Hongyang Wang

Controlling Urban Groundwater in Delta Areas

A case study at Turfmarkt, Gouda, the Netherlands

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Control Urban Groundwater in Delta Areas

A case study in Turfmarkt, Gouda, the Netherlands

By

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This thesis is confidential and cannot be made public until November 10, 2016.

TUDelft

Preface

The water resources management in the urban area is never an easy task since the complexity of the urbanization and artificial infrastructures alters the natural water cycle and makes the hydrological processes more complicated and less visible. Together with climate change and land subsidence, urban water resources management becomes more and more significant when it comes to investigating the potential interventions for the purpose of mitigating the negative impacts brought by them. I feel honored to have the privilege to be a part of the Gouda project brought forward by the Dutch Coalition “Stevige stad op slappe bodem (Solid city on soft soil)”, which select the historical city center of Gouda to be the initial location for the investigation.

Over the last thirteen months, I completed my internship and this MSc thesis at Hoogheemraadschap van Rijnland (Rijnland District Water Control Board), which I believe is one of the most memorable and unique experience during my Master study in the Netherlands. I think I am lucky enough to have a chance to work in a water-related governmental institution in the Netherlands, which has a totally different enterprise culture from where I come from. Since the project is based on the collaboration among multiple parties, I also have the opportunity to meet and work with many knowledgeable and amazing individuals from the government, companies, and other scientific institutes, such as Deltares, Gemeente Gouda, Wareco INGENIEURS, Royal HaskoningDHV, and Cyclus NV.

For this opportunity, I would love to express my thanks to Dr. Ir. Frans van de Ven and Drs. Mark Kramer, who are my supervisors in the university and the company. Dr. van de Ven gave me the chance to participate in this project at the beginning, and he always arranges meetings to follow up my progress and provides insightful recommendations on the following procedures. As my daily supervisor, Drs. Kramer is so kind and patient to me, and enthusiastic to help with my confusions, no matter how tiny they are. With the help from them, it keeps me on the correct track to finish my thesis without extra obstacle. At the same time, it motivates my interest and intention to work on urban water resources management in the future.

In addition, I want to show my best regards to Arianne Fijan (Gemeente Gouda), Ir. Marjon ten Hagen (Wareco), Ir. Andrét Jong (Royal HaskoningDHV), Jan Prinsen (Gemeente Gouda), Erwin Schreve (Cyclus), Ir. Neeltje Goorden (Deltares), Henkjan Faber (Rijnland), and Dolf Kern (Rijnland), for their valuable assistance and suggestions provided during my thesis period.

At the end, I would like to say thank you to my parents, who provide me moral and financial support remotely and unconditionally. It is not easy for me to study in a foreign country without parents around, and they always give me courage and strength to keep going. Eventually, I have been so far and I am so close to my Master Degree!

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Summary

In the light of the European Work Program DRS-11-2015, a Dutch Collision "Solid City on Soft Soil" was formed in order to mitigate disasters brought by subsidence and safeguard cultural heritage assets. As an initial approach, the Dutch Collision planned to explore and implement solutions for this challenge in the historic city center of Gouda and made this project as a paragon for other cities suffering from the similar problem. Multiple parties – governments, scientific institutes, private parties, and civilians – are interested and involved in this project.

The inner city of Gouda is an independent polder built on the Holland-Utrecht peat. The combination of subsidence and climate change increases the overall vulnerability, and a sustainable and efficient urban water management plays a significant role when it comes to the solutions. As the starting point for the Gouda project, this research focuses on the investigation of the fluctuation characteristic of groundwater under different influential factors (primarily precipitation, evapotranspiration, and sewage water) in two contiguous drainage areas – Nieuwe Haven and Centrum, where the impacts of climate change and land subsidence had been identified enormously negative.

In order to have a comprehensive understanding about the groundwater flow pattern in the research area, a new groundwater observation network was designed and implemented on 20th of May, 2016 to collect sufficient groundwater level data. The network took into consideration of the influential factors on groundwater, susceptible areas to groundwater fluctuation, and accessibility for subsequent validation and maintenance. Once the new observation network started to measure the groundwater levels properly, a field experiment was implemented to investigate the leaky extent of the back-stowed sewer system as well as the response of the groundwater to the variation of the sewage water levels. Both the time series and cross-section analysis among groundwater levels, sewage water levels, precipitations, and evapotranspiration were carried through. At the end, a preliminary construction and parameter analysis of a 2D layered finite-difference transient daily groundwater flow model were performed with the help of iMODFLOW v2.6.37. However, further calibration of the model is required to improve the model performance in the future.

The current results illustrate that the leaking extent of the back-stowed sewer system depends on the year of the construction and the pipe material. Generally speaking, the reactions of groundwater levels to the changes of the sewage water levels are up to the leaky extent of sewer pipes and the distance to the leaky system; groundwater in the vicinity of vegetation displays a similar variation pattern with the potential evaporation within a day; and groundwater in the unpaved and riparian areas are more sensitive to the precipitation.

The whole project is still in progress, and certain aspects of current investigation require further improvements. Additional recommendations related to data collection, model calibration, and investigation in other areas of the inner city of Gouda were given at the end of this thesis. Although there is no "one-size-fits-all" solution, due to the fact that seldom research has focused on the urban groundwater management related to land subsidence so far, the methods, procedures, experience and lessons accumulated in this research can still be considered as a guide or assistance for other areas or cities which are willing to make a change.

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1 Introduction

On December 11th, 2013, a new search topic named "DRS-11-2015: Disaster Resilience & Climate Change topic 3: Mitigating the impacts of climate change and natural hazards on cultural heritage sites, structures and artifacts" was published and embodied in the European Work Program 2014 – 2015. It was underlined exclusively that the cultural heritages in historic cities are becoming more and more vulnerable owing to natural decay, human impacts, environment and climate changes (*European Commission Decision, 2015*).

In the light of "DRS-11-2015", a Dutch Coalition "Stevige stad op slappe bodem (Solid city on soft soil)" was formed in order to alleviate damages brought by subsidence and construct sustainable and climate-resilient cities for cultural heritage assets. Accordingly, the subsidence progress is at an average rate of 10 to 20 cm per century in the Netherlands, which may lead to potential pecuniary loss around 40 billion euros, not only in the aspect of cultural heritages, but also of economy, tourism, public health, urban water management, spatial network planning, infrastructural facilities, and governance (*European Commission Decision, 2015; Coalitie in Dutch, 2015*). As an initial approach, the Dutch Coalition planned to explore and implement solutions for this challenge in the historic city center of Gouda and made this project as a paragon for other cities suffering from similar problems. Multiple parties – governments, scientific institutes, private parties, and civilians – are interested and involved in this project.

Generally speaking, "vulnerability" refers to the propensity of a system to be adversely affected when it is exposed to shocks, stresses, and disturbances. It is usually evaluated by the sensitivity to harm and capacity to adapt (*De Graaf et al., 2009; IPCC, 2014*). From the current circumstances in the inner city of Gouda, it can be said that the vulnerability of inner city to climate change and land subsidence is in a relatively high degree. The combination of climate change and subsidence makes it prone to be flooded on the street and in the houses during long and high-intensity rainfall events. However, in dry and hot weather periods, evapotranspiration leads to relatively low groundwater level, which accelerates the rates of peat oxidation and compaction, as well as put on a line of "dry rot" of wooden foundations. In addition, subsidence causes uneven settling of buildings and damages on underground infrastructures (e.g., sewer system). The leaky sewer system has negative impacts on groundwater in both quality and quantity aspects. And it will take lots of money and time to fix the problems. The present water management in the inner city is insufficient, thus, sustainable and efficient countermeasures are required to build the resilience to climate change and land subsidence.

The research focused on two contiguous drainage areas in the inner city, namely, Centrum and Nieuwe Haven. Because these two areas are comparatively more representative than other areas. The boundary between them is a canal called Turfmarkt, the freeboard of which is no more than 5 cm, and the water level is completely dependent on the pumping control and capacity. The street Nieuwe Haven used to be a harbor. It was abandoned and filled with sand in 1980's. The old harbor quays still exist underneath the surface, and the sewer systems are rather complicated, involving back-stowed system ("Opgeboeid" in Dutch), pumping system ("Bemalen" in Dutch), and storm water system (RWA, "Regenwaterriool" in Dutch). In accordance with my previous internship research, which applied the program "Menyanthes" to indicate the qualitative interactions among precipitation, evapotranspiration, sewage water, and groundwater levels measured at the existed observation wells in the whole inner city, the groundwater levels in the Nieuwe Haven area have a rather closer relationship with sewage water levels, while those in the Centrum area is more influenced by precipitation and evapotranspiration. Nevertheless, the quantitative models to simulate the interactions among them are still largely absent from investigation and comprehension (*Coalitie in Dutch, 2015*).

From the present point of view, it seems that the negative consequences caused by land subsidence are so far-reaching. However, it would be hazardous in a long run if no sustainable and efficient measures are going to implement soon. Therefore, the Dutch Coalition wants to lead the way in terms of public awareness-raising and possible counter-measures in the research area. Although there is no “one-size-fits-all” solution, the experience and lessons accumulated from this project could still provide assistance and serve a useful function at the international level. Since available groundwater management related to subsidence in the urban area has been rarely studied so far, the research methods and procedure, as well as the collaboration between stakeholders and Coalition would be an example for other cities which are willing to make a change.

This research can be considered as a starting point for the realization of good policy decisions on controlling groundwater level and attenuating the hazards brought by climate change and land subsidence, as well as an alternative strategy for surface water level regulations and sewer system renovation in spite of consuming excess energy and money. In the following parts of the report, it will focus on identification of present problems, the design of a groundwater level monitoring network with a more intensive distribution, data analysis, and an initial setup of local groundwater flow model for the research area. The work for the whole project has not been completed, and further investigation will be required. Nevertheless, this research will give the first impression and understanding on how groundwater level fluctuates quantitatively under different external influential factors. According to the results from this research, some useful recommendations can be given for further investigations.

2 Problem Statement

Climate change can be considered as one of the most intricate issues needed to be alleviated for the last several decades on a global scale. It is defined as the properties of climate are identified to change in the mean values and/or variability through statistical analysis on the basis of a relatively long period of observation, and which is as a result of natural processes and direct or indirect anthropogenic activities (*IPCC, 2014*). The impacts of climate change on the aspects of the environment, economy and society have already been aware of in Europe and other parts of the world (*EEA, 2012; IPCC, 2014*). And it is important to take actions to mitigate those negative influences sustainably and efficiently at the local level, which is also emphasized in the 1987 Brundtland Report and at the 1992 UNCED (United Nations Conference on Environment and Development) in Rio de Janeiro, Brazil (*Bulkeley & Betsill, 2003*). According to the reports from Intergovernmental Panel on Climate Change (*IPCC, 2014*), European Environment Agency (*EEA, 2012; 2015)*, and *Government of the Netherlands (2016)*, low-lying areas like the Netherlands are confronted with hazards such as extreme weather events, flooding, coastal erosion, infrastructure damage, and human health threat, as consequences of climate change.

Based on data and observations, the total amount of annual precipitation was increased around 20% in the Netherlands over the last century (*Coalitie in Dutch, 2015*). The frequency of extremely intense rainfall is expected to be higher during autumn and winter, leading to the rise of river flow and menace to the life and property safety of inhabitants living in the riparian areas (*Government of the Netherlands, 2016*). At the same time, due to global warming, the average temperature in the Netherlands has risen by 1.7 $°C$, which is approximately two times higher than that around the world (*Coalitie in Dutch, 2015*). Thus, summer is likely to be drier and hotter. The changes in precipitation and temperature pattern are transforming the hydrological system concurrently, which goes a step further to impact urban water system and resources in both quantitative and qualitative respects (*IPCC, 2014*).

The inner city of Gouda is an independent polder built on the Holland – Utrecht peat, where has a massive accumulation of decayed vegetation from about 3500 BC. Both of the layers above and beneath the peat are sandy and clayey sediments from rivers, and Pleistocene sand and gravel are embedded in the bottom layer (*Klaassen, 2008; Van Winsen et al. in Dutch, 2015*). On account of the influence of the river Hollandsche Ijssel, a thicker deposit bed was formed under the peat in the south-eastern part, which results in a comparatively lower elevation in the north-western part of the inner city. The research areas Nieuwe Haven and Centrum are located in this region with lower surface levels (see *Figure 1*). Land subsidence is a common phenomenon occurring in the west and north of the Netherlands since the last century, primarily owing to the oxidation of peat and compaction of clay (*Langevin et al., 2004; Oude Essink & Kooi, 2011*). Accordingly, the changes in the elevation of instrumentation and surface water level around Turfmarkt indicate the average settlement rate is about 2 $mm/year$, which is slow and barely visible. Nevertheless, the risk of hazards increased by subsidence cannot be negligent, for example, flooding, uneven settling, cracking of sewer system, access to premises becoming too high or too low, as well as financial loss and accidents imperiling public health from dysfunction of underground cables and pipes (*Coalitie in Dutch, 2015; De Graaf et al., 2009; Den Nijs in Dutch, 2015*).

The foundation types in the inner city are rather various than unique, but in general, they can be assorted into three classifications, “op staal”, “op kleef”, and “op stuit” in Dutch, respectively (*Schot & Oosterhoff in Dutch, 2013; Van Winsen et al. in Dutch, 2015*). The foundation “op staal” was substantially applied for constructing light buildings since the 16^{th} century and prevailed until the 20^{th} century in Gouda. However, the measurement and record data from Ingenieursbureau Hopman indicate that the buildings constructed on “op staal” sank around 1.5 $mm/year$ on average over the last 32 years, in correspondence with land subsidence. The foundation “op kleef” started to be used in Gouda for heavier buildings like the monastery since the 15th century. For the sake of enhancing the weight capacity, longer and more closely spaced wooden piles were placed underground firmly and touched off the compaction of soil. And the foundation “op stuit” with wooden piles was applied widely until concrete piles emerged on a large scale after the World War II (*Klaassen, 2008*).

Figure 1 Surface elevation (ahn3_05_dtm) of the inner city of Gouda (http://nationaalgeoregister.nl/geonetwork/srv/dut/search#|94e5b115-bece-4140-99ed-93b8f363948e)

In order to prevent the decomposition of wooden piles caused by fungi, the head of wooden piles should always be located under the groundwater level (*Klaassen, 2008; Van Winsen et al. in Dutch, 2015*), yet the impact of climate change makes it harder to achieve this requirement. *Suess (in Dutch, 2007)* believed that the upper heads of wooden pile foundation of Turfmarktkerk might experience the risk of decay during dry seasons for the top of the pile (probably -0.7 $m\ NAP$) will be exposed to air when the measured groundwater levels could only reach -1.6 $m\ NAP$. What is more problematic is a difference in foundations of two properties sharing a common wall as this gives rise to uneven settling. The distortion of building structure constitutes hidden safety hazard to human life. It has been presented that there is a palpable relationship between foundation type and subsidence (*Coalitie in Dutch, 2015; Van Winsen et al. in Dutch, 2015*). However, the knowledge on foundation types in the inner city is primarily on the basis of the available recording material and visual inspection, which is incomplete and limited. And it is not practically and financially possible to investigate the foundations and potential risks in the research area comprehensively (*Den Nijs in Dutch, 2015*).

