above the pipe between the manholes is 172 m^2, corresponding with a maximum infiltrated amount of water of 5,0 m^3/h through the pavement. This has to be discharged via the DIT sewer with an infiltration surface of 18,6 m^2. The potential difference is 0,86 m, assuming a filled unsaturated zone. The achieved fluxes from the infiltration and drainage test are 465 L/m^2/h at dH = 0,11 m and 101 L/m^2/h at dH = 0,03 m, respectively. Linearizing these values to dH = 0,86 m gives fluxes of 4,0 and 2,9 m^3/m^2/h for the infiltration and drainage test respectively. To know the volume of water the DIT sewer can coop with, these values are multiplied with the infiltration surface of 18,6 m^2, giving 74,3 and 53,7 m^3/h for the infiltration and drainage test respectively. For an overview, see Table 12. The DIT sewer is capable of immediately discharging the maximum amount of infiltrated rainfall which can flow through the pavement, depending on the chosen flux from the tests.

	Achieved flux at dH = 0,10 m (inf) and 0,03 m (drain) (L/m^2/h)	Achieved flux at dH = 0,86 m (m^3/m^2/h)	Infiltrated volume (m^3/h)
Infiltration test	465	4,0	145
Drainage test	101	2,9	168
Inundation test			296

Table 12 Kuiperstraat comparison fluxes through the pavement from the inundation test and achieved fluxes in infiltration and drainage test

6.2 Model

Naturally, a model has lots of simplifications and limitations. Choices have been made on the structure and settings of the model. The first choice made when building the model is to simulate only half of the area (with half of the drain). The assumption here is that soil conditions on both sides of the drain are the same and, additionally, the monitoring wells are also placed on just one side of the drain. However, in reality, the drain is at both locations not placed in the center of the street. On one side, the cross-section is exactly as build in the model, on the other side there are houses close to the drain (at 1,5 to 2,5 meters). This can influence the development of the groundwater level at this side which consequently can influence the groundwater level at the other side of the drain. In the Schaepmanstraat, S_1.1 and S_2.1 are placed on the other side of the drain compared to the other monitoring wells due to practical reasons at the time of placing. Ideally, all monitoring wells would be on the same side for more accuracy.

The soil in the model was composed of just two soil types, namely the drain envelope (Argex clay granules) and medium fine sand. This soil was assumed to be homogenous and isotropic at first. Since the soil was composed of fill sand which different layers were constructed at different times, as mentioned in Chapter 2 and excavated and filled again, this soil is presumably not isotropic. Therefore, anisotropy was added whereafter the model fitted better. The report of Lankelma mentioned the presence of silt in the soil which can be the cause of this anisotropy. The value of this anisotropy is estimated on 1/5 ($K_{s,v}/K_{s,h}$).

Also, anisotropy of 1/10 has been tried. A comparison can be found in Appendix 9. For the groundwater level at K_2.2, anisotropy of 1/5 gave better results than anisotropy of 1/10, as this graph shows.

Following from the assumption of a homogenous soil, the water flow parameters K_s and θ_s are also presented as distinct values for the entire soil. Especially in the Kuiperstraat, these parameters can vary from place to place. From the different calibrations performed, these parameters gave also different values. In the Kuiperstraat, the reason can also be the position of the monitoring wells as mentioned. In the Schaepmanstraat, it is not sure if S_1.3 is in the same soil as the other two monitoring well as mentioned in the discussion in Chapter 4. It can be discussed if the found values have to be presented as a value or as a range of values.

A shortcoming of this model is it gives no information about the behavior of the drain itself. Processes like clogging of geotextile, drain envelope or pipe perforations and the entrée resistance are not modeled but incorporated in the found infiltration and drainage values. This implies that the model is only generally usable once you know the specific infiltration/drainage values of the pipe. Another approach for the drainage test which could have contributed to a more general model is if the boundary of the pipe was chosen as a seepage face. To account for the entrée resistance, a slightly smaller diameter than in reality can be used. This approach was not chosen since in the manner chosen more accurate results could be achieved and both different tests were preferred to execute in the same manner to make a fair comparison. There was also no indication of resistance due to clogging in the tests.

Since the model solves the Richard equation, which has a nonlinear nature, it uses iterations to solve it. The first iteration criterium is the maximum number of iterations Hydrus is allowed to perform during any time step to solve the Richard equation. This value is set on the recommended default value of 10. According to the User Manual, it is not helpful to use a larger value than 10. If Hydrus does not converge in 10 iterations, there is a relatively small probability it will do so in more iterations. It then would be more efficient to reduce the timestep. The second iteration criteria chosen is a pressure tolerance of 1 cm, which means that the iterative process continues until for all nodes in the region the absolute change in pressure head between two successive iterations is less than this value. This tolerance is recommended by the User Manual of Hydrus. If the model was not successful, a smaller tolerance can be chosen, or the maximum iterations could be raised. In the model, this was not needed since it worked fine with these settings and choosing for lower tolerances did not improve the model. As proof, running the model of the infiltration test of the Schaepmanstraat with a pressure tolerance of 0,5 cm delivered the same results as running it with a pressure tolerance of 1,0 cm.

Time steps were chosen based on two criteria. The first was the results of the performed tests and in what range the head did change over time, so the model did not miss the peak for instance, and the time the model did run. The second criterium was whether the model could find the solution with this time step. The minimum timestep is set on $1/10^{th}$ of the initial timestep. If a timestep is chosen to low, Hydrus automatically goes up to higher timesteps until the maximum timestep, which is chosen as half the run time of the model.

It is chosen to work with average fluxes. In the infiltration test of the Kuiperstraat, the average flux is over the entire test. Firstly, average fluxes were chosen over 6 minutes. This let to reaching the peak way to fast since the fluxes in the first 18 minutes of the test were twice as high compared to the fluxes from 19 to 57 minutes. This is presumably caused by a discharge via the Argex envelope instead of into the soil. Working with an average flux gave more reasonable results. Also, in the other test, average fluxes were used over a period of two hours. Working with these periods partly averages out the uncertainties in the infiltration/drainage values. Since the pipe did not deliver the same flow into/out of the soil in every timestep, or even at every length step of the pipe, this had to be averaged. Best example of these fluctuations can be found in the drainage test in the Kuiperstraat, the fluctuations of the fluxes and the used average fluxes can be found in Appendix 11.

Outcomes of the values for K_s and θ_s are especially for the drain envelope in the Schaepmanstraat strange, if not restricted by values. θ_s is restricted to 0,6 as found in the Kuiperstraat, but unrestricted it gives a value of 0,99, which is basically an open canal. Reason can be the buffering capacity of the Argex granules or the discharge of water via this envelope along the drain. In Appendix 11, the graph shows the outcome if the envelope is neglected in the infiltration test of the Schaepmanstraat and the soil is only composed of pumice. It shows a slightly lower peak than the original, but the return to the initial groundwater level is the same. The root mean square error (RMSE) of this neglection is 0,32 cm, 0,03 cm and 0,01 cm for S_1.1, S_1.2 and S_1.3 respectively. Questioned can, therefore, be the use of Argex granules if pumice is used as a soil in the sewer trench. Besides, the K_s of 0,03 cm/min found in the infiltration test in the Schaepmanstraat let to convergence errors in the model of the drainage test. Therefore, this value is changed to 0,15 for the drainage simulation. This value is then tried in the infiltration test. As in the neglection of the complete envelope, this change did not differ the outcomes much. The root mean square error (RMSE) of this change is 0,052 cm, 0,004 cm, and 0,002 cm for S_1.1, S_1.2 and S_1.3 respectively.

Since Hydrus does not have an inverse solution option for the fluxes, these had to be found manually. The final value is calibrated on the peak of the test results and set on 0,31 cm/min which corresponds to 1885 L/h for this specific drain. If 0,32 cm/min was chosen, this would correspond with 1945 L/h. Working in this simulation with an accuracy of 0,01 leads to an inaccuracy of 60 L/h. Therefore, the drainage capacity found is merely an indication of that value than a distinct outcome. In addition, the θ_s used in the infiltration test is used in the drainage test. The infiltration test and the drainage test in the Kuiperstraat showed a difference in θ_s presumably caused by hysteresis as explained in Chapter 5. Pumice is assumed to be less prone to this effect since it is a coarse material capable of fast discharging of a large amount of water.

7 Conclusions and recommendations

7.1 Conclusions

This research is a part of the Gouda project carried out by the Dutch coalition "Stevige stad op slappe bodem". It concentrates on the hydrological performance of the constructed DIT sewer, which function is to counteract strong groundwater fluctuations leading to both groundwater flooding and low groundwater levels. Groundwater flooding leads to water damage and nuisance in houses and gardens, while low groundwater levels enhance land subsidence and possible wooden pile foundation rot. To investigate their performance, DIT sewers were tested in Gouda at two sites. The Kuiperstraat was chosen as a worst-case scenario in the inner city of Gouda, because of the low-permeability type of subsurface. At the second location, the hydrological performance of the drainage/infiltration sewer was investigated in the H.A.J.M. Schaepmanstraat in Korte Akkeren, where the subsurface has a much higher permeability. In this manner, differences between the two case studies can be observed and linked to the different characteristics of the locations.

Knowledge and experience on the performance of DIT sewers are scarce. Although DIT and IT sewers are also used in cities like Amsterdam and Eindhoven, observed and reliable values on the infiltration and drainage capacity are hardly available. Some research is available on the drainage time and clogging of IT sewers in Eindhoven and on a DIT sewer in Amsterdam. Other research has focused on the performance of regular drains and groundwater flow into drains. Clogging risks, like the formation of iron (hydr)oxides on the geotextile and perforations of the pipe, are not widely studied, despite this being a commonly known process, especially in areas with iron-rich groundwater.

How much water infiltrates directly into the soil and influences the operation of the DIT sewer depends on the infiltration capacity of the pavement above a DIT sewer. To assess this capacity, inundation tests were performed. In the Kuiperstraat, this test gave an infiltration capacity of 3,8 mm/h. The inundation test performed in the Schaepmanstraat resulted in an infiltration capacity of 29 mm/h. Due to larger paving joints, the Schaepmanstraat is capable of infiltrating more stormwater than the Kuiperstraat.

Essential for the performance of a DIT sewer is its capacity to drain and infiltrate water. The infiltration test performed on a segment of DIT sewer in the Kuiperstraat showed an infiltration capacity up to 84,5 L/h on an infiltration area of 9,3 m^2. The corresponding k, defined as the infiltration capacity over the area of infiltration (the perforated pipe wall surface) at a given potential difference between the water level in the pipe and the groundwater level at 10 cm perpendicular to the DIT sewer pipe, is 9,0 $L/m^2/h$ at dH = 0,28 m. The groundwater level next to the DIT sewer returned to its original level within 3,5 hours. The drainage test performed on the same segment of DIT sewer in the Kuiperstraat showed a drainage capacity up to 33,0 L/h. The groundwater level next to the DIT sewer returned to its original level within 2 days. The corresponding k-value is 3,5 $L/m^2/h$ at dH = 0,10 m. The groundwater analysis on both the infiltration and drainage test indicates that the DIT sewer in the Kuiperstraat is fulfilling its intended purpose.

The infiltration test in the Kuiperstraat was performed a second time after cleaning the pipe with a high – pressure hose. Sediments could have clogged the pipe, as gully-pots in the street are connected to this DIT-sewerage system. Monitoring results of the second test showed no traces of clogging; the results were very similar to the results or the first test.

The infiltration test performed on a segment of drainage/infiltration sewer in the Schaepmanstraat showed an infiltration capacity up to 2,4 L/s on an infiltration area of 18,6 m^2. The corresponding k-value is 465 $L/m^2/h$ at dH = 0,10 m. The drainage capacity could not be measured reliably due to the high flow but is estimated to be 0,5 L/s with the Hydrus2D groundwater model. This corresponds to a k-value of 101 $L/m^2/h$ at dH = 0,03 m. The groundwater analysis showed in both test an almost immediate return to the initial groundwater level. Concluded is that the drainage/infiltration sewer in the Schaepmanstraat is excellently fulfilling its intended purpose.

The soil texture in the Kuiperstraat is silty to medium fine sand, while the soil in the Schaepmanstraat consists of pumice. The major difference in infiltration and drainage capacity between both locations shows the importance of soil texture. The soil hydraulic parameters K_s and θ_s (saturated hydraulic conductivity and saturated volumetric water content) were estimated using the unsaturated and saturated groundwater flow model Hydrus2D. For the infiltration test in the Kuiperstraat, K_s was estimated on 2,5 m/d and θ_s is 0,28. For the drainage test in the Kuiperstraat, the estimated K_s was 1,7 m/d and θ_s is 0,11. The low θ_s in the drainage test is presumably caused by hysteresis effects. The soil hydraulic parameters of the Schaepmanstraat were estimated on 25,9 m/d and 0,55 for K_s and θ_s respectively. It can be concluded that a DIT sewer constructed in a soil with a high permeability is capable of coping with a large amount of stormwater while maintaining the intended groundwater table.