On the basis of the available research, threateningly high groundwater and surface water levels have been identified near Turfmarkt since they are merely a few centimeters below the ground level (*Coalitie in Dutch, 2015; Schot & Oosterhoff in Dutch, 2013*). For the moment, the surface water level completely depends on technology control. There are three pumping stations (see *Figure 2*), Gemaal Hanepraai, Gemaal Mallegat, and Gemaal Spruit, to maintain the water level at around -0.72 $m\ NAP$. However, this is far from enough. Owing to the fact that half of the pavements in the research area are not connected to the combined sewer system, during the extreme rainfall events, rainwater falling on the streets discharges to the canals straightly, thus, surface water level responds quite drastically and flows back to streets and houses instead (*Coalitie in Dutch, 2015; Zandee in Dutch, 2009*). Not only does high groundwater level aggravate this hazard, it will cause other impairments as well, for example, reduction of the foundation bearing capacity, structural uplift damage, “wet feet” for vegetation, and extra moisture in the premises, emerging fungi and mold and threatening public health directly (*Coalitie in Dutch, 2015; Van de Ven, 2016*).

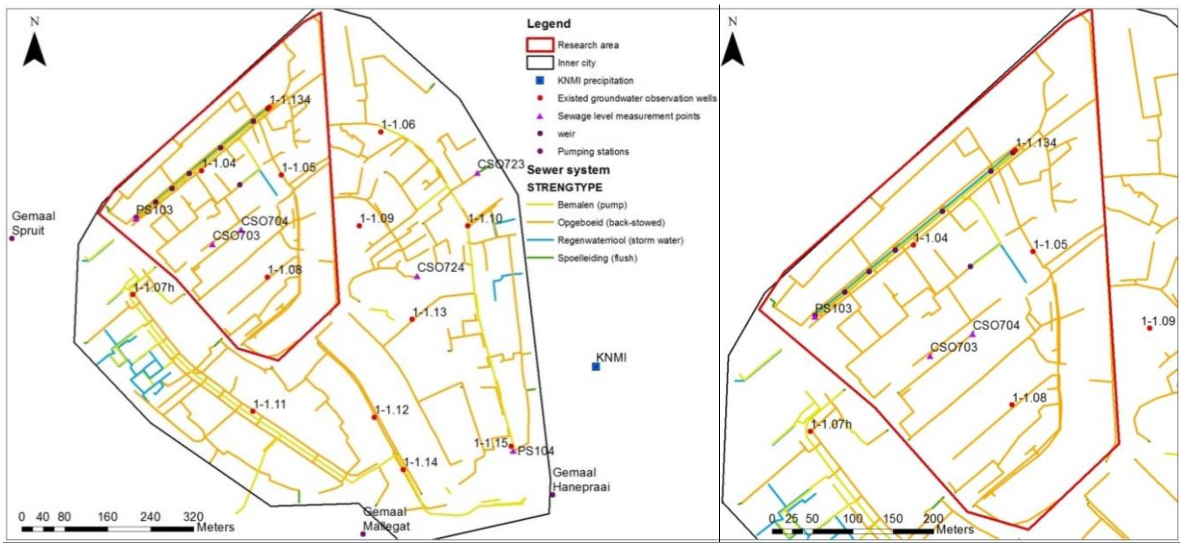


Figure 2 Scheme of sewer system, existed groundwater monitoring wells, sewage water measurement points, KNMI precipitation station, and surface water pumping stations

Nevertheless, stubbornly pursuing low groundwater level will become problematic as well since it will lead to degradation of wooden poles due to dry heads and accelerate the rate of subsidence caused by the oxidation of peat and extra effective stress in the soil matrix, especially during the dry period (*Suess in Dutch, 2007*). On the northwest side of Turfmarkt, where the drainage area Nieuwe Haven is located, the groundwater levels at monitoring wells 1-1.04, 1-1.05, and 1-1.134 (see *Figure 2*) are considerably (about 0.2 to 0.3 m) beneath the surface water level. Even one of them (Well 1-1.05) maximizes to 0.57 m lower (*Schot & Oosterhoff in Dutch, 2013*).

The drainage area Nieuwe Haven is one of the most significant drainage areas in the inner city. The sewer system consists of the back-stowed system (“Opgeboeid” in Dutch), pumping system (“Bemalen” in Dutch), storm water system (RWA, “Regenwaterriool” in Dutch), and flushing system (“Spoelleiding” in Dutch) (see *Figure 2*). The RWA gather rain water from the street and then they are pumped out by pumping station (“PS 103” in *Figure 2*). And the back-stowed sewer lines are connected to households and collect waste water from there. Afterward they will flow into the pumping pipes which are linked to the pumping station “PS 103” as well. The junction points between the back-stowed system and pumping system are the locations where weirs are situated (see *Figure 2*). The back-stowed sewer system was built rather old, and some parts of it can be dated back to 1869. Moreover, the flushing system is used to scour the sewer system preventing the reduction of storage room because of waste material sedimentation.

On account of the high groundwater level in history, all the pipelines were constructed under the ordinary groundwater level (*Coalitie in Dutch, 2015*). Theoretically, the sewer system in the Nieuwe Haven area should discharge sewage water in its own area as well as its secondary drainage area Centrum through the pumping station “PS 103” to the wastewater treatment plant. But, in line with a survey carried out by *Zandee (in Dutch, 2009)*, it indicated that approximately 80% of the dirt and pollution loads emitted from the sewer system via CSOs (Combined Sewer Overflow, see “Weir 703” and “Weir 704” in *Figure 2*). At the same time, due to subsidence and lack of regular maintenance, some back-stowed sewer lines became leaky and cracked extensively, especially the pipelines constructed behind houses made it inconvenient to get essential servicing. On the basis of previous field research and modeling analysis, it has been proven that the interaction between groundwater and sewage water is comparably intense in the Nieuwe Haven area. And the groundwater level is under the impact of the performance period of pumping stations (*Den Nijs in Dutch, 2015; Zandee in Dutch, 2009*). Thereby, the backup-stowed sewer system is basically functioned as a groundwater drainage system and the groundwater level is maintained by the weirs and fluctuated at an average level of -0.8 $m\,NAP$ (ranging from about -0.6 to -1.1 $m\,NAP$). Furthermore, during the sewer cleaning project conducted in 2011, it was revealed that there were at least 12 locations in the sewer system having problems such as blockage, flat tubes, and breaches. The impact of leaky sewer system affects groundwater in both quantitative and qualitative respects (*Schot & Oosterhoff in Dutch, 2013*).

In consideration of the vulnerabilities discussed above, a sustainable and efficient management of urban water resources (surface water, groundwater, and sewage water) plays a significant role when it comes to the mitigation strategies for land subsidence and climate change. The management of water levels needs to be more accurate in the future, which signifies more capital and labor required. At present, the qualitative interaction between surface water, groundwater, and sewage water is generally recognized. Whilst, the quantitative relationship is largely absent from investigation and comprehension (*Coalitie in Dutch, 2015*). The in-depth insight into interactions between these subsystems is needed in quantitative terms from aspects of scope, timing, and impact of vulnerabilities. None but when we fully understand the effect mechanisms of influential factors on groundwater level, other suitable alternative strategies could be identified to improve the current situations.

3 Aims of Thesis

For the sake of alleviating the negative impacts of land subsidence and climate change along the canal Turfmarkt and in the drainage area Nieuwe Haven, this research will investigate the existing groundwater regime, taking into consideration of lithology, as well as interactions with the sewage water system, surface water, and local weather conditions (primarily precipitation and evapotranspiration). In addition, it will provide some feasible recommendations on interventions to improve the groundwater regime on the basis of rational and scientific analysis on the whole urban water system. Thereby, the main research question is:

“How to investigate the fluctuation characteristics of groundwater in a local urban area such as Turfmarkt, Gouda, to provide preconditions for the purpose of mitigating the vulnerabilities brought by climate change and subsidence?”

In accordance with the research question, the following aims are expected to be achieved in the thesis:

- Establish a directed and representative observation network on groundwater level;
- Carry out field experiments on interactions between groundwater and sewage water, surface water, evapotranspiration, and precipitation;
- Utilize a regional groundwater flow model to simulate the local groundwater flow patterns and ameliorate understandings of groundwater system;
- Bring forward of recommendations for further investigations.

4 Review of Literature

4.1 Subsidence in delta areas – present circumstances, causes, damages and countermeasures

Across the globe, delta areas are intensively prevalent for agriculture, residence, and economic activities, owing to highly fertile soil with organic matters (e.g., peat) and adjacent locations to estuaries of primary navigable watercourses. In the Netherlands, roughly 1/3 of the 17 million inhabitants live in these delta areas, and those used in agriculture are basically for dairy farming, where require a relatively low surface water level to guarantee the growth of grass in the meadows (*Boersma & van Lenteren, 2015; Hoogland et al., 2012; Querner et al., 2012*). Nevertheless, approximately half a billion inhabitants living in the delta regions become more and more vulnerable to flooding and other inevitable hazards as the consequences of climate change and land subsidence (*Boersma & van Lenteren, 2015; Schmidt, 2015; Syvitski et al., 2009*).

Land subsidence is still an underrated and less advertent problem comparing to sea-level rise. Accordingly, the sea level is estimated to rise by 320 $mm/century$, yet delta places where subsidence occurs severally like Jakarta, Indonesia can sink about 50-100 $mm/year$ on average (*Abidin et al., 2015; IPCC, 2013*). It would only take a considerably short time (3 – 6 years) for the effect of subsidence to outpace that of sea-level rise. One of the conceivable reasons is the sinking rates in the most delta areas are slow and hardly noticeable. In addition, the extent of subsidence is various and equivocal on a global scale, while sea level rise can always give an explicit value (*Erkens & Sutanudjaja, 2015*). Nonetheless, the awareness of subsidence for local governments and the public should not be raised until there is a catastrophe happened in the future. Because the process of land subsidence is irreversible, the state-of-the-art countermeasures are expensive, and the outcomes of these measures could only be testified in a long-term observation (*Boersma & van Lenteren, 2015; Erkens, 2016; Schmidt, 2015; Vázquez-Suñé et al., 2005*).

4.1.1 Present circumstances

Nowadays, land subsidence happens on a global scale, striding across Asia, Europe, and America. The maximum measured rates of settling vary from 6 mm per year (in Kolkata, India) to 205 – 250 $mm/year$ (in Tehran Basin, Iran) (*Galloway & Burbey, 2011*). Some delta areas with peaty soil have been recorded with subsidence, for example, Sumatra and Kalimantan, Indonesia in Asia, the west and north of the Netherlands, East Anglia, UK, Po Valley, Italy, and North-German coastal plain in Europe, as well as Mississippi Delta, Everglades, and Sacramento-San Joaquin River Delta in the US (*Boersma & van Lenteren, 2015*).

The regions with the most critical problem of subsidence are basically located in Asia, where most developing countries are centered with high speed of population growth. And the majority of the deltas there are dominated by agriculture and the primary exporters of rice or other crops in the world (e.g., Mekong Delta, Vietnam, and Ganges–Brahmaputra Delta, Bangladesh) (*Chand, 2016; Schmidt, 2015*). As mentioned before, Jakarta is sinking at an average rate of 50 – 100 $mm/year$. In the northwest of Jakarta, the rate can even achieve 200 $mm/year$. Based on the measurement by the GPS (Global Positioning System), the maximal value in Jakarta was 250 $mm/year$ during the period from 1997 to 2008 (*Abidin et al., 2009*). If there is no additional strategy to implement in order to alleviate subsidence, the city could end up with sinking to 6 m in this century (*Schmidt, 2015*). The Mekong Delta is facing the problem of subsidence with a rate of 16 $mm/year$ as well. On the southern tip of the delta, where Cà Mau province is located, the subsidence accelerates almost to double (about 30 $mm/year$) (*Erban et al., 2014*). Another country suffering subsidence is Bangladesh. In the eastern part of the Ganges-Brahmaputra Delta, subsidence rate was simulated between 0 and 18 $mm/year$ from 2007 to 2011 by using InSAR (Interferometric Synthetic Aperture Radar) (*Higgins et al., 2014*).

Not only in Asia but the land subsidence occurring in America and Europe should also be taken seriously. The first record of subsidence is in the Netherlands since the polders were constructed in the 9th century. It was documented that the rate of sinking during the 8th to 10th centuries was 0.04 to 0.1 inch (approximately 1.02 to 2.54 mm) per year (*FESSRO, 2012*). Another delta place with detailed information on subsidence in Europe is Po Delta, Italy. The highest sinking speeds of land surface in the central part of the Delta were recorded to be 250 $mm/year$ from 1951 to 1957, 180 $mm/year$ in the period of 1958-1962, 33 $mm/year$ during 1962 and 1967, and 37.5 $mm/year$ in 1967-1974 (*Fabris et al., 2014*).

Furthermore, the records in the US are available in some regions as well. Before the mid-1970s, historical measurements in the San Joaquin Valley and San Jose in the Santa Clara Valley showed that these two places sank nearly 30 feet (about 9.14 m) and 14 feet (about 4.27 m), respectively (*Ingebritsen et al., 2000*). Additional historical measurement in the Sacramento-San Joaquin River Delta from 1922 to 1981 indicates the subsidence rate was 1 to 3 inches (roughly 25.4 to 76.2 mm) per year, and logarithmic model predicted it would be between 27 $mm/year$ and 42 $mm/year$ in 1980 and 1990 (*Rojstaczer et al., 1991*). According to a recent research, the subsidence of the peat soil in the Delta is similarly equivalent to 0.4-0.6 inches (approximately 10.16 to 15.24 mm) per year (*FESSRO, 2012*). What is more, the subsidence also occurs in Mississippi Delta and urban area of New Orleans with estimated rates of 3 $mm/year$ and 6.4 $mm/year$, separately (*Dixon et al., 2006; Meckel et al., 2006*).