With the Hydrus model, not only groundwater level response of the tests could be modeled, but also the role of the drain envelope with the Argex granulates. The drainage test in de Kuiperstraat showed a very fast drop of the groundwater level adjacent to the drain. This drop could not be modeled in the used model. It could indicate a high water content in the envelope around the DIT pipe, so water was directly available and started to flow into the drain. It can also indicate a flow along the pipe, so water was discharged away from this measuring point through the Argex granulate. Furthermore, calibrations during the modeling process on the groundwater level behavior next to the pipe gave very high values for θ_s (approaching 1). This indicates a different behavior of the water flow in the Argex envelope compared to the water flow in the subgrade soil itself. Again, a flow along the pipe in the envelope could be the reason for this high value. The modeling of the Schaepmanstraat showed a very low difference in soil hydraulic parameters K_s and θ_s between the pumice and the drain envelope. Questioned could, therefore, be the need for an Argex envelope if high-permeable soil such as pumice is used as subgrade material.

The modeling of the Kuiperstraat indicated the presence of anisotropy in the soil, presumably caused by the presence of silt in the fine sandy soil. The anisotropy can be caused by the way of refilling the trench where the silt ends up in a microlayer and hampers the vertical groundwater flow. This is another reason to avoid using the excavated soil as subgrade material for filling the excavation of the sewer trench and road foundation. As can be seen in the in Schaepmanstraat, pumice of any other course, permeable and lightweight material would perform better.

7.2 Recommendations

In this research, only a small part of the DIT sewer is investigated. It would be interesting to know the behavior of a complete DIT sewer system. By performing a long-term research with strategically chosen groundwater level measuring points, the effect of several rainfall events on the performance of this system could be studied. By finding a way to measure the flow in the DIT sewer pipe an estimation of the amount of stormwater entering the pipe in relation to the groundwater level at that time could be made. By continuously monitoring the groundwater level at several locations along and perpendicular to the DIT sewer, more information comes available on the stability of the groundwater table due to the effect of a DIT sewer pipe instead of a regular, watertight stormwater drain pipe.

The master thesis work of Weicheng Chen (2018) started with the development of a groundwater flow model for the inner city of Gouda. The presented values for the infiltration and drainage capacity of the DIT sewers in the Kuiperstraat and Schaepmanstraat can now be used to enhance this model. With the model and these values, the effect of the implementation of DIT sewers at other locations in Gouda's inner city could be investigated together with possible other scenarios such as more intense rainfall events due to climate change and application of other adaption measures such as more permeable pavements.

For further research, it is recommended to do a CCTV camera inspection inside (a part) of the DIT sewer pipe. In this way, possible clogging can be observed. At the same time, the inclination could be measured, which would indicate (uneven) subsidence of the pipe. Ideally, a part of the DIT sewer could be dug up after a chosen period (years) of operation and together with the Argex granules envelope be studied on its clogging conditions. There is little known about the clogging of DIT sewer pipes. Our experiments in Gouda showed no indication of clogging or significant sediment accumulation, but no distinct conclusions can be drawn from the observation on one pipe segment. However, since the DIT sewer pipe also discharges stormwater, sediment accumulation is a real risk. These pipes have to be inspected regularly and cleaned when required. How regular and how extensively is however unknown. Also, other clogging risks such as iron (hydr)oxide deposits are so far unknown in the Gouda's inner city. Research on the groundwater quality, and in particular on the iron concentration, is lacking. Investigating the groundwater quality at other locations in Gouda where a drainage system is present could expose at what locations iron (hydr)oxide deposits are a risk.

The sewer system in the Schaepmanstraat consists of a foul water sewer, a stormwater sewer, and a drainage/infiltration pipe. As the research shows, the drain is capable of high infiltration rates. Therefore, replacing the drainage/infiltration pipe and the stormwater sewer with one DIT sewer as in the Kuiperstraat would be a possibility. This solution would be cheaper to construct and the stormwater ends up directly into the soil. This solution is also applicable in other streets with similarities to the Schaepmanstraat.

In the Kuiperstraat, the tested pipe length between the manholes amounts 24 meters, from which only 12 meters midway are perforated. Only half of the infiltration area is used and therefore it is recommended to replace the two non-perforated pipe lengths of 6 meter with a perforated pipe or at least re-evaluate the current construction so more pipe length is perforated and contributing to the infiltration or drainage function of the DIT sewer.

One of the functions of the constructed DIT sewers is to infiltrate stormwater into the soil. The Kuiperstraat has, however, a pavement allowing very little infiltration. Recommended is to re-evaluate the chosen pavement and subgrade material which is used above the DIT sewer constructions. Pavement bricks with spacers and porous, permeable, lightweight subgrade soil material as used in the Schaepmanstraat seem to be a more logical choice to enhance stormwater infiltration.

This research shows significantly lower permeability of the subgrade soil of the Kuiperstraat compared to the Schaepmanstraat. The soil of the Kuiperstraat is excavated during construction and again used to fill the trench. Presumably, the reason for this choice is the contamination of the soil, which would induce costs for the municipality of Gouda if replaced and processed. However, this soil's low permeability and the presence of anisotropy, as found in the model, could hamper the operation of the DIT sewer. It is recommended not to use this soil, but instead, use a high permeable subgrade material for filling the road excavation or at least the area surrounding the DIT sewer pipe. The more porous and permeable the soil material surrounding the pipe, the better the DIT sewer is capable of controlling the groundwater levels in its vicinity. So, using lightweight filling soil as pumice or Argex (broken) granules would be a more attractive and effective solution in similar situations.

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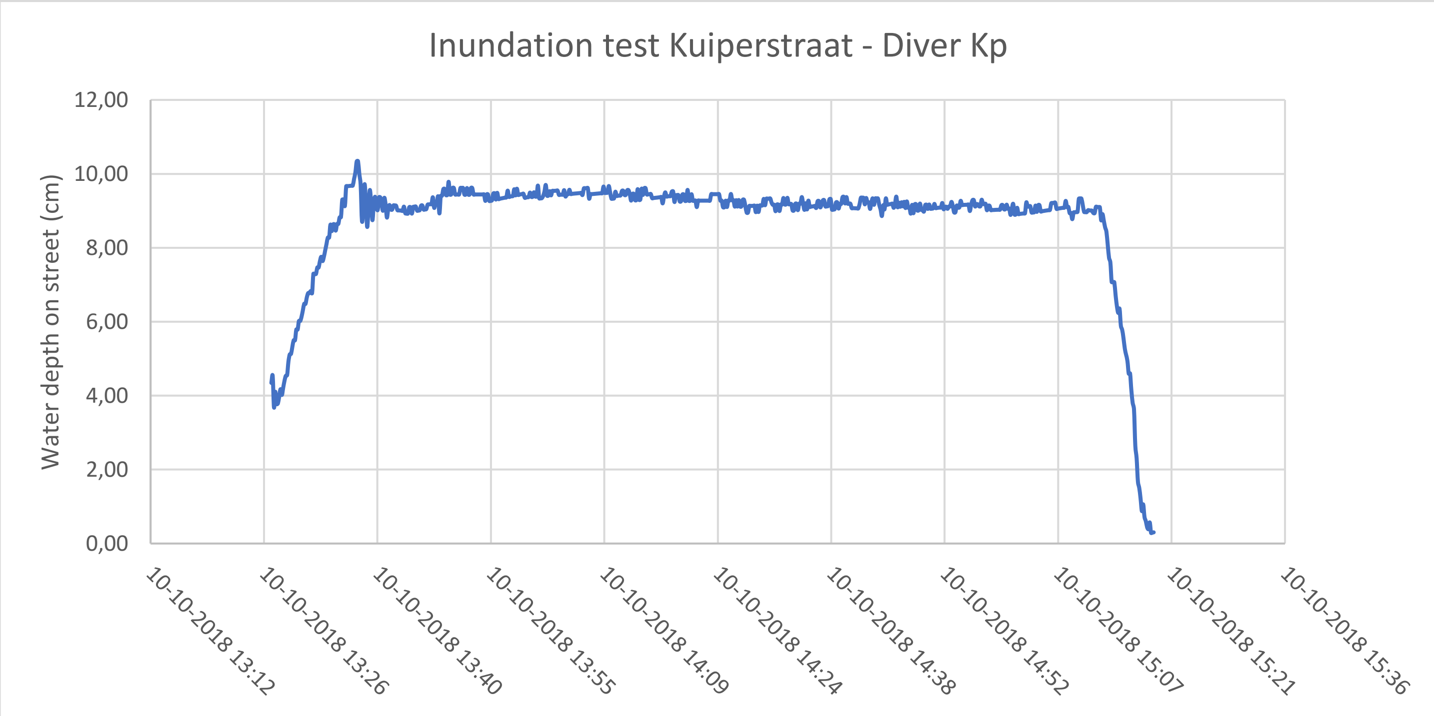
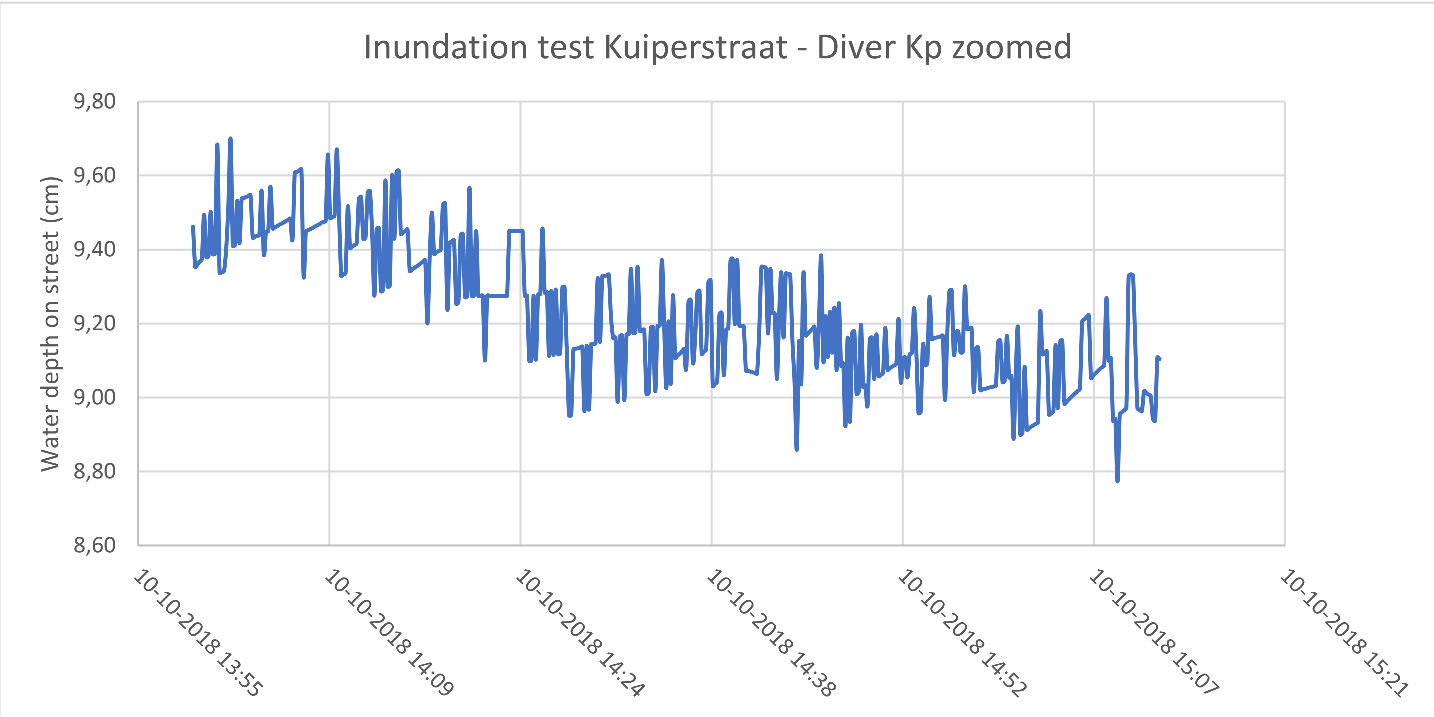
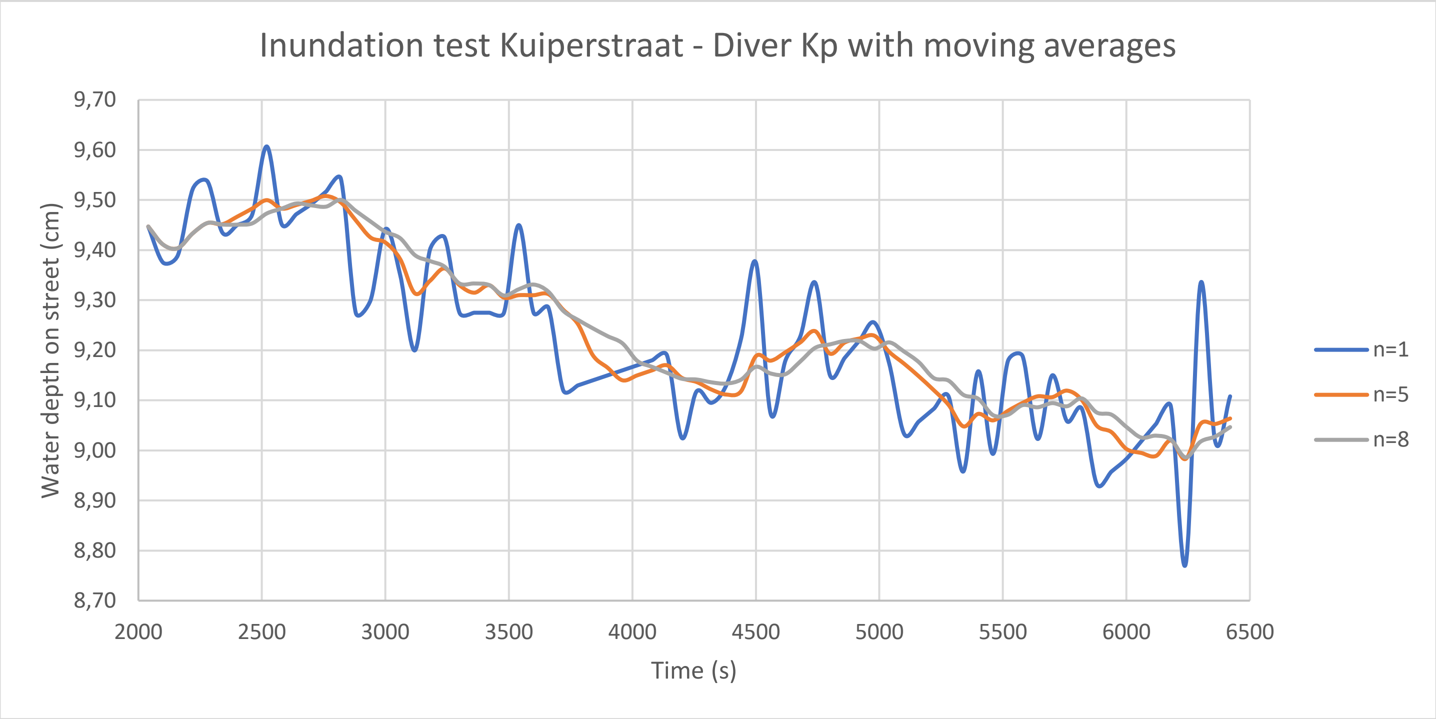
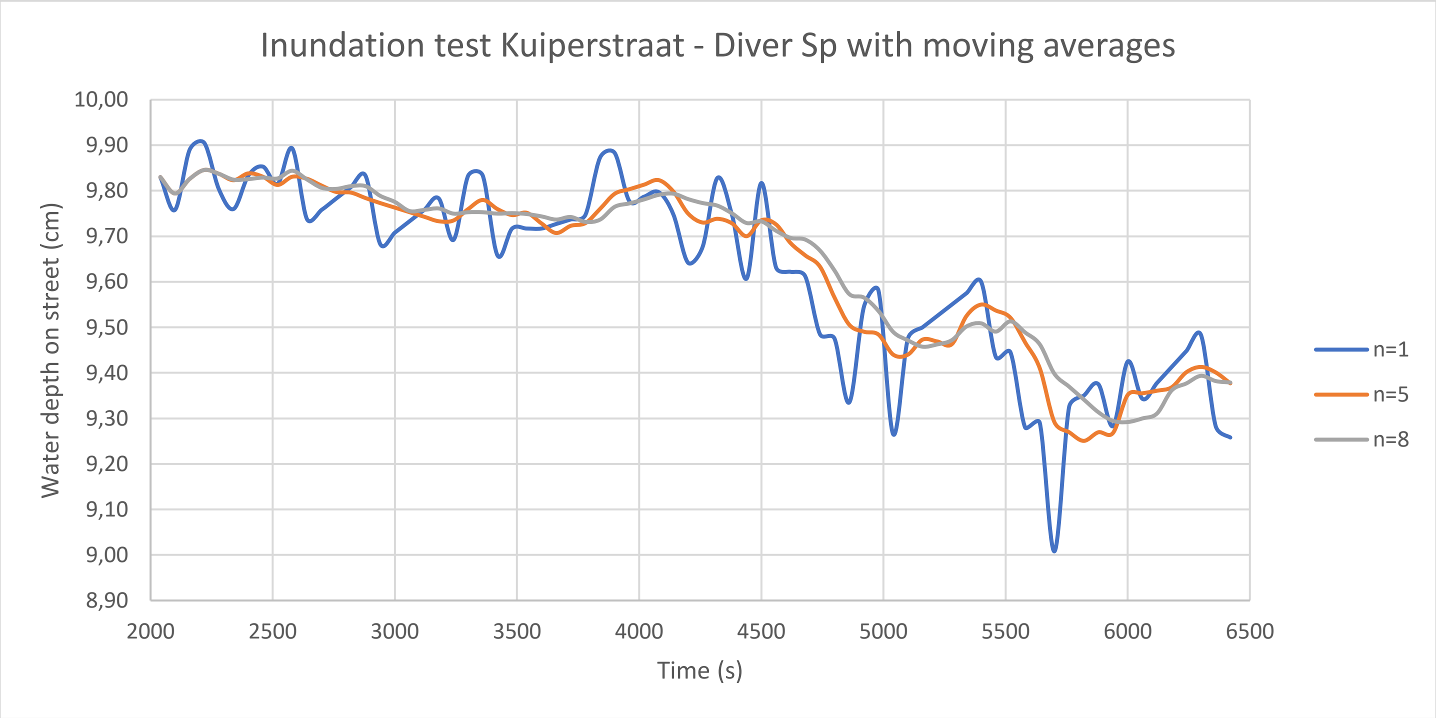
Appendices