4.1.2 Causes

The causes of land subsidence are principally owing to anthropogenic activities, comprising land reclamation, artificial drainage, overexploitation of groundwater resources, urbanization and industrialization, as well as construction of hydraulic projects (*Boersma & van Lenteren, 2015; De Mulder et al., 1994; Galloway& Burbey, 2011; Ingebritsen et al., 2000; Langevin et al., 2004; Törnqvist et al., 2008*). Among all these drivers, decomposition, shrinkage, and compaction of Holocene coastal organic-rich deposits (notably as peat) are considered as the dominant ones, especially in the delta regions where land reclamation and artificial drainage commenced and polders were constructed (*Boersma & van Lenteren, 2015; Hoogland et al., 2012*; *Ingebritsen et al., 2000; Langevin et al., 2004; Querner et al., 2012; Törnqvist et al., 2008*). For example, in the west and north of the Netherlands, inhabitants drained the swamps and deep lakes and lowered the water tables factitiously to build and live on them for the past centuries. *Schothorst (1977)* found out that 65% of subsidence emanated from shrinkage and oxidation of peat above the groundwater level in the western Netherlands. And *Hoogland et al. (2012)* confirmed that the predicted subsidence rate on the peat area (8 $mm/year$) was nearly 11 times faster than that without peat (0.7 $mm/year$).

Before any artificial drainage proceeded, the peat soil was submerged by water and the net accumulation of organic matters prevailed. Once the water level was lowered by the human, the peat was exposed to air and microorganisms started to oxidize them to carbon dioxide (CO_2) or nitrogen dioxide (NO_2) and released to the atmosphere. Due to oxygen "bearing the carbon away", the peat layer became flimsy and compact, thus, land subsidence occurred (*Schmidt, 2015*). While the land is sinking, the surface water level needs to be reduced for the ranching and farming polders, such as Sacramento-San Joaquin River Delta and Polder

Groot-Mijdrecht, the Netherlands, where a certain thickness of the unsaturated zone is required for vegetation growth. A larger mass of peat are brought into contact with air and oxidized, the rate of subsidence accelerated further (*Boersma & van Lenteren, 2015; Hoogland et al., 2012; Ingebritsen et al., 2000*). It has been studied that the main driving force for the subsidence happened in the New Orleans is artificial drainage instead of excess loading (*Törnqvist et al., 2008*).

Another influential characteristic of peat-related subsidence is that peat could swell and shrink in accordance with water availability. During the dry summers, peat could contract for several centimeters, causing temporary sinking; and in the extremely wet winter, it would swell so that the surface seems to be elevated a little bit (*Querner et al., 2012*). Nevertheless, along with the gradual disappearance of peat due to oxidation, the effect of water swelling by peat becomes more invisible and less obvious. In addition, in the warmer areas such as the Sacramento-San Joaquin River Delta, California, high-temperature speeds up the decay reaction of organic matters, hence, the problem of subsidence is more critical (*FESSRO, 2012*). *De Mulder et al. (1994)* summarized that 40 – 60% of settlement bound up with peat would take place in the first three years of reclamation, and the majority part of subsidence could be completed in the next 14 years.

Apart from peat oxidation, overexploitation of subterranean natural resources including fresh groundwater, mineral, and fossil fuels (mainly crude oil and gas) is another all-pervading reason for subsidence, especially in those mega-cities located close to seacoast like Jakarta, Ho Chi Minh City, Vietnam, Bangkok, Thailand, and other coastal deltas like Ganges-Brahmaputra Delta, Mekong Delta (Cà Mau province), Sacramento-San Joaquin River Delta (San Joaquin Valley and Santa Clara Valley), and Po Delta (*Boersma & van Lenteren, 2015; Carminati & Martinelli, 2002; Erkens, 2016; Galloway et al., 1999; Galloway & Burbey, 2011; Schmidt, 2015*). These liquid and gas flowing through the pores of soil are functioned as "pillars" to bear the weight of land. In case they are extracted, the underlying deposits lose their support, the pore water pressure is abated, and effective stress is augmented. Therefore, the sediments as soft and compressible as clay and peat will be deflated and shrink like a "dried sponge", subsequently subsidence occurs (*De Mulder et al, 1994; Schmidt, 2015*).

In the interest of catering to the high demand of groundwater for industrial, agricultural, zootechnical, and domestic purposes, the amount of groundwater extraction nearly quadrupled in Emilia-Romagna Region, Po Plain since the second half of the 20th century. In Jakarta, the daily extraction of groundwater (both with and without a license) is estimated to be as high as 180 – 250 million m^3. And in Cà Mau province, over 1,000,000 wells have been drilled for water-intensive industries, for instance, shrimp farms (*Erban et al., 2014*). The water pressure is diminished because of excessive pumping of groundwater, encroachment of seawater in delta areas is amplified, and the salt ions lead to chemical reactions with soil sediments, therefrom the situation of land subsidence is apt to exacerbate.

The compaction of sediments is also contributed by incrementally developing extent of urbanization and industrialization, along with over-extraction, which happens in Ganges-Brahmaputra Delta, for example (*Schmidt, 2015*). The sheer weight of urban area is increased with the expansion of urban and industrial infrastructure simultaneously, and it presses the surface to sink. The impervious roofs and pavements preclude precipitation and surface water from percolating down to and recharging the groundwater. At the same time, some underground infrastructures can take effect as drains (e.g., leaky sewer pipes) or barriers impeding natural flow (*Vázquez-Suñé et al., 2005*). In addition, the construction of reservoirs and flood defense engineering – dams, levees, and embankments – prevent natural sedimentation to replenish the polders or lands and create a "collective bowl" for deluge during the rainy seasons (*Schmidt, 2015; Syvitski et al., 2009*).

4.1.3 Damages

As stated above, the menaces from land subsidence are becoming more and more exigent in many respects related to society, economy, infrastructure, as well as living environment. At present, the most urgent damage caused by subsidence is inundation. Approximately 60% of the land in the Netherlands is at or below sea level due to hundreds of years of persistent factitious drainage and land reclamation (*Hoogland et al, 2012*). And tidally influenced islands in the Sacramento-San Joaquin River Delta are 10 to 25 feet (roughly 3.04 to 7.62 m) below the sea level (*FESSRO, 2012*). Accompanied by sea level rise, the low-lying areas like these are more prone to be inundated by sea water and heavy rain storms. For instance, large coastal areas in Sumatra Island, Indonesia have been submerged by sea for exorbitant drainage to maintain the industries of palm oil and paper making (*Boersma & van Lenteren, 2015*). These lands cannot create any economic profits anymore since they are useless for cultivation.

Sometimes, flooding could also imperil human lives and properties. In 2007, Jakarta suffered a catastrophic flooding seriously. More than 70,000 houses were submerged by standing water for weeks. It was reported that 68 people died, 200,000 people became destitute and homeless, and 1,395 patients needed to be treated because of waterborne diseases during this disaster (*HOPE Worldwide Indonesia, 2007*). Without additional aggradation from the river, once the embankments, levees, and banks which are supposed to protect the land within fail or collapse, the consequences are too ghastly to contemplate. In May 2009, when the Cyclone Aila struck Bangladesh, the flood stage was so high that it brimmed over embanks, transforming the polder into a lake (*Schmidt, 2015*). And in the Sacramento-San Joaquin River Delta, there has been 35 levee failures happened since the 1930s on account of instability, percolation, and overflow. The property loss and expense for post-disaster reconstruction could be hundreds of millions of dollars (*FESSRO, 2012*).

Not only does it cause financial troubles, inundation will disturb the water balance, nutrient wash-out, and contaminate freshwater resources by accelerating the movement of saline water into deltas as well. This will impel water suppliers to extract groundwater in order to guarantee ample fresh water for utilization in the areas where is used to be dependent on imported Delta water, such as Santa Clara Valley, San Joaquin Valley, and Antelope Valley in the United States (*Ingebritsen et al., 2000*).

In the delta urban districts, underground infrastructure – gas lines, sewage water pipes, water supply system, and foundations – will be stressed and cracked owing to subsidence (*Boersma & van Lenteren, 2015; De Mulder et al., 1994; Schmidt, 2015*). In the Netherlands, the frequency of renovating the sewer systems built on the soft soil is the double of that built on a more stable ground, and it takes roughly 250 euros for each citizen to carry out essential sewer maintenance (*Boersma & van Lenteren, 2015*). *De Mulder et al. (1994)* concluded that the subsiding effect of wooden foundations in the Polder Markerwaard, the Netherlands was similarly equivalent to 75% of the land subsidence. And he calculated that the pecuniary loss of 100,000 properties from 35 mm of land sinking would amount to 800×10^6 Netherlands Antillean Guilders at the 1981 level, which corresponds to approximately $1{,}250\times10^6$ dollars at the 1992 level.

Furthermore, the oxidation process of organic matter will aggravate the burden on the atmosphere by emitting greenhouse gasses. Accordingly, 1 mm of subsidence due to peat oxidation will release 2259 kg of CO_2 to the air in each hectare area (*Van den Akker et al., 2008*). Let alone the fact that biodiversity would be influenced by subsidence as natural habitats are replaced by sea progressively in the coastal regions (*Boersma & van Lenteren, 2015*).

4.1.4 Current countermeasures

The existing countermeasures for land subsidence can be assorted into two sections, either attenuating the subsidence rate or accommodating the influence from land sinking. In order to slow down the land subsidence rate, several measures can be implemented, including water level control, coastal restoration, and weight reduction for urban regions (*Boersma & van Lenteren, 2015; Ingebritsen et al., 2000; Törnqvist et al., 2008; Schmidt, 2015*). Furthermore, adaptation to subsidence is another optional strategy, which involves hoisting the elevation of houses and roads, converting underground constructions above the surface (e.g., cables), consolidating the bearing capacity of soil (in the combination with geotextiles and smart soil technologies), and improving the height and stability of dikes and levees.

The main purpose of regulating water level is to decrease the rate of peat decomposition by submerging peat layer enduringly, which was indicated in a research carried on by the USGS (U. S. Geological Survey) and the California Department of Water Resources on Twitchell Island in the western Sacramento-San Joaquin River Delta (*FESSRO, 2012; Ingebritsen et al., 2000*). Nevertheless, this approach is not simple and easy to put into practice. The effectiveness could take decades to validate and the impacts on environment, agriculture, and infrastructures are long-range. Abundant data and rigorous science analysis on subsidence and peat elevations are indispensable requirements when searching for sustainable solutions, which demand financial support from governments and authorities, especially in the areas where problems of subsidence are critical (*Boersma & van Lenteren, 2015; Erkens, 2016*).

In the regions where subsidence is induced by exhaustive withdraw of groundwater, regulation and reduction of exploitation, as well as artificial recharge are two cardinal methods to control subsidence (*Poland & Davis, 1969*). Keeping strict limitations on extracting groundwater has been proved to be an effective way of mitigating subsidence in Tokyo, Japan, Bangkok, Thailand, and Po Delta (*Fabris et al., 2014; Schmidt, 2015*). The subsidence rate in Tokyo was reduced from 240 $mm/year$ in 1968 to 10 $mm/year$ by 2006. The government of Bangkok achieved the abatement of groundwater consumption by increasing the royalties drastically (from 1.2 million $m^3/year$ in the 1980s to 0.8 million $m^3/year$ in 1985). Therefore, the land subsidence rate in 1985 was decreased by more than 6 times as compared with that in the 1980s (from 120 mm to 10 – 20 mm/day) (*Phien-wej et al., 2006; Schmidt, 2015*). And the subsidence situation in the Po Delta had been obtained great reduction as the cease of groundwater use since the late 1970s. The rate cut down from 180 $mm/year$ during 1958 and 1962 to 33 $mm/year$ during the period of 1962 to 1967 (*Fabris et al., 2014*).

Artificial recharge, which is also known as management of aquifer recharge (MAR), has shown its validity on the ascension of groundwater level and retardation of land subsidence (*Dillon, 2005; Schmidt, 2015*). In accordance with the distinctions on time duration (permanent or temporary), space, and vulnerable condition, there are three ways to achieve artificial recharge, namely, injection wells/recirculation system, infiltration grooves, and infiltration wells (*De Mulder et al., 1994*). But it becomes a contentious option as some people argued that artificial recharge could not prevent the distortion of building foundations and roads from subsidence, after all, the effectiveness is assumed to be unilateral and unpredictable. While others believed that this countermeasure is the most promising and worthy approach so far to compensate the drawdown of groundwater level, which could retard the subsidence and avoid damages to constructions (*De Mulder et al., 1994; Schmidt, 2015*).

In order to investigate the effects of this measure, *De Mulder et al. (1994)* compared those three methods of artificial recharge in the Polder Markerwaard, making use of a numerical finite-element model (FIESTA). The method of injection wells/recirculation system is to empty purified surface water into the aquifers with the help of recirculation system. The method of infiltration grooves is to dig incisions of 10 to 15 m deep through the Holocene cover beds, of which the effectiveness can culminate to 90%. But the grooves are easily blocked and need extra maintenance. And the method of infiltration wells is a combination of the former two and diminishes the risk of clogging, which can achieve 80 – 90% of compensation maximum. The results indicated that the injection wells/recirculation system method was a more pliable choice than others. But it suggested that the combination of them might obtain a better result in the cost-effective aspect.

With regard to the peat regions for agriculture use like Sacramento Delta and western and northern parts of the Netherlands, where the groundwater levels fluctuate with seasons (because of temperature and precipitation), controlling groundwater between sufficient levels is considerably significant in the interest of preventing inundation and mitigating land subsidence. *Querner et al. (2012)* proved that subsurface drains beneath the surface water level were the best measure on alleviating the process of land sinking in the Polder Zegveld, the Netherlands, in comparison with other two strategies – comprising of high surface water levels and water level control in combination with subsurface drains. The subsurface drains could moderate the variations of groundwater level in different seasons. For example, in the wet seasons (like autumn and winter), groundwater would drain rapidly to surface water system once it is higher than the drain pipes, averting flooding and providing sufficient unsaturated space for vegetation. And in summer, the surface water could replenish groundwater conversely when it drops too low, preventing exposure of underground peat in the atmosphere. In this way, groundwater could be maintained practically at the same level as subsurface drains, and vulnerabilities including inundation and subsidence could acquire commendable domination. From the simulation results of MOGROW model, the subsidence could be reduced as high as 50% in summer. However, approximately 30% more of surface water should be supplied in summer in order to actualize the permanent submergence of peat layer under groundwater, which suggested the higher capacity of pumps, more energy, and more cost were required.

At the same time, coastal restoration can be considered as a plan to mitigate subsidence via a better management of water system in the delta areas. Through renovating the wetland and riparian habitat, not only does ecological environment attain improvement, it also enables the growth of wetland and hygrophilous vegetation for biomass accumulation. In the United States, several deltas have already made plans for this subject (*Boersma & van Lenteren, 2015; Ingebritsen et al., 2000; Törnqvist et al., 2008*). Nevertheless, one condition of success is to have an exhaustive understanding and comprehensive information on subsurface sedimentary architecture. Moreover, in the Sacramento Delta, the government considers to redeem the local farmers and inundate the land to let natural sedimentation create new grounds. Conflicts with farmers and more serious troubles like salinization slow down the whole progress of the project. Except for this, it is also feasible to decrease the total load on the land by shifting heavy traffic away from the weak streets and replacing the road foundations with lighter materials, such as pumice stone.

On balance, the above statements certify the intimate relationship between land subsidence and groundwater level in the delta areas. The rate of land subsidence is slow and barely visible, together with the fact that it is not universally conscious by the public. The consequences of damage from land subsidence can be beyond expectation if no countermeasures are implemented in advance. In accordance with diverse causations of land sinking, difference countermeasures should be put into effect suiting to the local circumstances. For the polders built on peat lands, as those in the Netherlands, efficient regulations on the local water system will make the most significant contribution to the realization of good policies to mitigate subsidence.

4.2 Foundation techniques in the Netherlands

The historic cities and towns like Gouda in the low-lying areas of the Netherlands appeared and grew out of small agricultural settlements, fishing villages, or trading centers since the High Middle Ages (1000 – 1250). The houses during that period were built, extended, demolished, and rebuilt, which makes it nearly impossible to leave any historical remains for archaeological study. Only from the Late Middle Ages, buildings and foundations were in a relatively good state of preservation, and foundation techniques evolved with time and scientific progress (*Van Winsen et al. in Dutch, 2015*). In terms of the main foundation types in the research area, three types of foundations will be introduced in the following part.

4.2.1 Foundation "op staal"

The foundation "op staal" refers to the houses built on the "solid soil" with a certain extent of soil reinforcement instead of piles inserted into the soil. The "solid soil" consists of dense sand, clay, or loam layers, which were believed to have a high bearing capacity. This type of foundation was extensively constructed for light buildings since the 16th century, due to the fact that it is simple and affordable. For many historic cities like Gouda, Alkmaar, Dordrecht, and Amsterdam, it prevailed until the 20th century (*Van Winsen et al. in Dutch, 2015*).

The ways of land improvement differ in diverse cities. For instance, in Alkmaar, residents dug a foundation trench of less than 0.5 m and filled the soil with sand or mortar powder to make the land surface at the same horizontal level. But for smaller houses, a simpler method was implemented with shallow brick foundations. In Dordrecht, more various materials were used, such as dirt, debris, rubble, mortar, and even secondhand planks and beams from deserted ships. Sometimes, it also possible to mix two or more kinds of materials together. The most prevalent land improvement in Amsterdam was to place the trunks or scrap timbers crosswise over each other. The spaces between the beams would be filled with sand or branches. And it was usually constructed on the tamped lands with sand, peat, or pebbles (*Van Winsen et al. in Dutch, 2015*).

Nevertheless, the "solid soil" is not as firm as it was expected to be. Some investigations have confirmed that this type of foundation would sink along with land subsidence in the regions with peat and clay underneath (*De Mulder et al., 1994; Van Winsen et al. in Dutch, 2015*). And *De Mulder et al. (1994)* pointed out the sinking rate of historical houses with shallow foundations like "op staal" was equivalent to around 75% of the land subsidence in the Polder Markerwaard, the Netherlands.

4.2.2 Foundation "op kleef" and foundation "op stuit" with wooden poles

The foundation "op kleef" utilizes the adhesive resistance between timber poles called "slieten" in Dutch (about 1 – 5 m long) and soft soil such as peat and clay. This technique was first introduced in Amsterdam from the 13th and early 14th century when the foundation "op staal" was still in the common use. The actual development of "op kleef" on a large scale was originated from the application of wooden stakes for strutting stone walls in Amsterdam during the 15th century, while "op staal" occurred simultaneously for lighter buildings in Alkmaar and Dordrecht, for example (*Klaassen, 2008; Van Winsen et al. in Dutch, 2015*). For the sake of enhancing the weight capacity, longer and more closely spaced wooden piles were placed underground firmly and touched off the compaction of soil.

The foundation "op stuit" means wooden or concrete poles are long enough (10 m on average) to insert into the Pleistocene sand layer beneath the peat and clay, which could reach up to -12 $m\ NAP$ (*Van Winsen et al. in Dutch, 2015*). During the 17th century, this kind of foundation with wooden poles was widely applied until concrete piles emerged on a large scale after the World War II (*Klaassen, 2008*). In comparison with "op kleef", this type foundation has a higher bearing capacity. But the consequences of land subsidence raise the effective stresses on the piles, resulting in a more and more unstable capacity in pace with subsidence (*De Mulder et al., 1994*). At the same time, "op stuit" with wooden poles is the most frequently reused foundation among all the others after a building was destroyed or burned down. Yet, in order to cut down the cost of a new foundation, new structures were continued to construct on the old foundations, in spite of the fact that they were no longer suitable for higher and heavier buildings than the previous ones. Thus, the government of Amsterdam initiated to set regulations on foundation reuse. Subsequently, other cities in the Netherlands started as well at different times with different provisions (*Van Winsen et al. in Dutch, 2015*).

In general, wooden pile foundations are broadly used to stabilize urban settlement and support historic constructions along coastal areas and river sites in Europe for several thousand years (*Huisman et al., 2008; Kretschmar et al., 2008*). In the Netherlands, more than 12 million wooden poles are estimated to be in place. They were applied to the majority of light structures, for example, single-family houses when Dutch cities developed rapidly at the beginning of the 20th century (*Klaassen, 2008; Van Winsen et al. in Dutch, 2015*). And in the past 50 years, they were still used for the small structures like sewer systems, greenhouses, and sheds.

Lately, the buildings on wooden foundations have undergone structural instability on account of fungal bacterial degradation (*Klaassen, 2008; Huisman et al., 2008*). When the wooden poles are exposed to air and get in the contact with oxygen (when groundwater level is low), fungus start to grow on them and cause "dry rot". On the other hand, under the anoxic conditions (when groundwater level is high), erosion bacteria can colonize and corrode wood as well (*Holt & Jones, 1983*). Nevertheless, the rate of degradation of bacteria is slower than that of fungi. In the interest of slow down the progress of all kinds of wood degradation, it is indispensable to ensure the wooden foundations are waterlogged, which indicates the groundwater level should always be sufficiently high (*Huisman et al., 2008; Kretschmar et al., 2008*).

4.2.3 Foundation "op stuit" with concrete poles

From the second half of the 20th century, the requirement for concrete foundations increased while that for wooden foundation became less. Because concrete piles have higher supporting strength and little sensitivity to groundwater level fluctuation. This technique utilizes steel tubes filled with concrete to provide desired bearing capacity for industrial factories or other heavy constructions (*Van Winsen et al. in Dutch, 2015*). Most new houses built in this century are with this type of foundation.

4.3 Urban groundwater level management

Groundwater is a significant component of urban water resource since it has strong links to multiple stages of urban development, as well as issues related to science, economy, society, legislation, environment, and politics (*Foster & Vairavamoorthy, 2013; Vázquez-Suñé et al., 2005*). The underground aquifers can be functioned as a buffer to cope with extreme weather conditions and artificial activities, due to its large storage capacity. At the same time, it can be a provider of domestic water with good quality (*Morris et al., 2003*). Yet, the importance of groundwater in the urban areas is underappreciated, because it is invisible, complicated, slow responding, and basically untouchable by man (*Foster & Vairavamoorthy, 2013; Van de Ven & Rijsberman, 1999*). Local inhabitants find it would be hard for them to connect their problems to groundwater owing to limited understanding of the system. Thus, governments hardly solve these problems through groundwater management.

Consequently, groundwater comes down to the world where it is treated as the last available resource for municipal water supply and ultimate place for contaminants. In the Netherlands, the urban water management basically concentrates on surface water instead to control subsidence and other damages which are primarily caused by insufficient groundwater management (*STOWA, 2016*). Therefore, the awareness of the groundwater importance should be raised, and corresponding measures for efficient management should be in question for urban water managers.

4.3.1 Influential factors on urban groundwater level

The rapid development of urbanization has become the major challenge for water management globally. The infrastructure development and engineering usually only consider the short-term economic benefits, which often have adverse impacts on natural water cycle in both quantity and quality respects (see *Figure 3*) (*Barrett et al., 1999; Epting et al., 2008; Foster & Vairavamoorthy, 2013; Sekhar et al., 2013; Vázquez-Suñé et al., 2005*). In general, urbanization could alter the recharge mechanisms in the natural system and raise the pollutant loads in the aquifers. Nevertheless, the problem related to groundwater quality is more widely investigated in various cities. When it comes to groundwater level, it seems it is less concerned, particularly in the developing countries (*Adelana et al., 2008*). For the sake of managing urban groundwater effectively and sustainably, it is important to consider groundwater within an integrated framework compromising of identifying and quantifying various influential factors as well as their relationships (*Barrett et al., 1999; Foster et al., 2010; Jacobsen et al., 2012*). Therefore, the following parts will be discussed the main contributors of recharge on groundwater in the urban area.

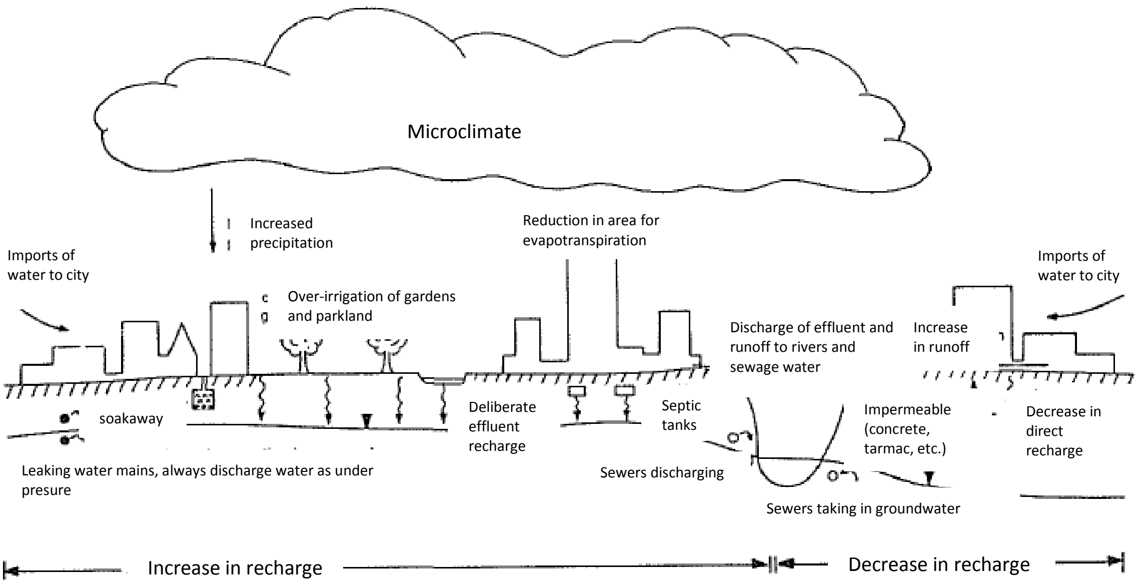


Figure 3 Influence of urbanization on groundwater (Lerner, 1990)

In the natural environment, the sources of recharge for groundwater in the unconfined aquifers are basically infiltration from precipitation, evapotranspiration by vegetation, surface waters, deeper aquifers, and irrigation. As there is a micro-climate produced in the urban areas due to land use change, some processes might be modified as well, such as infiltration from precipitation. Concurrently, originally nonexistent recharge would be added, for example, stormwater runoff, underground drainage system, artificial recharge and extraction, as well as leakage from water supply and sewer system (*Barrett et al., 1999; Lerner, 1990*).

A large portion of the urban area is predominantly covered by impermeable roofs, roads, and parking lots. When it starts to rain, the storm water runoff will considerably increase, thus the direct infiltration to the underground will decrease (*Adelana et al., 2008; Barrett et al., 1999; Jai Kiran, 2000; Tellam et al., 2006; Vázquez-Suñé et al., 2005*). This runoff will be carried out of the city or collected in the storage reservoirs via storm sewers, drains, or other artificial facilities, rather than percolate through the soil. But not all of those paved areas are completely water-proof, precipitation could still recharge groundwater through crevices of the paving surface, joints between bricks, and tile pavers. Sometimes, permeable pavements are used to reduce peak flows, therefore, groundwater level rise (*Barrett et al., 1999; Lerner, 1990; Van de Ven & Rijsberman, 1999*). At the same time, precipitation could also infiltrate into the ground by city parks and gardens, and coupled over-irrigation in these locations leads to excess water recharges and rises groundwater level. This has caused the groundwater level in the low-lying areas of Doha (Qatar) went up at the surface level.

Evapotranspiration is modified by urbanization as well owing to the micro-climate and changes of radiative properties of the surface material (from vegetation to impervious pavements) (*Tellam et al., 2006*). Less green areas imply less transpiration, so there is less loss from groundwater system (*Van de Ven & Rijsberman, 1999; Vázquez-Suñé et al., 2005*). And in hot summers, there is an obvious groundwater level difference between areas with trees and those without (*Tellam et al., 2006*).

Once the head differences – no matter between confined and unconfined aquifers, or between phreatic aquifers and surface water system – exist, it will result in a seepage flow. It is difficult to measure the interflow rates between different aquifers, but it is normally in the order of 1 to 2 mm/day. In some cases, like in the north of Gouda, it can achieve as high as 10 mm/day (*Tellam et al., 2006; Van de Ven & Rijsberman, 1999*). During the urban construction period, the aquitard between two aquifers might be perforated for building foundations, the seepage and percolation rates will be accelerated. Moreover, the groundwater in the riparian areas has a close relationship with surface water, only if the transmissivity is high. In the urban areas, the man-made hydraulic facilities (e.g., drainage system) allow direct connection between surface water and groundwater. The urban canals are functioned as both a compensatory source and extra storage room for groundwater, just as the condition for the Biscayne aquifer in the Miami-Dade County (US) (*Hughes & White, 2014*).

One of the most palpable alterations on urban water system is the installation of sewer and water distribution systems. However, due to most of water-carrying pipes in cities are leaky, old, or rusty, large amount of water, whether it is sewage water or drinking water, replenishes to groundwater, which counteracts the effect of less infiltration from precipitation and is considered as the greatest contributor to groundwater level rise (*Adelana et al., 2008; Barrett et al., 1999; Lerner, 1990; Vázquez-Suñé et al., 2005; 2010*). The water distribution system brings a great deal of fresh water to support residence and industry in the urban areas. For example, in Nottingham (UK), the public supply system provides around 650 $mm/year$ of water and even 7000 $mm/year$ to those highly developed commercial centers. This water becomes a potential source of groundwater recharge if the water supply pipes start to leak. Accordingly, the water losses from supply system are beyond 50 $L/person/day$ on a global scale, in some extreme cases, they are estimated to reach 180 $L/person/day$ (*Kim et al., 2001*). In the developed countries, the proportion of water losses due to utility system leakage is comparably low, from 12% in Austin (US), 15% in Barcelona (Spain), to 25% in San Antonio (US) and the UK (*Adelana et al., 2008; Barrett et al., 1999; Sharp & Banner, 2000; Vázquez-Suñé et al., 2005*). Nevertheless, *Lerner (1990)* pointed out the loss rate could go up to 50% in the UK, which is equivalent to a potential recharge of 3000 mm/year. This is even higher than that in Cape Town (South Africa), which is primarily owing to pipe bursts and leakages (*Adelana et al., 2008*).