Appendix 1 Raw data of the inundation test in the Kuiperstraat and graphs with applied moving average method

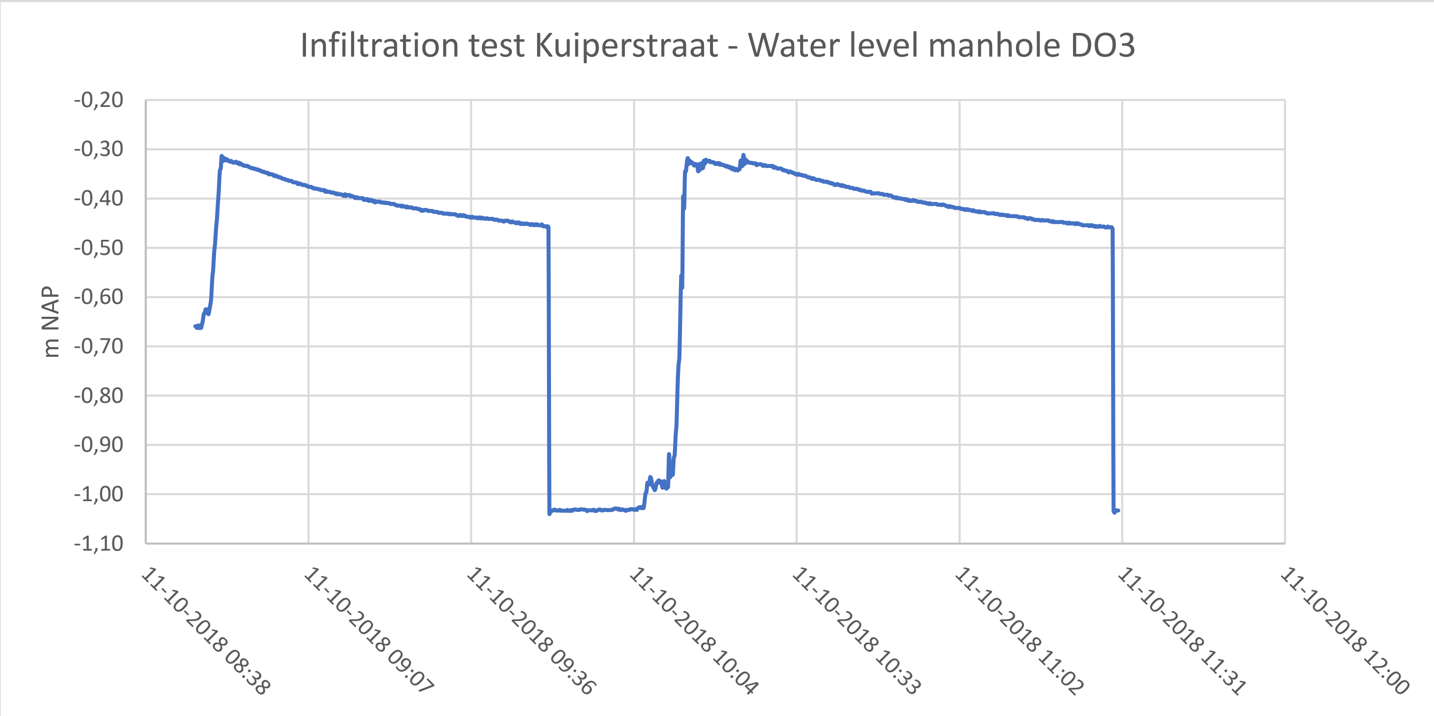


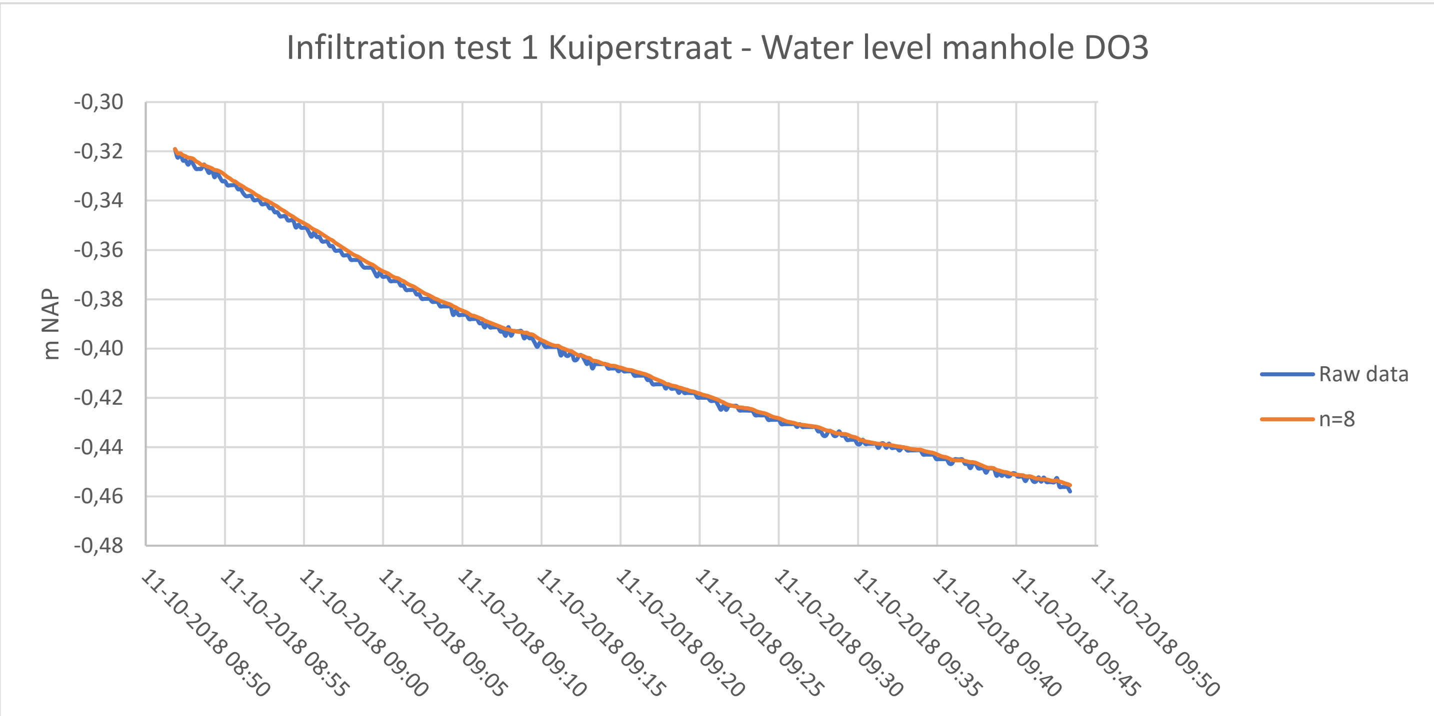


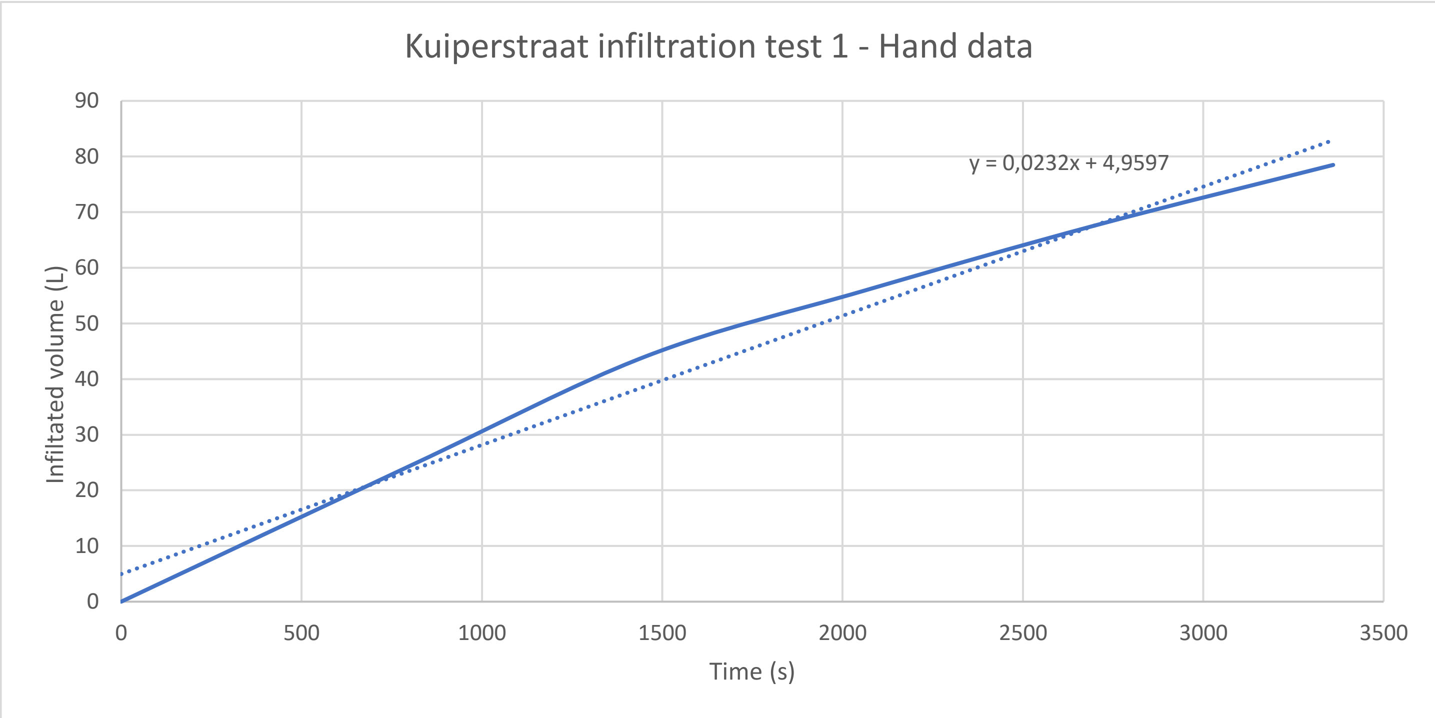

Inundation test Kuiperstraat - Diver Kp with moving averages
Water depth on street (cm)
9,70
9,60
9,50
9,40
9,30
9,20
9,10
9,00
8,90
8,80
8,70
2000
2500
3000
3500
4000
4500
5000
5500
6000
6500
Time (s)
n=1
n=5
n=8


Inundation test Kuiperstraat - Diver Sp with moving averages
Water depth on street (cm)