In addition, the leakage problem of sewer system has a significant impact on groundwater in both quantity and quality terms. Although the sewer pipes are designed to be leaky of approximately 10%, in reality, the leaking conditions are hardly controlled below this value (*Adelana et al., 2008*). This amount of water, together with the leakage from water supply networks, build up a massive contribution on groundwater. Based on the research carried out in Bermuda (UK), the leakage from sewer and water delivery system led to an additional recharge on groundwater from 365 $mm/year$ in rural areas to 575 $mm/year$ in the urban areas (*Lerner, 1990*). Furthermore, in the cities like Gouda, where relatively high groundwater level is desirable to prevent the oxidation of peat, the sewer system are basically installed under the groundwater level. Once the sewer systems are leaky, they begin to function as a drainage system to cause groundwater overdraft, corresponding damages like subsidence will be intrigued (*Van de Ven & Rijsberman, 1999; STOWA, 2016*).

In order to counteract the consequences of the exorbitantly high groundwater level, the subsurface drainage systems are introduced into the urban areas, which is another example of artificial alteration on the urban water system. The drainage system is devised to drain shallow unconfined aquifers to protect underground infrastructures when the groundwater level is higher than expectation (*Lerner, 1990; Wolf et al., 2006*). However, with the fast-speed urban expansion and climate change, those drains are no longer adequate to satisfy the demands since extra recharge on groundwater has already exceeded their loading capacities (*Graham et al., 2012*). Concurrently, due to underground construction work (e.g. sheet-piling) and lack of regular maintenance, the drainage lines would be ruptured or blocked, losing their original functionality (*Van de Ven & Rijsberman, 1999*). Hence, there is a possibility that rainwater will infiltrate from drainage system to groundwater, which has been proven to occur in Hong Kong (*Lerner, 1990*). And other underground facilities, such as electricity and telephone cables, require deep trenches, which are typically filled with highly permeable materials and might act as drains as well (*Van de Ven & Rijsberman, 1999; Vázquez-Suñé et al., 2005*).

In most developing countries, groundwater is becoming incrementally valuable owing to the deteriorative quality of surface water and high costs of hydraulic constructions. It is an alternative to reduce the pressure on conventional freshwater supply. The use of groundwater relies on multiple elements, such as geology, climate, availability, economic status, as well as the history of urban development (*Adelana et al., 2008; Vázquez-Suñé et al., 2005*). In Europe, the prosperity of majority metropolises began as highly industrialized centers, which demanded massive fresh water and drastically collected from groundwater. Whilst, since the last century, wells were gradually abandoned or extracted less water, because most manufacturing districts moved out of city centers, groundwater was too contaminative to use, or drinking water companies imported water from elsewhere for the environment. Thus, the groundwater level rebounded steeply and unexpectedly (*Foster & Vairavamoorthy, 2013; Van de Ven & Rijsberman, 1999*). For example, the rate of groundwater exploitation in The Hague (the Netherlands) diminished about 3 million $m^3/year$ during a decade of 1970 – 1980, which led to a significant increase of groundwater table in a large city area (*Van de Ven & Rijsberman, 1999*). The similar circumstances also happened to other major cities including Milan (Italy) and Barcelona (Spain) (*Vázquez-Suñé et al., 2005*). At present, the use of groundwater in developed countries is more emphasized on environmental perspective. All the same, groundwater is still the primal resource and substitution of surface water for municipal, agricultural and recreational water use globally. In some cases, groundwater is even the sole resource for domestic drinking water, like in Miami-Dade County (US) (*Hughes & White, 2014*).

On the basis of the analysis above, in most cases, a water balance model for urban areas can be established as follow (*Tellam et al., 2006*):

$$P + L_s + L_f + D + SWI + GWI + AR - ET - RO - IS - Q - SWO - GWO - L_o = \Delta S$$

Equation 1

Where:

P = Precipitation;

L_s = Supply leakage;

L_f = Foul leakage;

D = Industrial discharge;

SWI = Surface water inflow;

GWI = Groundwater inflow;

AR = Artificial recharge;

ET = Evapotranspiration;

RO = Runoff;

IS = Interception storage;

Q = Abstraction;

SWO = Surface water outflow;

GWO = Groundwater outflow;

L_o = Exfiltration to pipelines;

ΔS = Change in water stored.

However, it is rather difficult to assess how much of each factor contributes to the urban hydrological cycle, and correlative estimation standard has not been formed (*Tellam et al., 2006; Vázquez-Suñé et al., 2005*). *Vázquez-Suñé et al. (2010)* made a preliminary analysis of the proportions of potential recharge sources for the Barcelona (Spain) city aquifers via tracing and computing the selected chemical species representing different sources. The result suggested that approximately 30% of total recharge was from wastewater (combined sewer system), 22% from water supply network, 20% from runoff infiltration, 17% from rainfall recharge, and 11% from the Besòs River. Further investigation and research will be needed for this aspect.

4.3.2 Damages caused by unstable urban groundwater regime

As mentioned above, urbanization modifies the urban water cycle and balance considerably. In the light of the most dominant influential factors, the groundwater level would decline or ascend drastically, inducing potential damages to varying degrees, such as downward well and river yields, inundation, water quality deterioration, salinization, ground instability, threaten to public health, and social conflict (see *Figure 4*) (*Tellam et al., 2006*). In the following part, these damages would be described based on their origins, which are divided into high groundwater level and low groundwater level, respectively.

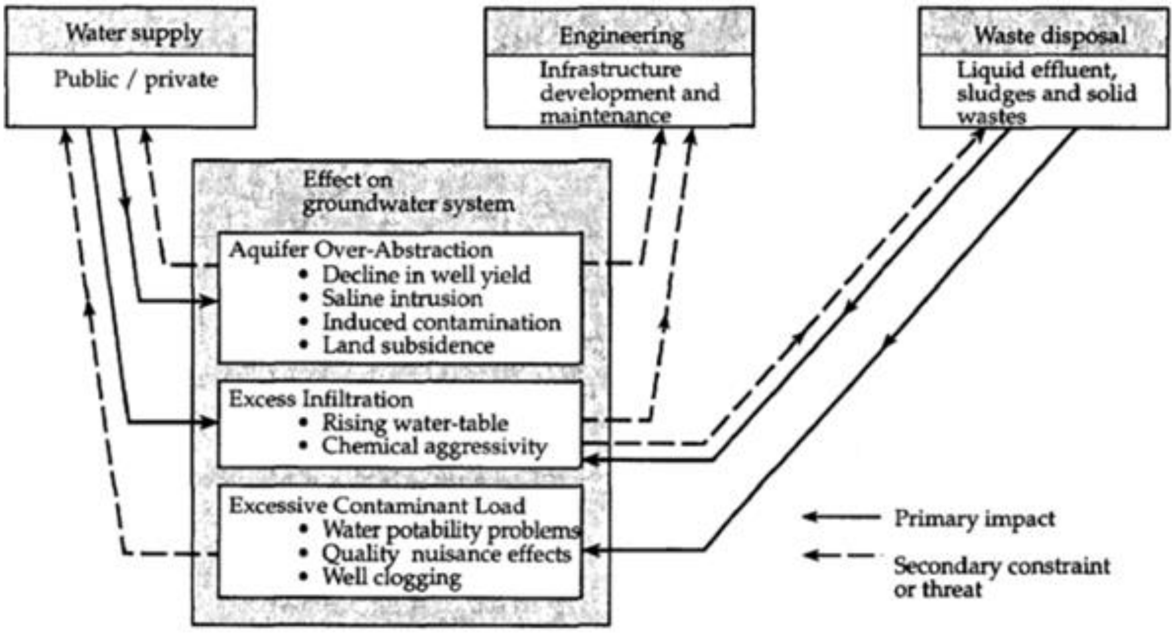


Figure 4 Damages caused by urbanization through groundwater system (Foster et al., 1998)

A lot of European cities are suffering from the negative consequences resulted from continuously rising groundwater (*Vázquez-Suñé & Sánchez-Vila, 1999; Vázquez-Suñé et al., 2005*). In the Netherlands, around 200,000 promises are exposed to the damages resulted from exorbitantly high groundwater level (*Van de Ven & Rijsberman, 1999*). These possible hazards will be summarized into four aspects, namely, inundation, detriments to subsurface infrastructure, contamination problem, and adverse influence on vegetation.

1. Inundation:

Progressive flooding is the cardinal consequence of high groundwater level. Most of the buildings and structures in Europe were designed and constructed in the period when the groundwater was largely exploited, and the possibility was not taken into consideration that one day the water level might bounce back (*Vázquez-Suñé & Sánchez-Vila, 1999*). However, this unexpected possibility comes true and leads to flooding in subsurface structures, for example, basements, tank, and tunnels. Thus, additional drainage systems need to be installed (*Foster et al., 1998; Foster & Vairavamoorthy, 2013; Tellam et al., 2006; Vázquez-Suñé et al., 2005*). What is more, the storage capacity in the unsaturated zone and drainage system are restricted when the groundwater is keeping rising. During the extreme precipitation events, rainwater is likely to be accumulated on the streets because groundwater reacts rapidly to rainfall and peaks are impossibly attenuated. Subsequently, it causes traffic obstructions, excess moisture and molds in houses, and public health threats as residents are exposed to bacteria and pollutants in the standing water (*STOWA, 2016; Van de Ven & Rijsberman, 1999*).

2. Detriments to subsurface infrastructures:

Due to hydrostatic uplift, high groundwater has adverse impacts on underground facilities, such as public transport railways, underground parking lots, domestic utility conduits, as well as foundations of roads, sewer system, and premises (*Foster et al., 1998; Vázquez-Suñé et al., 2005; Van de Ven & Rijsberman, 1999*). At the same time, the pore pressures in the unsaturated zone are increased as a result of rising groundwater level, hence, the effective stresses are reduced. This will bring down the bearing capacity of foundations and make them unstable and vulnerable to the external environmental change (*Foster &*

Vairavamoorthy, 2013; STOWA, 2016; Tellam et al., 2006). Unfavorably, it is possible to give rise to the foundation collapse and landslide initiation. Sometimes, it even increases the risk coefficient during a natural disaster like an earthquake.

3. Contamination problem:

As discussed before, high groundwater will make underground sanitation utilities malfunction, the exchange frequency between groundwater and sewage water is enhanced and the aquifer flow direction is reversed all the time consequently. The contaminants from sewage water will be carried by groundwater and pollute larger range owing to the countless change of groundwater flow direction. Furthermore, the chemicals – such as acid, sulfate, and organic solvents – existed in polluted groundwater will corrode concrete and other underground structures (*Foster et al., 1998; Foster & Vairavamoorthy, 2013*).

4. Adverse influence on vegetation:

The root zone in private gardens and public parks will be insufficient for vegetation growth because groundwater is so high that existed oxygen is squeezed out of the soil. Trees cannot survive in this situation, they can be easily blown over, or perish directly (*STOWA, 2016; Van de Ven & Rijsberman, 1999*). Therefore, extra tree pots are required to elevate ground level and cease the impact of high groundwater.

As for exceptionally low groundwater level, the relative damages can be assorted into land subsidence, environmental and ecological harm, quality degradation, and concerns on water supply, as explained below.

1. Land subsidence:

Lowering groundwater level results in land subsidence on the local and even larger scale, in particular, it commonly happens in the coastal regions (e.g., Jakarta (Indonesia), Bangkok (Thailand), and Mexico City). Peat oxidation and clay compaction in the aquitards are the main effects when groundwater is too low to submerge them. And differential settling endangers urban infrastructures, for example, roads, wooden foundations, subsurface tunnels, utility pipes, and so on (*Foster et al., 1998; Foster & Vairavamoorthy, 2013; Tellam et al., 2006; Vázquez-Suñé et al., 2005; Villholth, 2006*). Accordingly, there were at least 100,000 houses experiencing foundation damages due to low groundwater level in 2000. The amount of approximately 5 billion euros is estimated to invest in order to fix all those foundations (*STOWA, 2016*). Furthermore, another billion euros are spent on sewer renovation every year as well (*Van de Ven & Rijsberman, 1999*). Yet, on account of climate change, the damages caused by subsidence will aggregate and of course, a more financial loss will it be in the future. For more information, please refer to *Chapter 4.1.2* and *4.1.3.*

2. Environmental and ecological harm:

The drawdown of groundwater level in the unconfined aquifer would bring about dehydration or desiccation, which transforms the distribution of air and moisture underground (*De Mulder et al., 1994; Van de Ven & Rijsberman, 1999*). This alteration will go a step further to impact plants through changing oxygen, moisture, mineralization, oxidation, and nutrient conditions in the soil. *Claessen et al. (1989)* confirmed that these changes would lead to the decease of certain type of vegetation. In addition, the disappearance of plants in the city indicates less evapotranspiration will occur, cooling effect becomes less and occurrence possibility of heat stress increases simultaneously (*STOWA, 2016*). Hence, the ecological quality in the urban areas is compromised by low groundwater level.

3. Quality degradation:

The same as the condition when sewer system is cracked and leaky, groundwater quality would be affected owing to seepage of contaminated water. Meanwhile, the head difference between groundwater and sea level will induce seawater intrusion and increase the salinity and concentrations of other ions (*Adelana et al., 2008; Foster & Vairavamoorthy, 2013; Vázquez-Suñé et al., 2005; Villholth, 2006*). The corrosion rates of underground concrete and metal will be accelerated once they come into contact with polluted groundwater.

4. Concerns on water supply:

No matter for municipal water supply companies or private abstractors, both of them pay close attention to groundwater availability and quality. Nevertheless, in order to satisfy the increasing demands on fresh water, the groundwater table descend drastically. In turn, this will result in drying up wells seasonally (e.g., the Upper Guadiana Basin in the central Spanish Plateau), competitive overexploitation in deeper aquifers, going up the pumping costs, and complaints or even conflicts with consumers (*Foster et al., 1998; Morris et al., 2003; Villholth, 2006*).

4.3.3 Challenges and improvements on urban groundwater management

It is not an easy task to manage urban groundwater since it cannot be treated as a separate entity rather than a fundamental component of urban infrastructure planning and integrated water resources. Because of various hydrogeological settings, socio-economic evolutions, and institutional provisions, the damages and potential problems caused by groundwater are distinct. It is nearly impossible to establish a universal management strategy to solve all those issues at once, and one successful solution might become an additional burden in another situation (*Foster & Vairavamoorthy, 2013*). In addition, groundwater problems usually come into notice after a long period, which makes it way much harder to address. Likewise, the interventions could only display their benefits after a long-term accumulation (*Foster et al., 1998; STOWA, 2016*). However, in general, the challenges for the current circumstance of urban groundwater management are similar, and there is plenty room for improvement. Thus, instead of summarizing pragmatic approaches or successful cases, this section will emphasize on current challenges and future improvements for urban groundwater management.