10,00
9,90
9,80
9,70
9,60
9,50
9,40
9,30
9,20
9,10
9,00
8,90
2000
2500
3000
3500
4000
4500
5000
5500
6000
6500
Time (s)
n=1
n=5
n=8

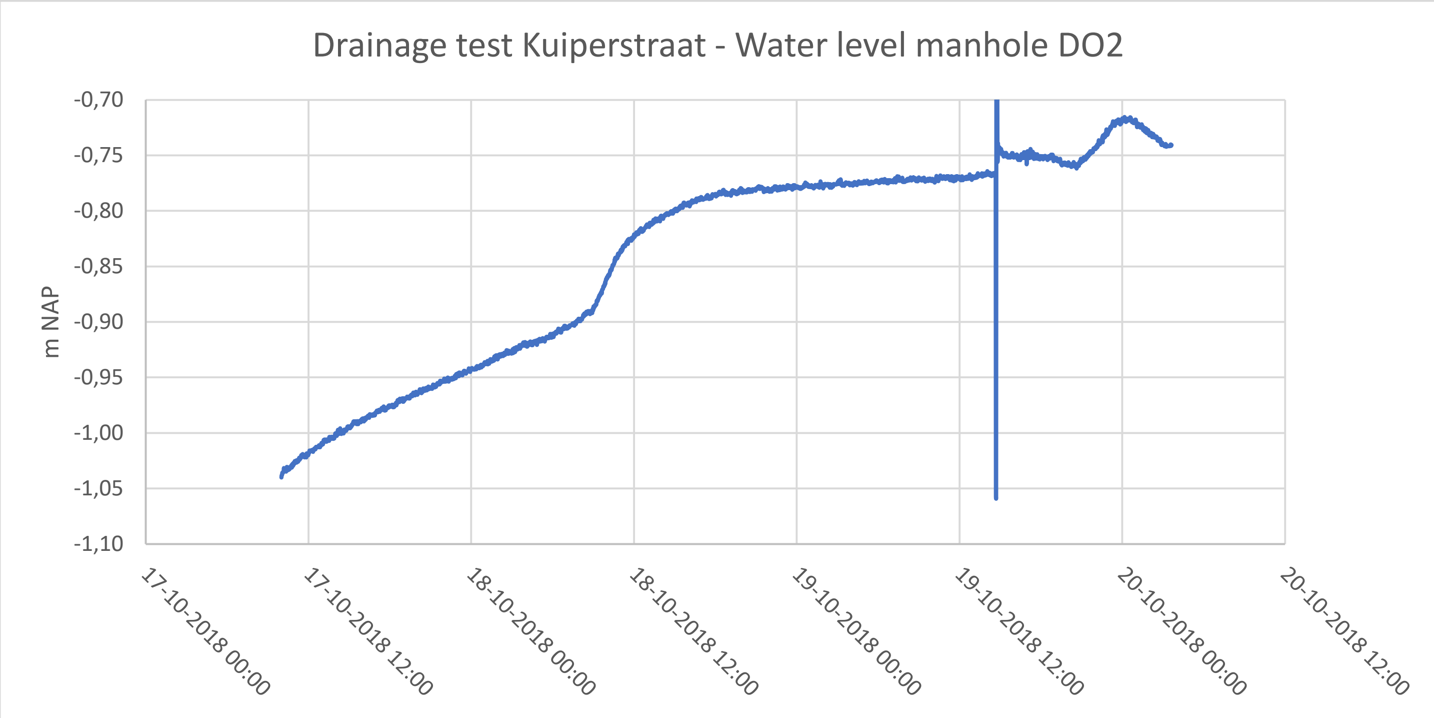
Appendix 2 Raw data infiltration test Kuiperstraat with applied moving average method and hand measurements

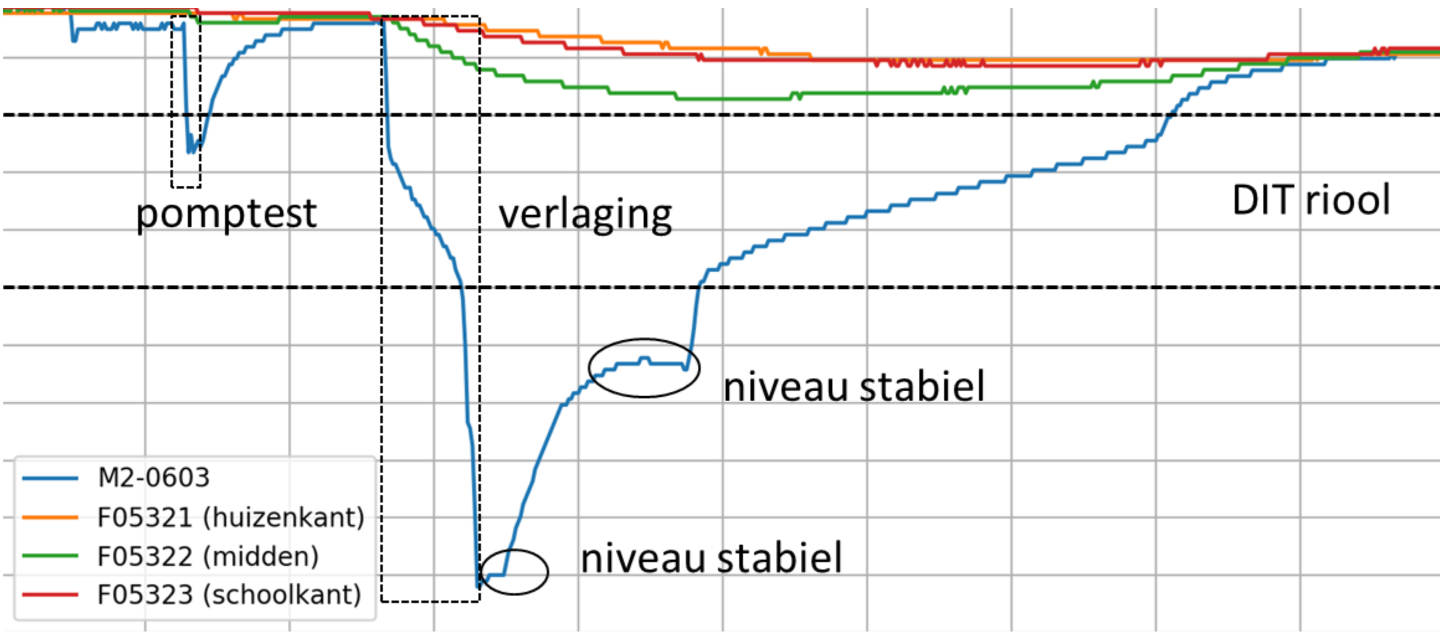




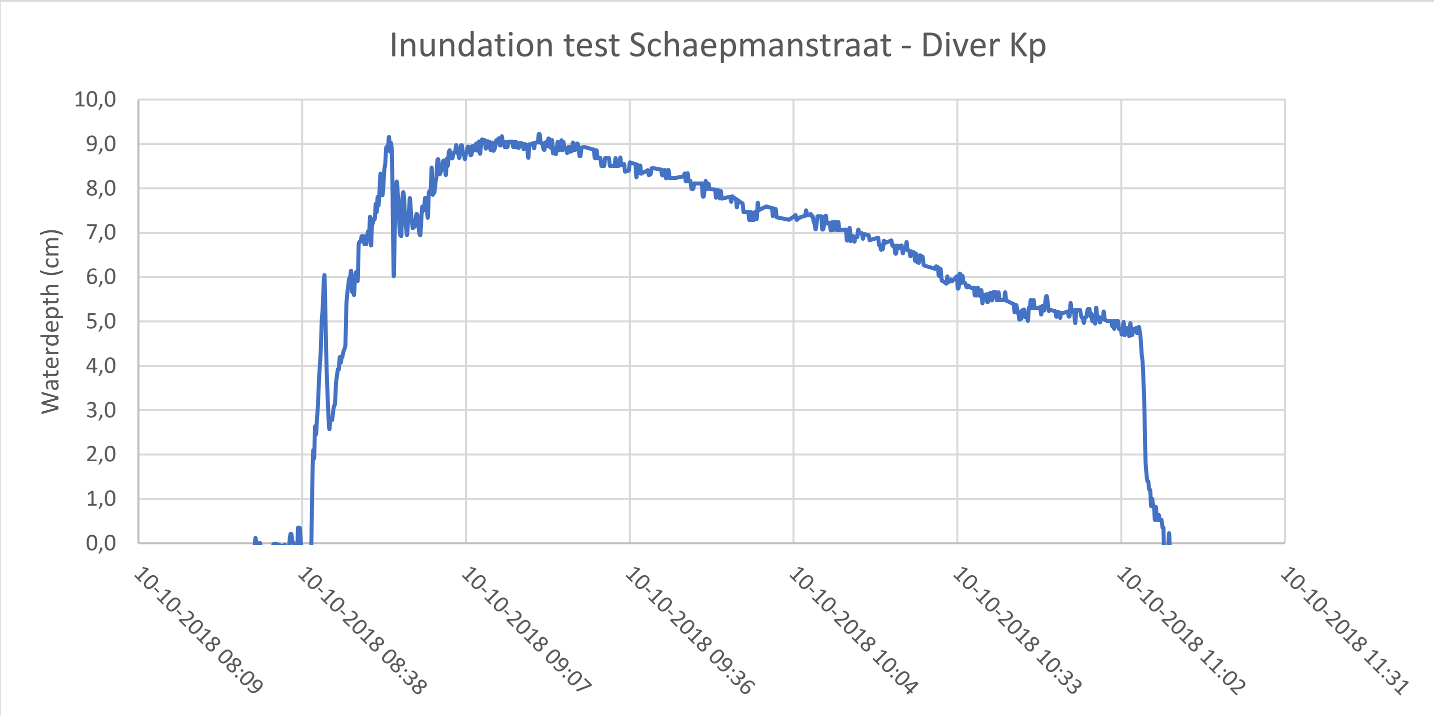

Kuiperstraat infiltration test 1 - Hand data
y = 0,0232x + 4,9597
Infiltated volume (L)
Time (s)
0
10
20
30
40
50
60
70
80
90
0
500
1000
1500
2000
2500
3000
3500