In order to establish a more sustainable and efficient urban groundwater management plan, it is essential to understand the current groundwater status, trends and management arrangements, which involve institutional provision, capacity, and effectiveness, water allocation arrangements and use regime, as well as adequacy of monitoring networks (*Foster & Vairavamoorthy, 2013*). In the light of these basic requirements, and comparing with the current situation, the following challenges urban groundwater management confronts with are summarized.

1. Lack of adequate understanding of urban groundwater system:

The significance of knowledge on urban groundwater system behavior is underestimated, partly due to the “out of sight, out of mind” characteristics of groundwater, and partly due to the time lag between causes and consequences (*Morris et al., 2003*). In some cases, inadequate technical and financial supports lead to limited data on aquifer system and groundwater status. At the same time, multiple underground structures alter the original geohydrological conditions and make it more complicated to interpret. And professional hydrogeological expertise is largely absent when it comes to urban water resources management (*Adelana et al., 2008*). Sometimes, this results in misunderstanding or misrepresenting in certain ways. Without reliable and comprehensive data, it is extremely difficult to estimate water balance and quantify groundwater fluxes, let alone establish an early warning system to inform planners and legislators about potentially hazardous problems rather than late reactions only on emergencies (*Foster & Vairavamoorthy, 2013; STOWA, 2016; Vázquez-Suñé et al., 2005*).

2. Lack of integrated cooperation and responsibility among fragmented institutions:

Urban groundwater problems are insidious and persistent. They have negative impacts on everyone, but neither governments belonging to municipality nor individual residents take their own responsibility, owing to the vacuum of cooperation between different institutions and even absence of awareness. It is obvious that the city council has the overall responsibility for the problems in the public areas, but this responsibility is shared by various sub-bureaucracies and none of which takes the initiative. This, together with the deficiency of regular communication and information exchange between scientific communities and city managers, are troublesome for each institution to perform its own function and accountability (*Vázquez-Suñé et al., 2005*). At the same time, local inhabitants do not realize they have responsibilities on managing groundwater in their own private land. They do not have specific information about possible vulnerabilities and hazards when their buildings are exposed to groundwater flooding and overdraft. Thus, they are not aware that their properties are in danger. The only thing they can do is to spend a large amount of money to repair the impaired foundations when the problems reveal, instead of taking necessary maintenance or measures to prevent in advance (*STOWA, 2016*). In the developing countries, the capacity of fledgling water management agencies is so limited that they could not deal with urban and economic development (*Foster & Vairavamoorthy, 2013; Morris et al., 2003*). Excess knowledge and equipment are required in order to form sophisticated management strategies.

3. Lack of a competent legal and jurisdiction framework:

At present, the legal and jurisdiction framework applied to groundwater management is fragmented, inconsistent, and incomplete (*Adelana et al., 2008*). The interpretations of these laws or regulations can be ambiguous and different in accordance with respective preference. For example, when a regulation defines groundwater as “a natural or man-made resource used simultaneously or sequentially by members of a community or a group of communities”, it authorizes ownership with landowners in spite of the fact that groundwater is a natural and communal resource (*Morris et al., 2003*). What is more, the legal enforcement can be inadequate or politically unacceptable, which creates a formidable challenge for urban groundwater management.

4. Lack of sufficient technical and financial supports:

As stated above, urban groundwater management demands comprehensive understanding on urban groundwater system, which needs advanced knowledge and equipment to fulfilling the requirements of groundwater monitoring and modeling (*Adelana et al., 2008*). Moreover, the cost-effectiveness of interventions is indistinct (*STOWA, 2016*). The potential measures are expensive to implement, and the effectiveness could only be visualized after a long-term experiment.

In light of the challenges for urban groundwater management these days, the future management needs to be improved in order to satisfy the requirement of utilizing groundwater sustainably and efficiently. Even though groundwater will not be for the use purpose, it is still possible to give rise to critical consequences without sufficient management measures (*Tellam et al., 2006*). The ultimate objective of urban groundwater management is to develop sustainable measures on urban groundwater system, which generally comprises cost-benefit analysis for management options, the feasibility of economic incentives, financial and institutional needs, stakeholder participation, and an adaptive management strategy (*Foster & Vairavamoorthy, 2013*). Therefore, the following aspects are highlighted on the basis of previous experience.

1. Centralize the groundwater management unit:

In general, the management unit is located in the area where groundwater flow regime is sensitive to the change of any major external factors, such as vulnerable regions due to over-abstraction and environmentally fragile zones in wet and low-lying lands. Because these areas need to be preserved at any cost and are highly affected by complex interactions occurring underground, even if urbanization seems to have little impact on them (*Foster & Vairavamoorthy, 2013; Jai Kiran, 2000*). Concurrently, the boundaries of a unit should be taken into consideration of the local political land divisions, as it will lay stress on the responsibility of public administration of municipalities. For instance, if the unit is only concentrated in a community region, consumers or inhabitants will be aware of their own interest and demands and manage the groundwater based on the overall benefits (*Morris et al., 2003*). This helps with the determination of ideal groundwater level regime and potential technical interventions to prevent adverse consequences from happening.

2. Carry on sound urban hydrogeological science:

Comprehensive understanding of groundwater system (e.g., occurrence and movement) is essential for sustainability in groundwater management and protection (*Adelana et al., 2008; Vázquez-Suñé et al., 2005*). The absence of professional experts in geohydrology is a common phenomenon in urban water management institutions. Even in the cases in which important and influential stakeholders cooperate and involve into decision making, without certain technical knowledge, what they actually care is to meet the present demands rather than develop groundwater sustainably for the future generations. Thereby, the primary effort should be focused on collecting basic geological and groundwater data. An indispensable and sensitive monitoring network on groundwater can be a prerequisite to identify where and when the problems occur or are going to occur, as well as observe the response to different incentives (*Foster et al., 1998; Morris et al., 2003*). Availability of this type of information could provide water managers with simple but robust matrices that help to develop strategies and guidelines for optimal groundwater management, opt efficient measures for problem remediation, and indicate potential risk to groundwater from a planned activity.

3. Establish realistic policies and effective institutional arrangements:

Institutional frameworks for groundwater management are usually formulated on the national level, they rarely exist for local urban areas. It is practical for municipalities to draw up clear, acceptable, and appropriate water laws and rights based on their own conditions and requirements (*Adelana et al., 2008; Morris et al., 2003; STOWA, 2016*). And another designated institution (e.g., the water boards in the Netherlands) will enforce the laws or the regulations. Therefore, the intimate coordination between policy and enforcement is evitable to minimize and prevent the effects of possible damages. An equilibrium in the urban area should be established since groundwater has close interactions with many “external interfaces”, for instance, sanitation, drainage, flooding, water supply, and infrastructure functions (*Foster & Vairavamoorthy, 2013; Tellam et al., 2006*). This requires active and effective cooperation with correlative institutions or authorities, as well as land use department or even power utilities. In particular, it has been suggested to promote such a coordination to be one part of management cycle in Sub-Saharan Africa (SSA) for monitoring data, improving the understanding and giving feedback under extreme conditions (*Colvin & Chipimpi, 2005*).

What is more, it is also essential to clarify the role and responsibility of each participant – water managers, hydrological engineers, local inhabitants, and so on – in the progress of urban groundwater management (*Adelana et al., 2008; STOWA, 2016*). For example, in the Netherlands, the managers from the municipal council have the responsibility to take measures in the public areas to deal with excess groundwater, prevent flooding, and reduce negative effects of groundwater levels on subsurface structures. And the responsibility of the managers and engineers from regional water institutions has bearing on the water levels in the public regions. They are in charge of making level decisions, issuing water permits or exemptions, and carrying out projects related to alterations on surface water or groundwater bodies. As for residents, they should take care of any inconvenience caused by groundwater overflow or shortage in their own properties and lands.

4. Heighten all the stakeholders' awareness on groundwater problems:

In view of present vulnerabilities caused by groundwater and climate change, it is rather significant to raise the awareness of the public and governments to understand what kinds of hazards they are and will be confronting with (*Adelana et al., 2008*). For the governments, better awareness indicates more suitable and efficient legal frameworks or measures could be formed or achieved. In addition, proper information and education to the public would stimulate the public participation, hence, to promote close cooperation between government departments and local residents (*Jai Kiran, 2000*). Not only will increasing awareness make governments and the public realize the importance to protect and control groundwater sustainably, but it will also help with expediting proceedings of passing legislations on groundwater issues and accepting by social society (*Morris et al., 2003*).

5. Encourage active participation of stakeholders.

Stakeholder participation is the root of the success of sustainable groundwater management and policy development (*Adelana et al., 2008; Jai Kiran, 2000*). *Foster & Vairavamoorthy (2013)* suggested a "permanent consultation mechanism" federate a well-structured interaction among stakeholders. Numerous advantages can be acquired from stakeholder participation, for example, integration and coordination of groundwater, land use, and environmental management, a guarantee of the equity of various users, better estimation on current and future demands, effective implementation of strategies and decisions, and active involvement of stakeholders in data collection and monitoring.

4.4 Groundwater flow model

It has been made mention of the primary challenges of urban groundwater management (see *Chapter 4.3.3*), which indicates the significance of groundwater observation system to have an adequate database for the purpose of interpreting the urban groundwater system in a local or regional area. Nevertheless, it is unrealistic and impractical to monitor groundwater level everywhere in order to not miss any single alteration of groundwater level in time and space. On account of this situation, the groundwater flow model is introduced in this chapter to explain how it can simulate groundwater behavior under different circumstances or scenarios in both temporal and spatial aspects. A successful groundwater flow model can be an irreplaceably valuable tool for water managers to understand the groundwater system and simulate the effectiveness of future interventions.

Various computer-based models have been applied in Water Resources Management (WRM) for a long term (*Kumar, 2002; Vermeulen et al., 2013*). Generally speaking, models make use of mathematical equations to describe a physical system conceptually or approximately. They can be assorted into simulation models and conceptual models (*Hare, 2011*). Simulation models are quantitative computational models to simulate the behavior of the representative system. Yet, conceptual models are qualitative models to describe the system according to the primary components and their structural relationships, which are usually shown by casual diagrams. At present, they can be considered as an initial step to develop a simulation model.

It is quite impossible to have a good look at groundwater flow conditions beneath the surface. However, it is essential to have a detailed understanding of urban geohydrology for the purpose of sustainable water management, especially in respect of the groundwater fluxes (*Vázquez-Suñé & Sánchez-Vila, 1999; Warren, 2015*). Thus, establishing a groundwater flow model is an evitable step in groundwater regime management. In general, groundwater modeling studies are carried on with either deterministic models or stochastic models. Whilst, since the 1980s, deterministic models became more and more popular and extensively used. In accordance with an elaborate comprehension of cause-and-effect or input-response relationships, a deterministic model could define or predetermine the subsequent response to any set of stresses in a system, even though the magnitude of the stresses has not been observed in the historical database (*Konikow & Mercer, 1988; Kumar, 2002*). Through a simplified hydrogeological system on the basis of mathematic equations and assumptions, such a model should be considered as an approximation rather than a duplication of real situations. The serviceability of a deterministic model highly relies on how well the mathematical equations and the assumptions represent the hydrological processes, which reflect the real physical system. For the sake of evaluating the usefulness of a model, it is essential to comprehend the mathematic equations and the assumptions built in the model (*Warren, 2015*). Usually, these assumptions comprise of the direction of flow, the geometry of the aquifer, and the heterogeneity or anisotropy of sediments or bedrock of the aquifer (*Kumar, 2002*).

4.4.1 Mathematic equations and assumptions

As a whole, the groundwater flow is expressed by the Darcy's Law, which illustrates the flow rate is proportional to the hydraulic gradient with a constant value of hydraulic conductivity if the porous material and the fluid are the same. Since it was derived from laboratory experiments on laminar flow of water through a sand column, the Darcy's Law has its own limitations when it is applied in reality. For instance, the turbulent flow might occur near the screens of large-capacity wells or in rocks with distinct secondary permeability. In order to neglect the negative influence from local or small-scale turbulence, the Darcy's Law is preferable to be applied at the regional scale (*Konikow & Mercer, 1988*).

In the light of simplifying groundwater flow problems, it is acceptable to assume the isothermal conditions predominate and fluid properties like density and viscosity are homogeneous. Therefore, the mathematical equations to describe groundwater flow are on the basis of the combination of the Darcy's Law and the continuity equations (*Konikow & Mercer, 1988; Kumar, 2002; Ros, 2008*). Furthermore, due to the variation of groundwater system over time and space, the governing equations are normally expressed as the change of the dependent variables (hydraulic head in this case) with respect to both space and time. Thus, the governing flow equation for the three-dimensional saturated flow in a nonhomogeneous and anisotropic aquifer is as:

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right)=S_s\frac{\partial h}{\partial t}+W(x,y,z,t)$$

Equation 2

Where:

x, y, z = Cartesian coordinates $[L]$;

K_{xx}, K_{yy}, K_{zz} = hydraulic conductivity along the x, y, z axes (LT^{-1});

h = hydraulic head (L);

S_s = specific storage coefficient (L^{-1});

t = time (T);

$W(x, y, z, t)$ = volumetric flux per unit volume (T^{-1}).

This governing equation can be solved either by analytical models or numerical models (*Kumar, 2002*). The analytical models (e.g., Theis type curve) go a further step to simply the equation with additional assumptions, such as radial flow and infinite aquifer extent. In this way, the solutions for the equation become more amenable and cover continuously in space and time. When it is impossible for analytical models to describe the physical system, numerical models are introduced to solve the problem via numerical approximation of the governing partial differential equation. In this way, the continuous variables are substituted with discrete variables. Hence, the continuous differential equation is replaced by a set of algebraic equations. And they can be solved with either iterative or direct matrix approaches, which can be achieved efficiently by computer programs (*Konikow & Mercer, 1988; Belcher & Welch, 2006*).

In principle, the techniques of numerical models can be assorted into two methods, the finite difference method and the finite element method, respectively (*Belcher & Welch, 2006; Konikow & Mercer, 1988; Kumar, 2002*). Both of them subdivide the area of interest into a number of units, cells, or elements by a grid system with one or more layers or dimensions. The finite difference method estimates the first derivatives in the partial differential equations to be difference quotients, and the grids are rectangular cells (see *Figure 5*); while finite element method evaluates the equivalent integral formulations of the partial differential equations in accordance with the presumptive understandings of each dependent variable and parameter, and the grids are irregular polygons (see *Figure 5*). For both of these methods, the continuous boundary-value problem for the solutions of the partial differential equation can be diminished to the simultaneous solution of the set of algebraic equations with the help of the discretization of space and time dimensions.