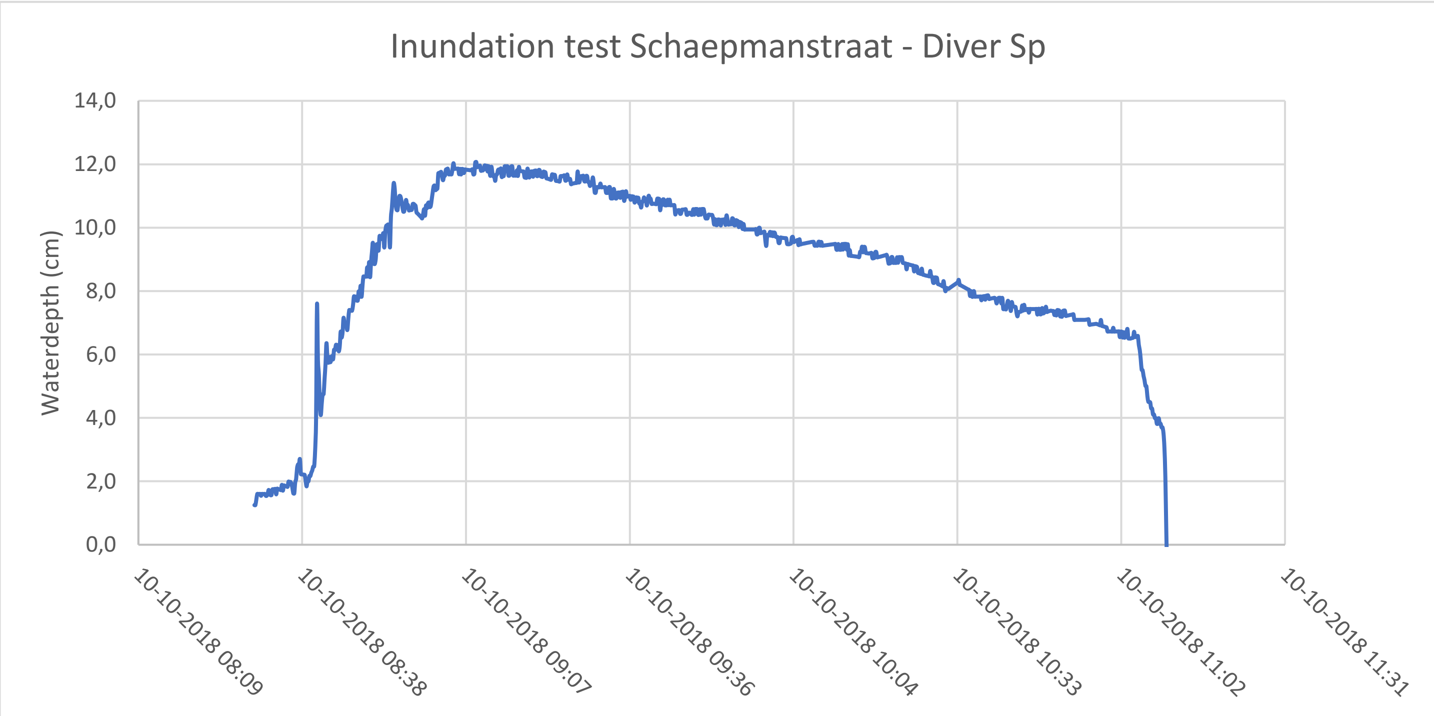
Appendix 3 Raw data drainage test Kuiperstraat and drainage test Argonautenstraat, Amsterdam



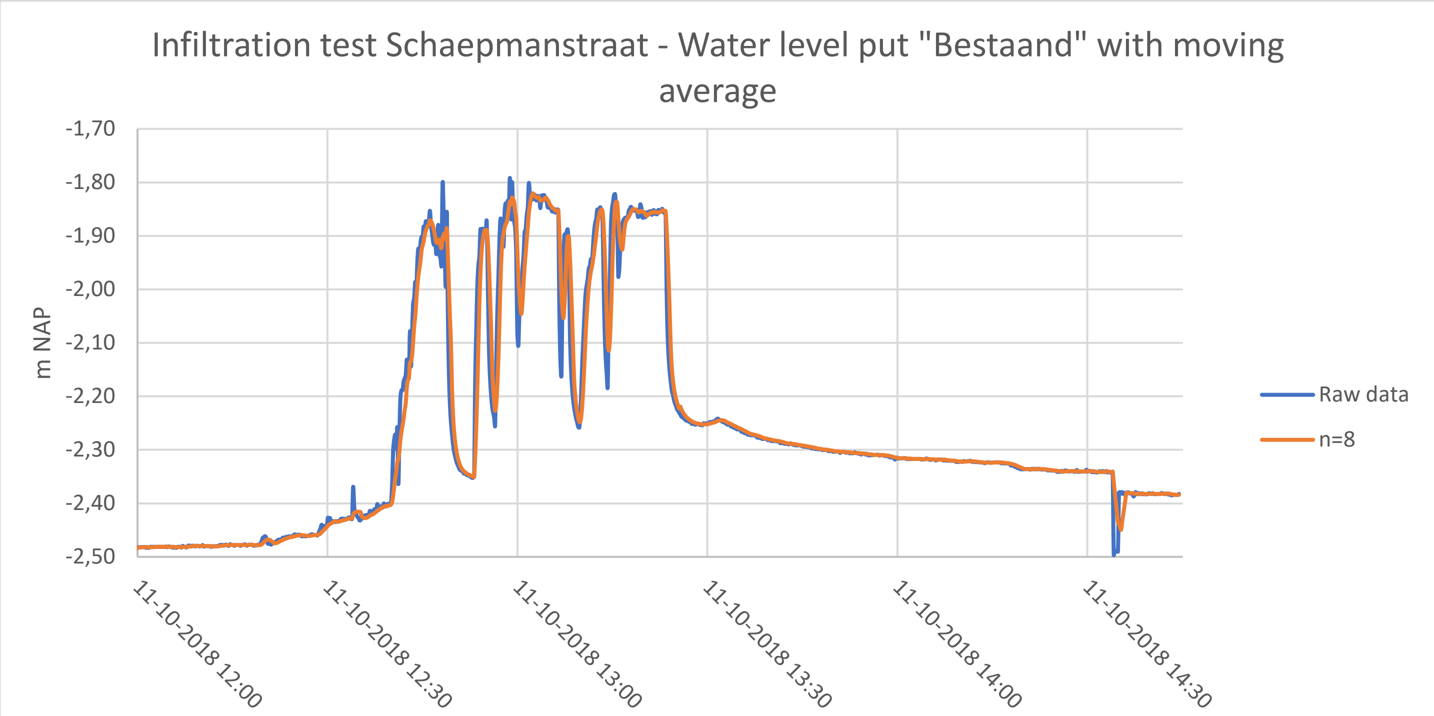


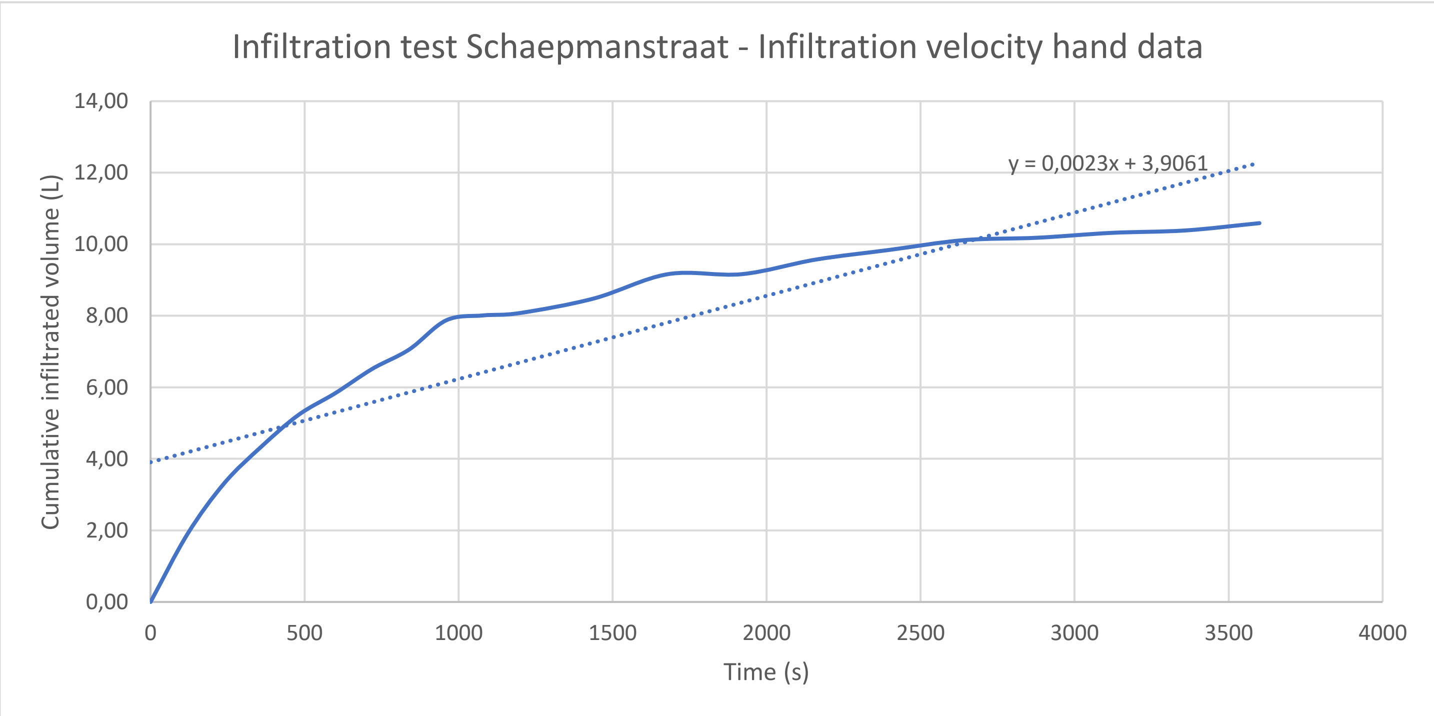
Appendix 4 Raw data inundation test Schaepmanstraat