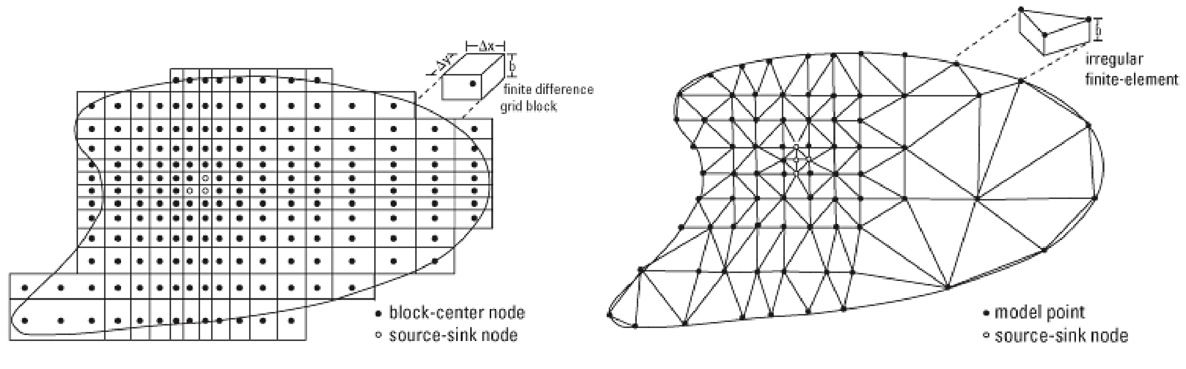


Figure 5 Examples of grids for finite difference method (left) and finite element method (right) (Belcher & Welch, 2006)

Both of the techniques have their own advantages and disadvantages in respect of availability, costs, user friendliness, applicability, and required knowledge for the user (*Konikow & Mercer, 1988; Kumar, 2002*). Sometimes, it is difficult to decide which one is superior to another. Generally speaking, in comparison with finite element method, finite difference method is less complicated in conceptual and mathematical aspects, as well as in computer programming. For finite element method, it demands more sophisticated and even more accurate mathematics, and the construction of the input data set for irregular grids is more complicated as well. Nevertheless, due to the flexibility of the grids, finite element method is able to acquire a better approximation when the boundaries of an aquifer or a parameter zone are irregular.

4.4.2 Input parameters

Since the groundwater system cannot be observed directly, the reliability of a deterministic groundwater model depends on adequate and sufficient data, even though it has been based on an accurate conceptual model of the governing process (*Konikow & Mercer, 1988; Warren, 2015*). However, in fact, these requisite data is usually not up to requirements on account of inadequate measurement frequency, measurement error, and nearly impossible determination of aquifer heterogeneities in small scales by available data. In order to ensure the reliability of the groundwater flow models, the following demands need to be fulfilled: 1) the state of the system (dependent variables) in time and space; 2) stresses and properties in the domain (coefficient in the equation); and 3) definition of boundary and initial conditions.

The dependent variable (hydraulic head in this case) relies on the groundwater level monitoring network. But in the cases where the fluid is nonhomogeneous due to the great variation of pressure, temperature, or chemical composition, additional measurements of pressure, density, and elevation are required to calculate flow rates and directions (*Konikow & Mercer, 1988*). The stresses refer to the water fluxes in the area of interest, including their magnitude, relative importance, and dependence upon hydrological parameters (*Vázquez-Suñé et al, 2005*). Other geotechnical data, such as types and thicknesses of soil layers, transmissivity, horizontal and vertical hydraulic conductivity etc., is needed as well (*Hughes & White, 2014; Warren, 2015*).

In addition, boundary and initial conditions are physical states of the processes which are used to obtain a unique solution of the partial differential equation. The boundary condition is an essential requirement for solving both steady-state and transient problems. It defines the amount, location, and method by which water enters or leaves the designated groundwater system (*Belcher & Welch, 2006*). In mathematical terms, it contains the geometry and the values of dependent variables or their derivatives at the boundary. And in physical terms, it can be specified as head, flux, or head-dependent flux. While for a mathematical solution of transient equations, except for boundary condition, the initial condition should be determined, too. It includes the values of the dependent variables appointed everywhere inside the boundary. Yet, when the equations are linear (e.g., in a confined aquifer), the boundary condition is useless as the computed drawdown can be superimposed on the natural flow system. Instead, the initial condition will be defined as the drawdown equals to zero everywhere (*Konikow & Mercer, 1988*).

4.4.3 Model calibration and sensitivity analysis

Due to the fact that the flow processes of groundwater are unseen, the physical subsurface phenomena – like aquifer properties, stresses, and boundaries – are in a considerable degree of uncertainty (*Konikow & Mercer, 1988; Warren, 2015*). This uncertainty leads to inaccurate and inadequate input data, which has been proven to be the most significant cause of errors in the model output. Hence, the model calibration procedure is introduced to estimate and adjust input parameters for the sake of matching field conditions within some acceptable criteria (*Konikow & Mercer, 1988; Kumar, 2002*). In other words, it attempts to minimize the differences between observed and computed values to achieve a good match (*Belcher & Welch, 2006; Konikow & Mercer, 1988*). The model calibration needs proper site characterization of field conditions, otherwise, the conditions after calibration might not be representative enough.

The traditional calibration is achieved via the trial-and-error method to adjust the model's input data, including aquifer properties, source and sinks, as well as boundary and initial conditions. This method is time-consuming and highly subjective as the factors interrelated with each other would influence the output. At the same time, it is hard to quantify the reliability of the calibration, thus, the predictions might be unreliable (*Konikow & Mercer, 1988*). For example, the MODFLOW-88 was used to construct the groundwater flow model in the Great Artesian Basin (GAB), Australia. The model was calibrated by the trial-and-error method, and hydraulic conductivities and recharges from precipitation were adjusted. The results of calibration showed that the Root Mean Square (RMS) error was 4.5 m with individual values ranging from -13.1 m to +12.8 m (*Zhou & Li, 2011*). In order to compensate the shortcomings brought by the trial-and-error method, parameter estimation was introduced in the calibration procedure of groundwater flow models (*Belcher & Welch, 2006; Konikow & Mercer, 1988*). It is a mathematical process to calculate inputs and make them be the best representative of the natural system. In addition, this technique could limit the influence of modelers' subjectivity and improve the efficiency.

Generally speaking, the parameter estimation method can be assorted into two classifications, respectively, automatic history matching (to calculate the best fit) and statistical evaluation of properties. The automatic history matching method can get the "best fit" between the observed and modeled values through estimating the system parameters. Firstly, a parameter set (e.g., transmissivity, storage coefficient, or stresses) should be selected to minimize the differences between observed and modeled data on the basis of sensitivity coefficients. For instance, if transmissivity (T) is selected, the minimization procedure would be in the form of $[\partial h(x,y,z,t)]/\partial T$, which means the change in hydraulic head is divided by the change in transmissivity (*Konikow & Mercer, 1988*). Next, the "truest model" should be chosen. Although the calculation procedure is objective, the selection of the "truest model" is a subjective task based on the hydrologic experience and judgment of the modeler. It is still largely influenced by human factor. Nevertheless, the statistical evaluation of properties (e.g., Kriging) is a more promising method. It enables to acquire the statistical properties directly from measurements rather than require initial estimations of parameters.

Currently, a non-linear parameter estimation and optimization package called PEST (Model-Independent Parameter Estimation & Uncertainty Analysis) was developed and has been utilized in multiple groundwater models (*Echo Valley Graphics, Inc., 2016; Kumar, 2002*). It could run the model automatically until the parameters are adjusted to make sure the deviations between model outputs and measurements are minimum in the weighted least-squares sense, even in the cases without model's source code. Since it was developed, PEST has been applied extensively around the world for automatic model calibration and data interpretation in groundwater and surface water hydrology, geophysics, as well as geotechnical, mechanical, and mining engineering. *Hughes & White (2014)* applied PEST software to calibrate the model by modifying hydraulic conductivities, specific storage coefficients, specific yields, evapotranspiration parameters, canal roughness coefficients (Manning's n values), and canal leakance coefficients.

In addition, a new approach for model calibration named Multi-Model Analysis (MMA) has been recommended recently. It was developed under the circumstances that limited understanding of groundwater systems, such as aquifer layers, boundary conditions, parameter distributions, and dominant stresses, leads to multiple plausible conceptual models. This method could evaluate those alternative models via calculating model-averaged quantities, including location and type of flow system boundaries, the definition of recharge areas, as well as variety interpretation of hydrogeological framework (*Poeter & Hill, 2007; Zhou & Li, 2011*). MMA has been utilized in the northern Yucca Flat area of the Death Valley Regional Flow System (DVRFS), USA. The result indicated the model uncertainty was more significant on predictive uncertainty in comparison with parametric uncertainty, and geological interpretations contributed more than recharge estimations on model predictions as well (*Ye et al., 2010; Zhou & Li, 2011*).

Sensitivity analysis is a common process to handle parameter uncertainty. It observes the relative variations of model outputs (normally hydraulic head, flow rate, or contaminant transport) responding to the changes of input parameters within a reasonable range (*Konikow & Mercer, 1988; Kumar, 2002*). Therefore, it could identify comparatively more sensitive parameters or inputs, which will require further characterization.

A traditional method for sensitivity analysis is by means of direct parameter sampling. It adjusts parameters in the manner of one-at-a-time (OAT), which means each time only one of the parameters is changed while others remain the same (*De Roover, 2015*). At the same time, the whole set of equations needs to be solved. The sensitivity coefficient for each parameter could probably be estimated based on a finite-difference approximation. This method can be considerable time and labor consuming. An alternative called adjoint sensitivity method was brought forward to carry out sensitivity analysis more efficiently. It allows users to determine the sensitivity of the selected system performance functions, such as Dirac delta function, by solving the corresponding adjoint variables of two system matrix equations instead. In this way, the sensitivity coefficients can be represented by the solutions of system matrix equations and derivatives of governing equations (*Konikow & Mercer, 1988*).

An example of sensitivity analysis was accomplished by *De Roover (2015)*. He used the OAT method to divide and multiply the four selected parameters by 2. In order to save running time, for each simulation, the selected parameter in every layer will be changed altogether. Thus, there would be eight models to run for sensitivity analysis. For each model grid, the change of hydraulic head was calculated as follow:

$$h_{ij,diff} = h_{ij,adj} - h_{ij,ref}$$

Equation 3

Where:

i, j = corresponding grid coordinates;

h_{diff} = difference in hydraulic head (L);

h_{adj} = hydraulic head resulting from parameter adjustment (L);

h_{ref} = hydraulic head resulting from reference situation (L).

In this case, the models were run as steady-state and would not stop running until the equilibrium of water balance was reached. And the closure criteria for each cell was when the residual head was 0.0001 m and the water balance was 10 m^3 (*De Roover, 2015*).

4.4.4 Modeling software – MODFLOW (Three-Dimensional Finite-Difference Ground-Water Flow Model)

Since the 1970s, the computer-based numerical groundwater flow models were extensively constructed and applied to describe the characters of groundwater system and simulate the response of groundwater to external changes. Among all those models, MODFLOW (*McDonald & Harbaugh, 1988*), originated by USGS in 1984, has become the most widely used and industrial standard for groundwater flow model in the global (*Belcher & Welch,* *2006; Hill et al., 2003; Kumar, 2002; Langevin et al., 2004; Ros, 2008; Storey et al., 2003; Zhou & Li, 2011*). It is a modular, three-dimensional, finite-difference (volume) groundwater flow modeling computer program, which gains its popularity on account of its flexible modular structure, complete coverage of hydrogeological processes, extensively documentation, public domain free availability, various simulation packages and utilities, as well as rigorous USGS peer review.

MODFLOW is a program which was designed to be modified, used, and maintained, as well as applied on different computer systems and able to manage large databases. The groundwater flow in the saturated zone can be simulated by MODFLOW using block-centered finite difference scheme, which involves a Main Program and a series of independent modules grouped into packages. For each package, it specifically simulates an external stress on groundwater flow such as flow to wells, areal recharge, evapotranspiration, flow to drains, and flow through riverbeds. At the same time, it could also solve the linear equation with a particular method like the Strongly Implicit Procedure or Preconditioned Conjugate Gradient (*Kumar, 2002*). The division of MODFLOW into modules allows users to test each hydrologic feature of a model independently and to add new modules or packages without extra modifying. This enables MODFLOW to achieve the optimal flexibility.

As mentioned above, MODFLOW has advantages consisting of facilities for data preparation, exchange of data forms, worldwide experience, continuous development, availability of code, comparably low price, and so on. It can simulate layers as confined, unconfined, or the combination of both (*Kumar, 2002*). And it can simulate groundwater flow for diverse water densities via formulating equations based on the equivalent freshwater head (*Langevin et al., 2004*). Hence, MODFLOW is popular to model water supply, containment remediation, and mine dewatering systems (*Kumar, 2002*). For instance, a 2D steady-state groundwater flow model called GABFLOW was developed on the basis of MODFLOW-88 to evaluate the effectiveness of the GAB Sustainability Initiative under different management scenarios through predicting the pressure recovery of Artesian groundwater (*Zhou & Li, 2011*). Since surface runoff and unsaturated flow are not involved in, MODFLOW-88 cannot simulate the transient flow when the flux depends on the calculated head and the function is unknown (*Kumar, 2002*). But MODFLOW-88 was keeping improved and updated to MODFLOW-2000 (*Harbaugh et al., 2000*) and MODFLOW-2005 (*Harbaugh, 2005*). In 2006, a 2D transient flow model based on GABFLOW and MODFLOW-2000 was created for the shallowest Artesian aquifer in the GAB. Yet, due to lack of historical discharge and water level data, it remains skeptical about the validity of the model predictions (*Welsh, 2006; Zhou & Li, 2011*). Another transient groundwater flow model based on MODFLOW-2000 was constructed by *Faunt et al. (2010)* to simulate the Death Valley regional groundwater system. The results indicated that the model could create good simulations where the hydraulic gradients were flat, while poor simulations where the hydraulic gradients were steep.

MODFLOW-2005 is the latest version of MODFLOW. It newly comprises processes for saturated-unsaturated flow simulation, groundwater simulation-optimization, irrigation, density dependent flow, parameter optimization, and solute transport (*Zhou & Li, 2011*). An example of the application of MODFLOW-2005 was to develop the MODFLOW-NWT model by *Niswonger et al. (2011)*. They designed the model to solve the drying and rewetting nonlinearities of the flow equations in the unconfined aquifer. With additional assistance from the Surface-Water Routing (SWR1) Process (*Hughes et al., 2012*) and the Seawater Intrusion (SWI2) Package (*Bakker et al., 2013*), MODFLOW-NWT is able to identify the effect of fluid density on groundwater flow and the position of the freshwater-seawater interface. They make MODFLOW-NWT become a suitable model for simulation groundwater flow system in the coastal areas, with regard to surface water as well (*Hughes & White,* *2014*). MODFLOW still requires further development, which will enable it to be a model for the integration of surface and groundwater systems in the future (*Barlow & Harbaugh, 2006*).