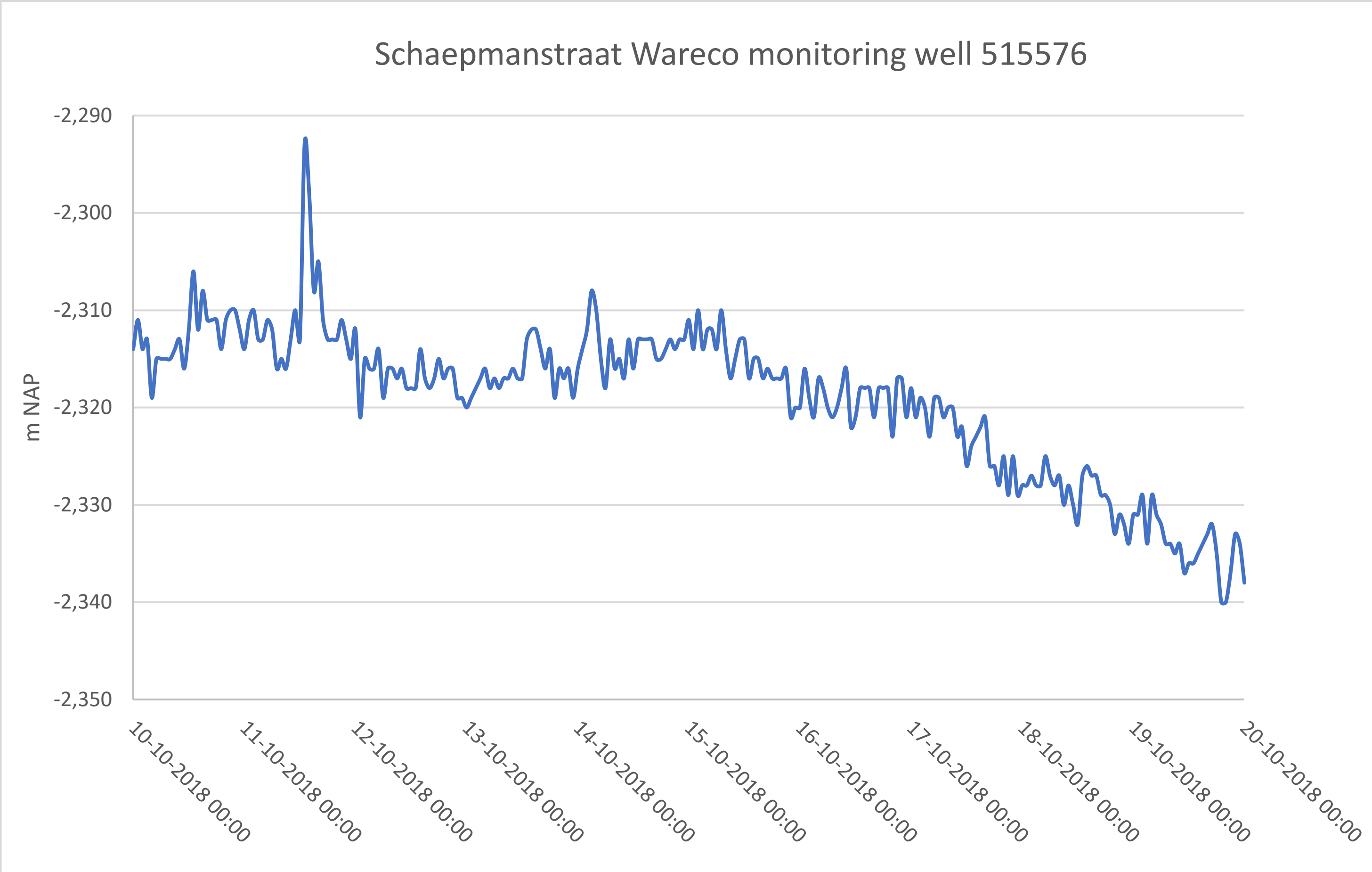




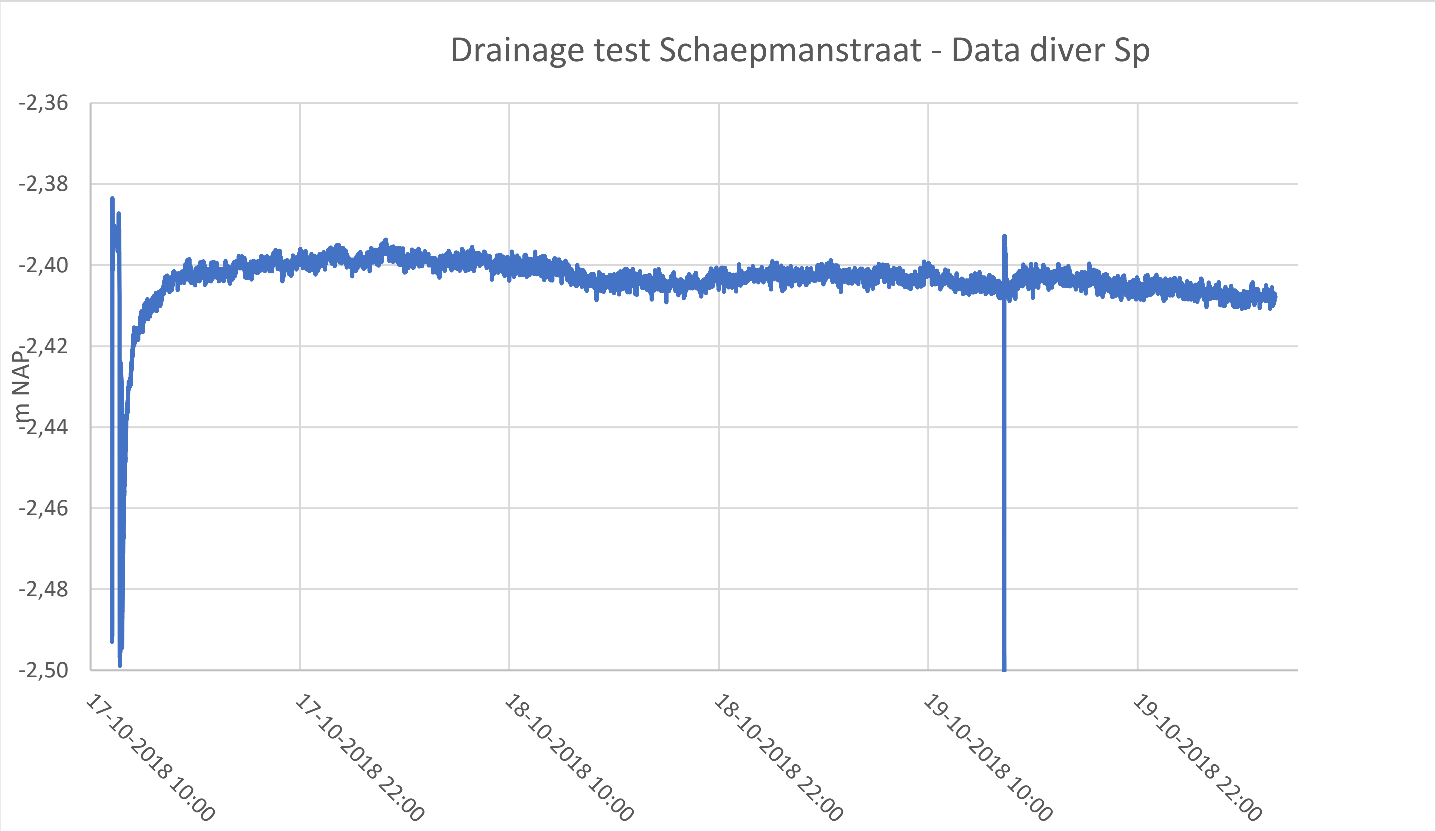
Appendix 5 Raw data infiltration test Schaepmanstraat with applied moving average methods and hand measurements and Wareco monitoring well data

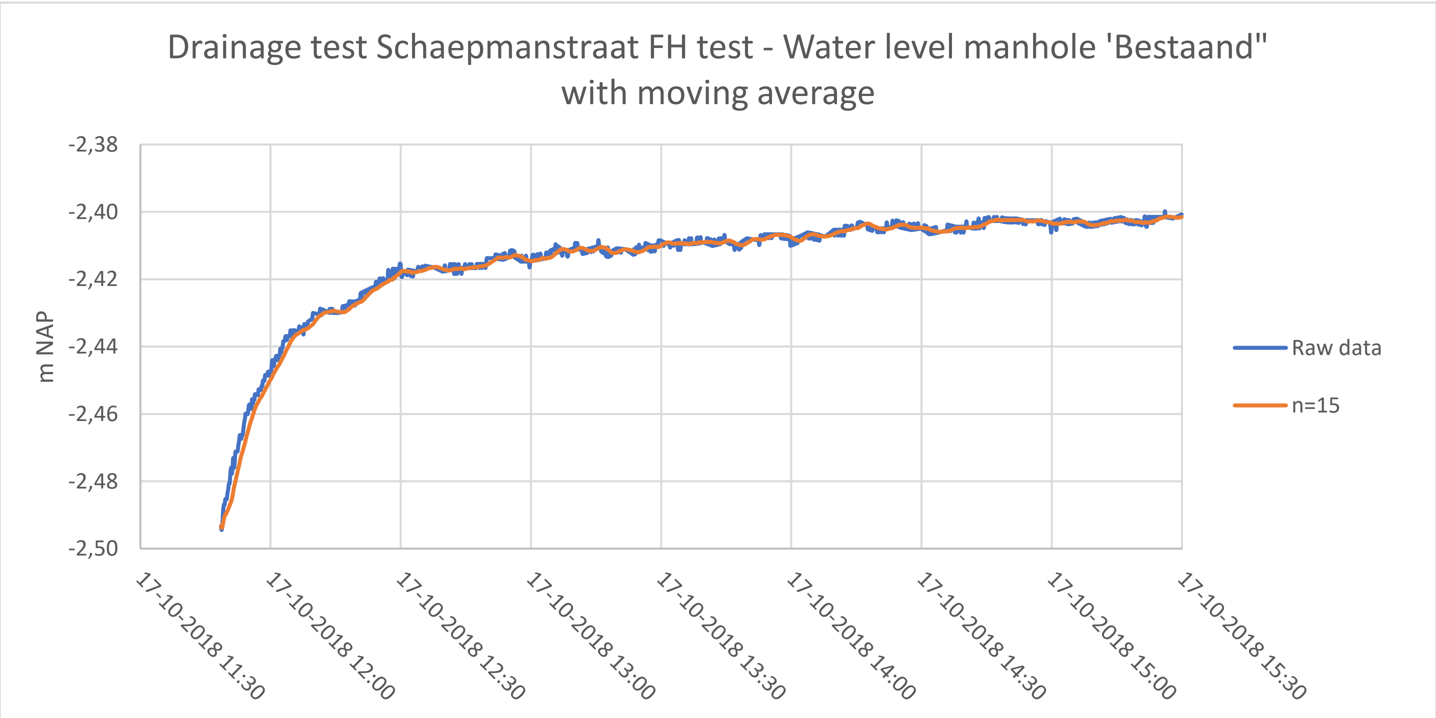


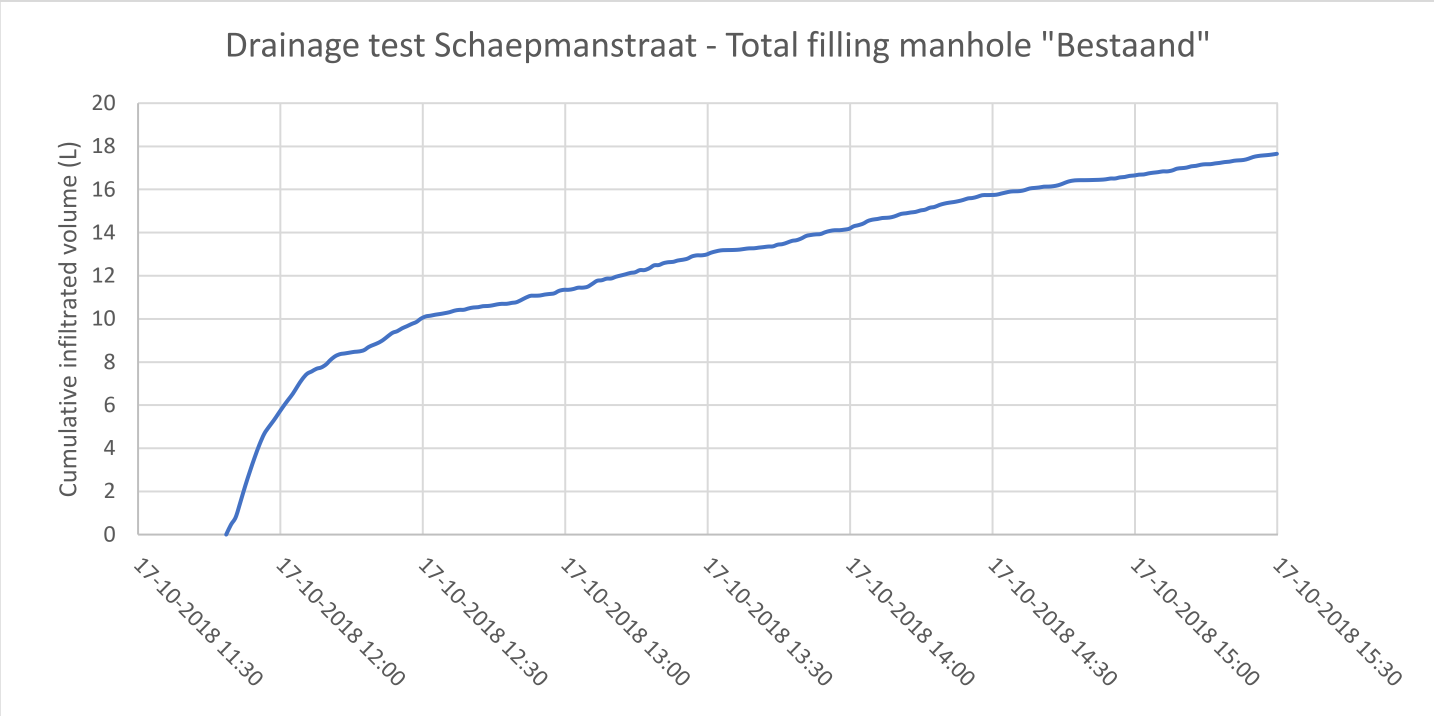



Schaepmanstraat Wareco monitoring well 515576
m NAP
-2,290
-2,300
-2,310
-2,320
-2,330
-2,340
-2,350
10-10-2018 00:00
11-10-2018 00:00
12-10-2018 00:00
13-10-2018 00:00
14-10-2018 00:00
15-10-2018 00:00
16-10-2018 00:00
17-10-2018 00:00
18-10-2018 00:00
19-10-2018 00:00
20-10-2018 00:00

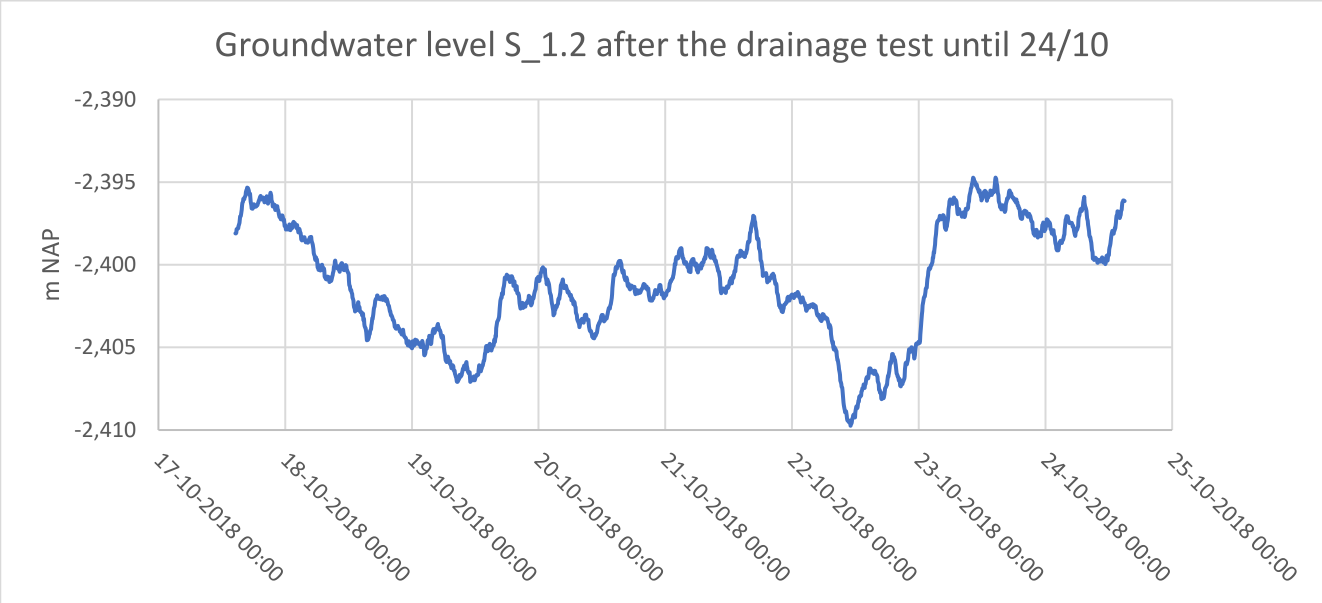
Appendix 6 Raw data drainage test Schaepmanstraat and applied moving average method

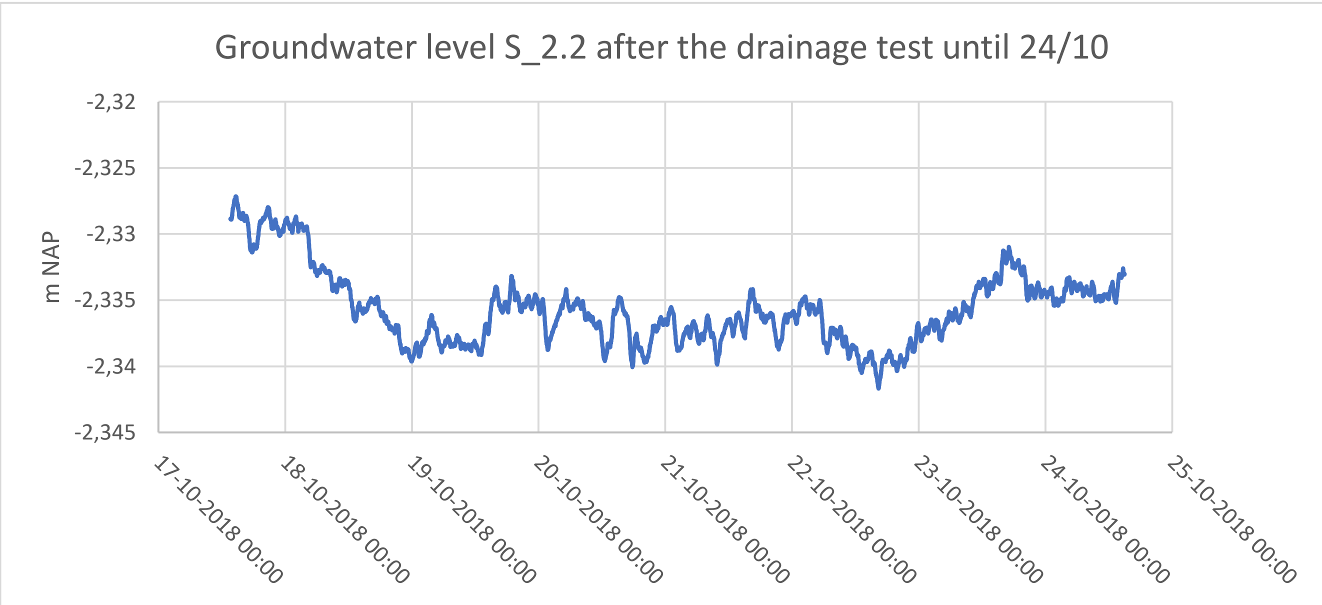




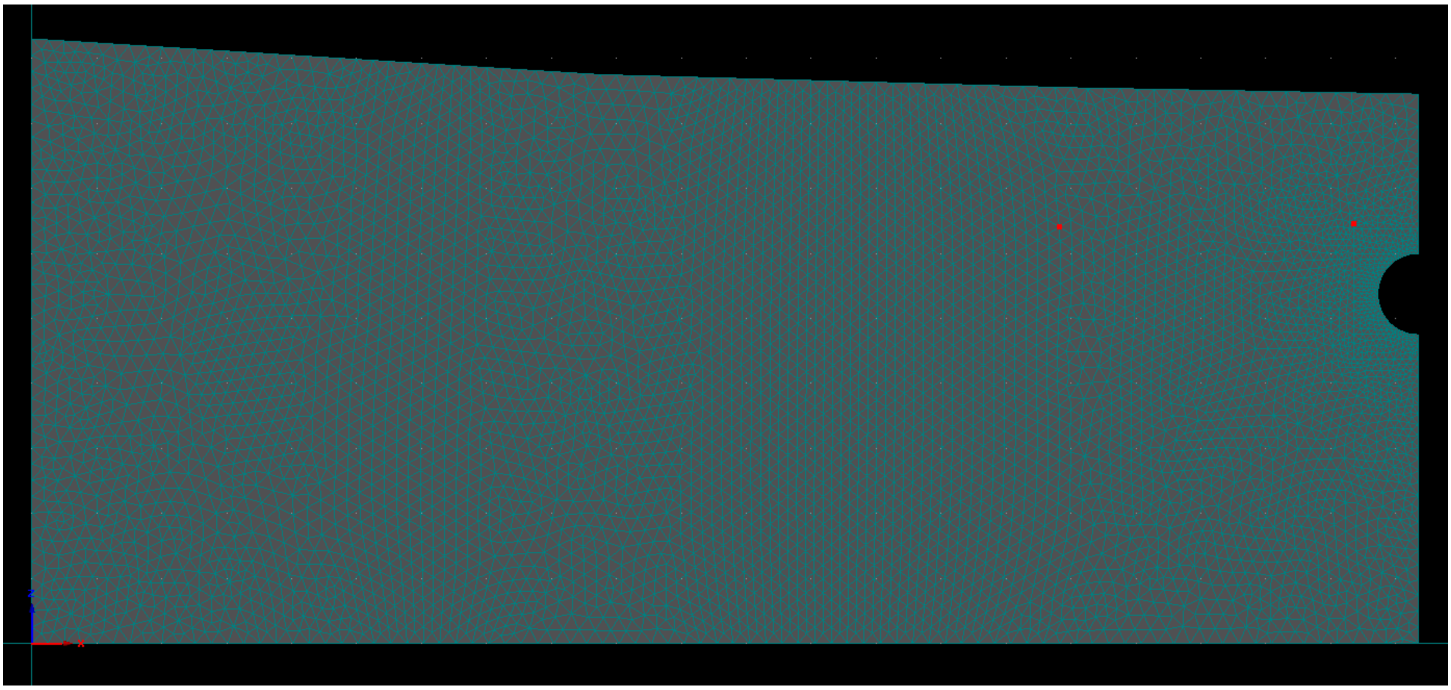

Drainage test Schaepmanstraat - Total filling manhole "Bestaand"
Cumulative infiltrated volume (L)
0
2
4
6
8
10
12
14
16
18
20
17-10-2018 11:30
17-10-2018 12:00
17-10-2018 12:30
17-10-2018 13:00
17-10-2018 13:30
17-10-2018 14:00
17-10-2018 14:30
17-10-2018 15:00
17-10-2018 15:30

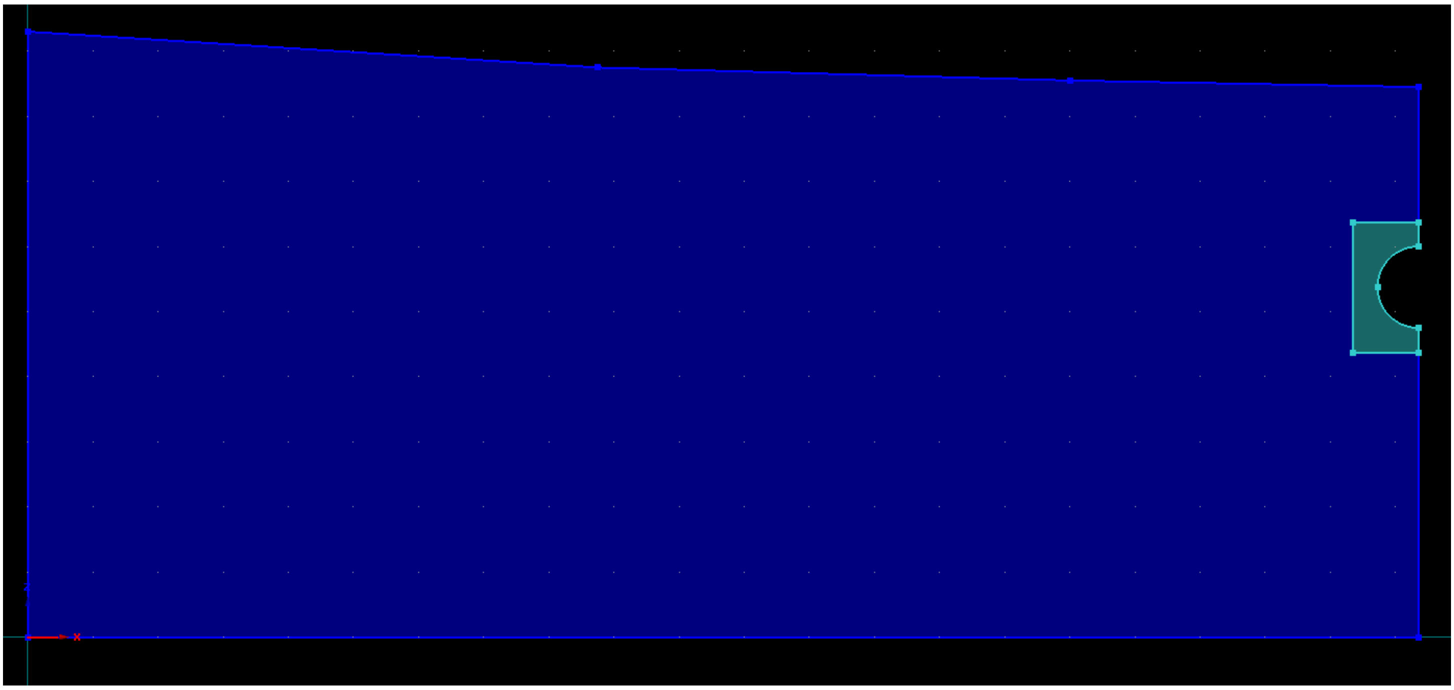
Appendix 7: Groundwater fluctuations Schaepmanstraat after drainage test at S_1.2 and s_2.2



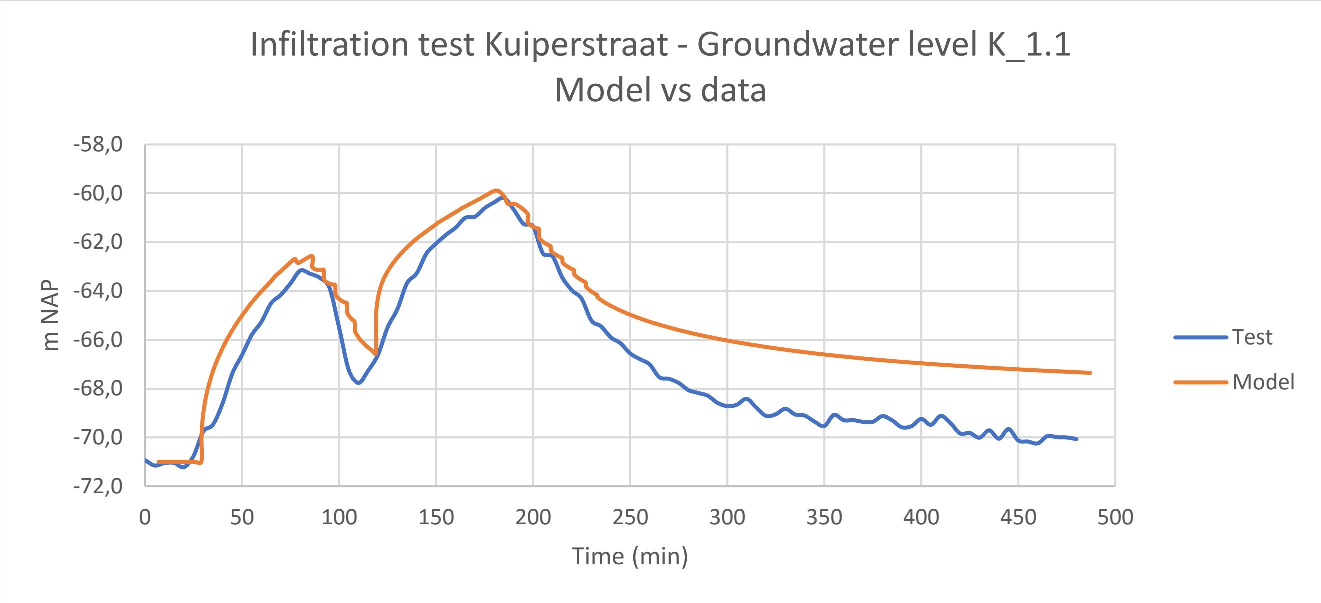


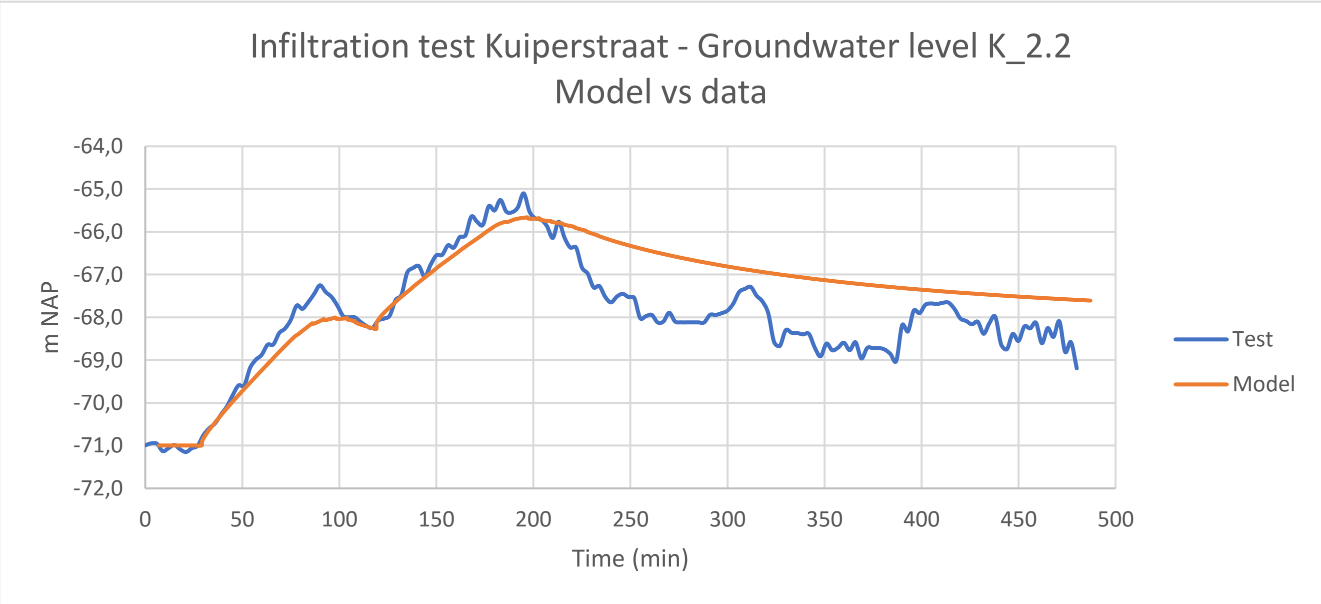
Appendix 8: FE Mesh of the Kuiperstraat model with the observation nodes and the material distribution of the model

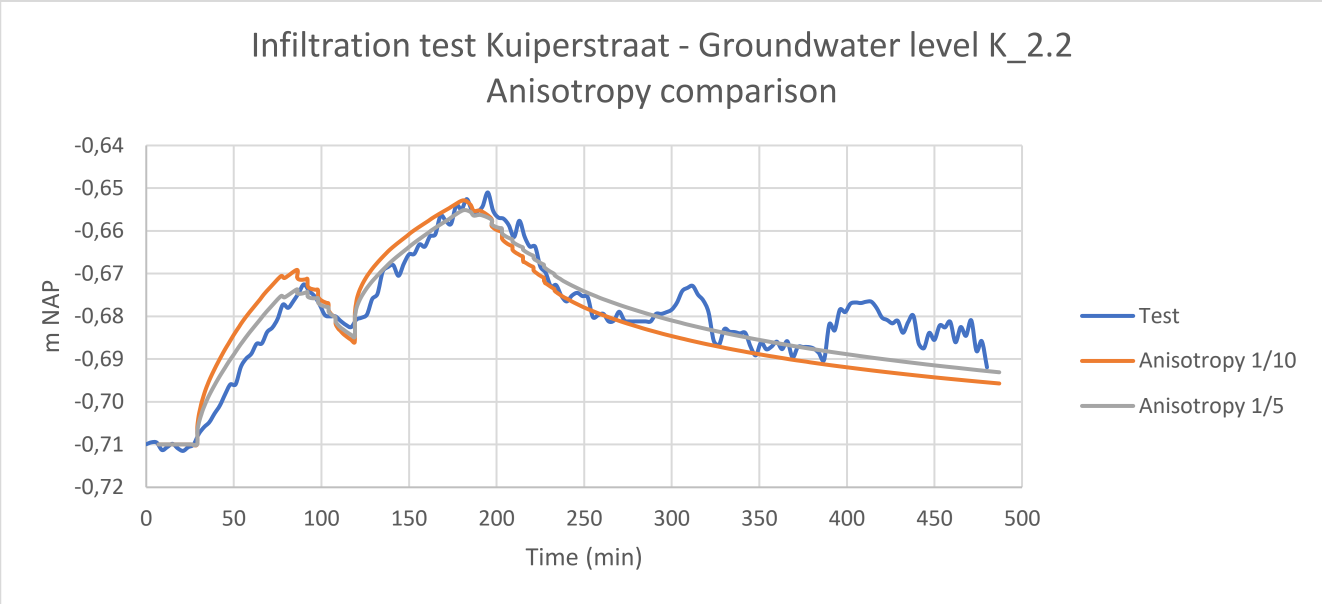




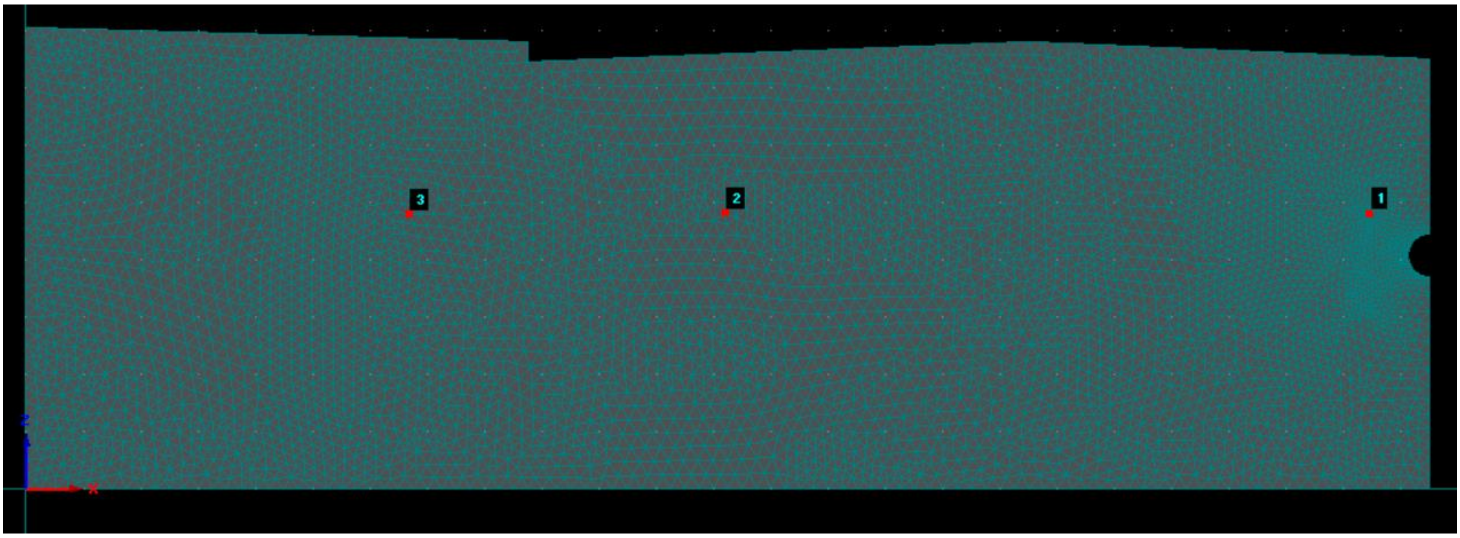
Appendix 9: Results of Hydrus simulation of infiltration test Kuiperstraat without addition of anisotropy and comparison of anisotropy values

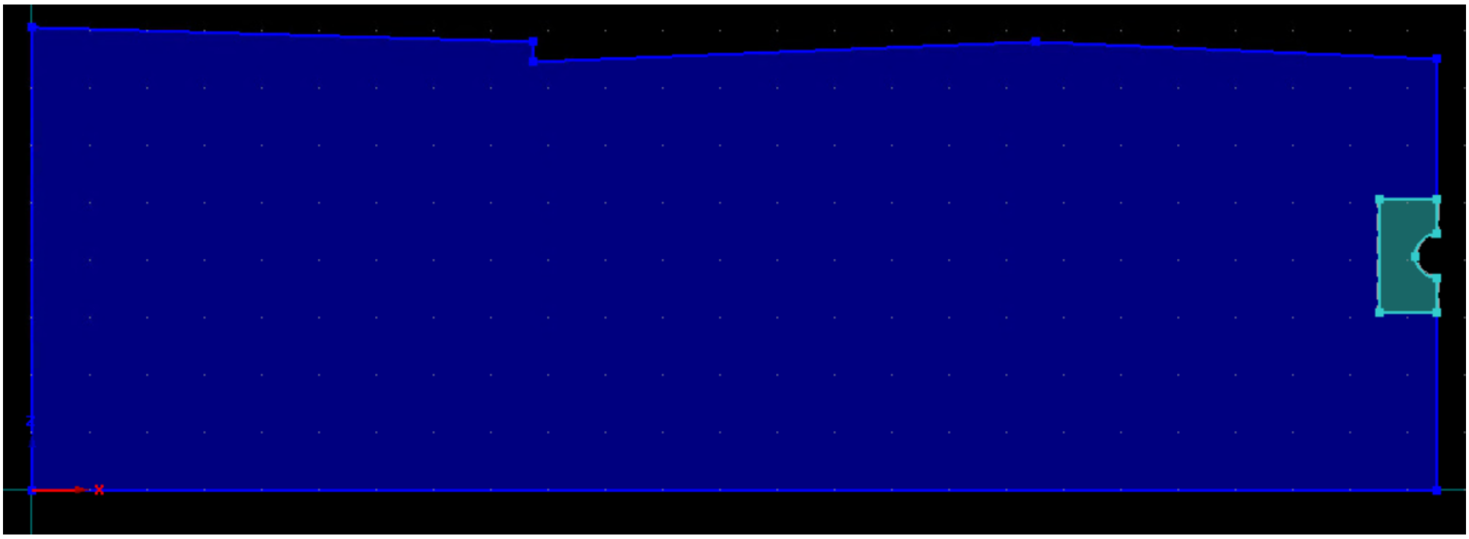





Infiltration test Kuiperstraat - Groundwater level K_2.2
Anisotropy comparison
m NAP
-0,64
-0,65
-0,66
-0,67
-0,68
-0,69
-0,70
-0,71
-0,72
0
50
100
150
200
250
300
350
400
450
500
Time (min)
Test
Anisotropy 1/10
Anisotropy 1/5

Appendix 10: FE Mesh of the Schaepmanstraat model with the observation nodes and the material distribution of the model





Appendix 11 Fluxes used in drainage test Kuiperstraat model and comparison of infiltration test Schaepmanstraat with and without drain envelope in model