5 Methodology and Data Collection

5.1 "No-regret" groundwater level observation network design

Since controlling groundwater level is considered as an efficient intervention to alleviate land subsidence indirectly, it is important to have a full assessment of groundwater flow conditions in temporal and spatial aspects. This is far from possible with three groundwater observation wells which have already been existed in the research area (see *Figure 2*). Therefore, a more representative observation network on groundwater level is necessary.

The primary objectives to design a dense groundwater observation network are: 1) to investigate how fast and how far the external factors (mainly precipitation, evapotranspiration, surface water, and sewage water in this case) could influence groundwater levels; 2) to understand the impacts from groundwater levels in the vicinity of those more vulnerable places, such as wooden poles, crawl spaces, house backyards, as well as relatively higher elevations of peat layers; 3) to study whether the heterogeneity of the subsurface material and properties could explain the differences in groundwater levels spatially; and 4) to analyze the flow patterns of groundwater in wet and dry periods around the year. 15 extra observation wells were installed in the research area on May 20th, 2016, by the company Wareco Ingenieurs. Considering the above objectives, the selection of well locations was based upon the influential factors on groundwater, susceptible areas to groundwater level fluctuation, and accessibility for subsequent validation and maintenance.

From previous internship investigation, the groundwater levels measured at those existed three wells responded relatively fast to the changes on sewage water levels (see *Figure 6*). The greens lines are the groundwater levels measured at each observation well, and the blue lines are the sewage water levels measured at the CSO 703. It indicated that at least the sewer pipes near to those wells were relatively leaky, and they were not "assumed" to be watertight anymore. Just like mentioned before, the results from merely three measurement points cannot be representative. Additional measurement points near sewer system should be compromised in the newly designed network. At the same time, in order to look into how far the sewage water could influence groundwater level once it is changed artificially, it would also be useful to put several observation wells in a line which is perpendicular to the same sewerage pipe. In addition, other "universally-known" factors should be taken into consideration, in order to have a comprehensive understanding of groundwater flow and fluctuation patterns in the whole area of interest. Those factors include precipitation, evapotranspiration, and surface water level.

Since the research area is rather small, the inhomogeneity of precipitation in space could be neglected. However, the amount of precipitation which could infiltrate and recharge to the groundwater varies on the basis of land cover. For example, the recharge rate of grassland would be higher than that of trees owing to the interception effect of tree leaves. In this research, the land use is separated into grassland, tree, brick pavement, the building roof, asphalt pavement, and canal (see *Figure 7*). And the reaction of groundwater with different land covers after a rainfall event is something needed to be analyzed. Similar to precipitation, the potential evaporation is hardly inhomogeneous in such a small scale. When it comes to the water balance in the urban area, evapotranspiration becomes more significant (*Van de Ven, 2016*). In general, evapotranspiration refers to transpiration from vegetation whose roots could extract groundwater for their own growth, as well as evaporation by capillarity in the unsaturated zone. Except for weather conditions, the transpiration rate would be diverse due to the plant type and soil moisture availability (*USGS, 2016*). Yet, in the highly urbanized city like Gouda, it is nearly impossible to tell whether the trees would extract a large amount of groundwater and therefore, lower the groundwater level, especially in the hot summer periods. Wells located near different types of vegetation (e.g., trees and grassland) would be helpful.

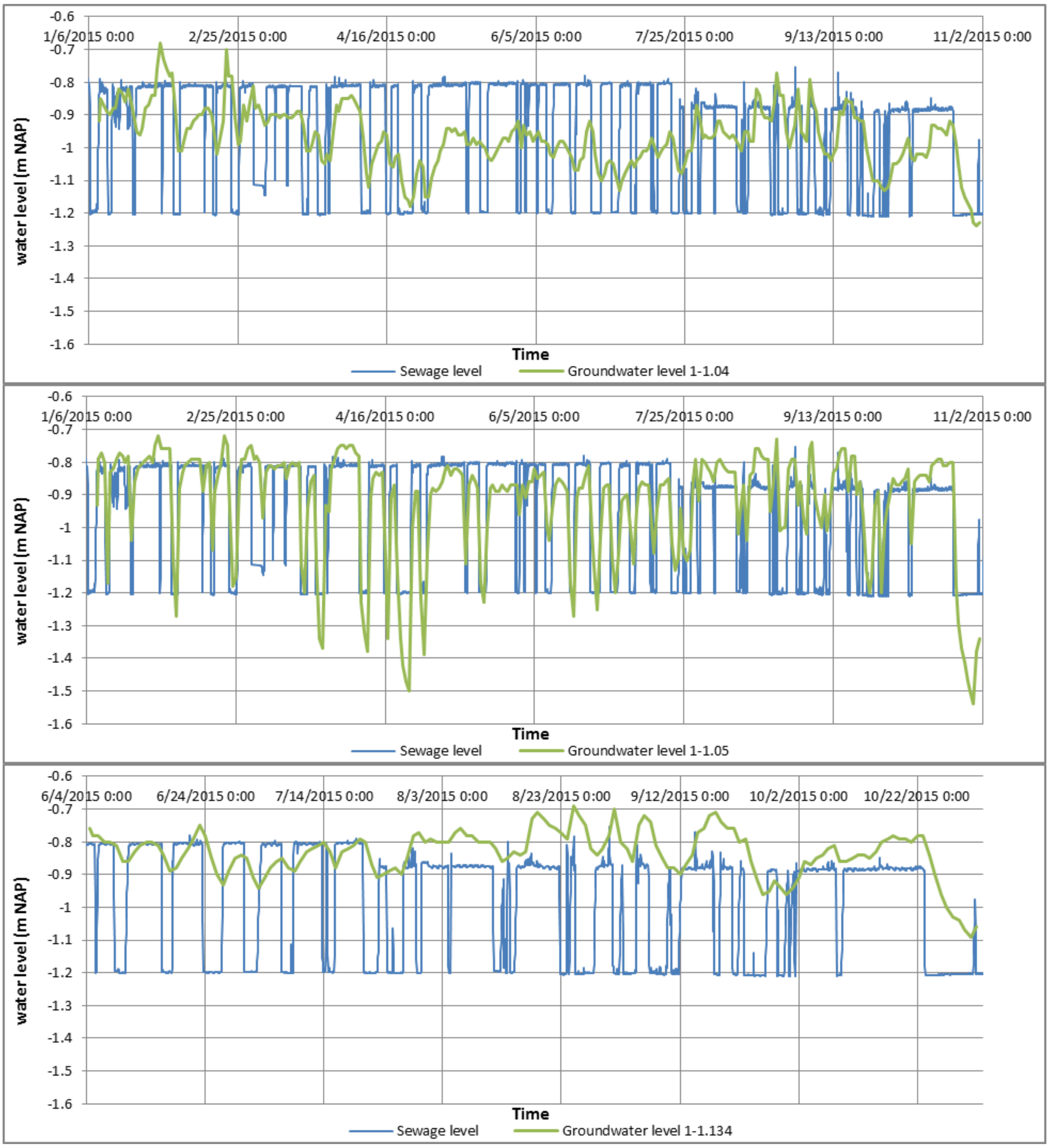


Figure 6 Relationships between groundwater levels and sewage water levers for Well 1-1.04, 1-1.05, and 1-1.134

Since the surface water levels are controlled by the pumping stations at the level of -0.72 $m\ NAP$ and fluctuate really small around the year, it was believed that surface water level had little influence on groundwater level before the project started. However, from *Figure 8*, it can be seen that during most of the time along a year, the groundwater levels are beneath the surface water levels. It seems there is an interaction between groundwater and surface water. This cannot be verified since all these three wells are not in the vicinity of canals. Hence, some wells are proposed to install near the canal Turfmarkt, where the surface water level need to be controlled precisely, to investigate the relationship between groundwater and surface water levels. As far as known, there is no extraction activity at least in the inner city. But in the private lands, such as backyards, when they are getting flooded because of high groundwater water after a rainfall event, some inhabitants would use diminutive pumps to get rid of excess groundwater. This is also an aspect to be taken into consideration for the network design.

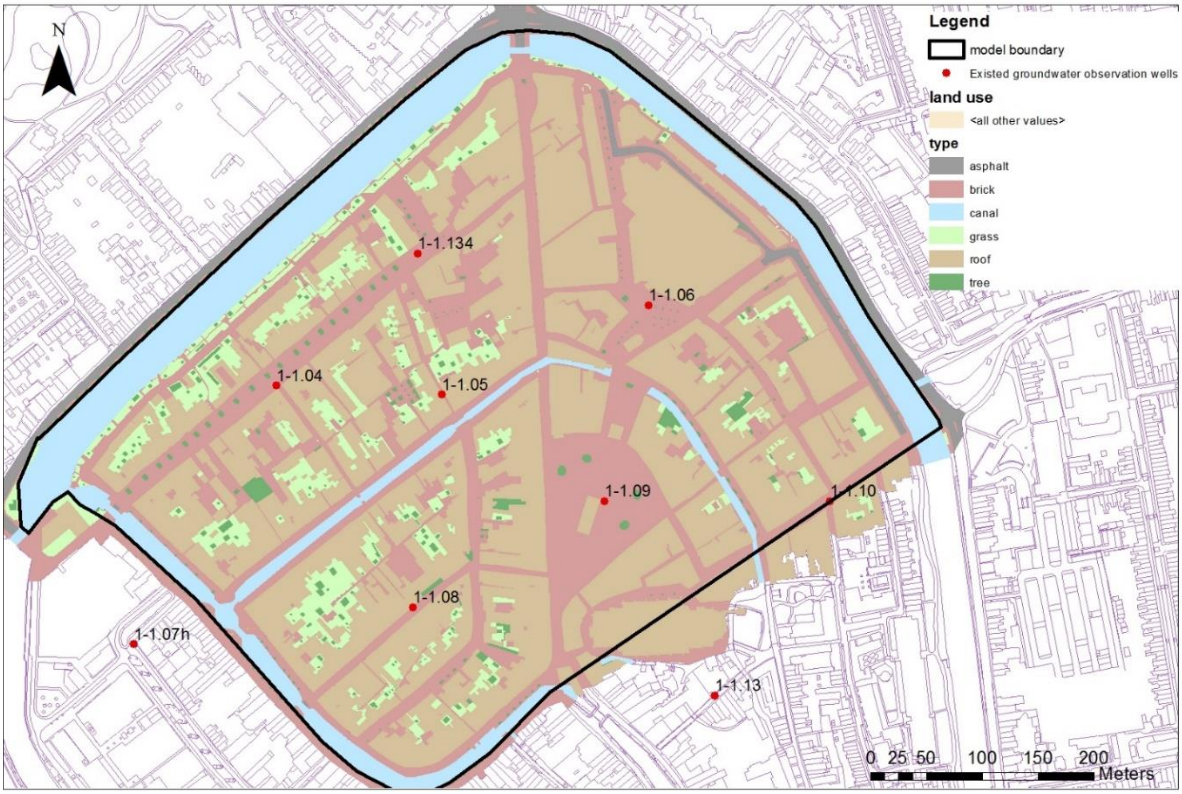


Figure 7 Land use and existed groundwater observation wells in the model area

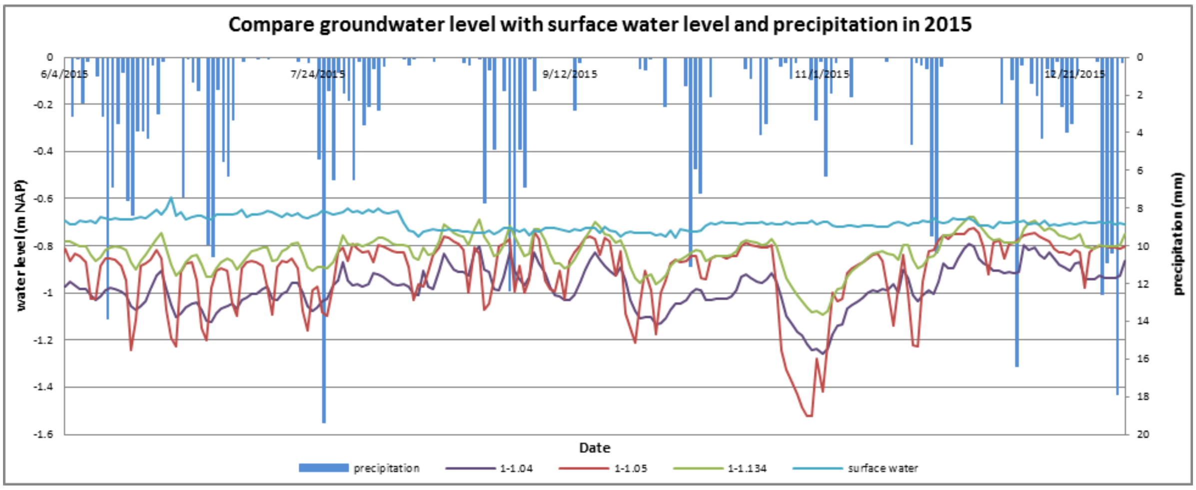


Figure 8 Compare groundwater level with surface water level and precipitation in 2015

In this research, the susceptible areas to groundwater level fluctuation generally consist of the areas with a relatively high elevation of the top of the peat layer, premises with wooden foundations, and crawl spaces underneath the houses. In 1993, Wareco Ingenieurs did a project in the Nieuwe Haven and Centrum areas. 17 boreholes were dug until 2 to 4 m below surface level, and most of them reached the peat or clay layer. Moreover, in 1994, Wareco Ingenieurs installed another 18 boreholes along the street Nieuwe Haven, and they were 4 to 5 m deep. The borehole information could be retrieved from a report by *Den Nijs (2015)*. Unfortunately, this information was collected around 20 years ago. During this 20-year time period, land subsidence occurred and the groundwater levels were regulated correspondingly. Not to mention that some of the borehole information is not available anymore. Thereby, it might be an existence of uncertainty in the data collected 20 years ago if it is going to be applied to represent the current circumstances. Furthermore, the top elevation of the peat layer is distributed relatively evenly (around -2.5 $m\ NAP$) across the whole area, according to the subsurface model GeoTOP v1.3 built by DINOlocket (Data and Information on the Dutch Subsurface, https://www.dinoloket.nl/en) (see *Figure 9*). Ergo, the elevation of peat layer is not the major element taken into consideration for the network design.

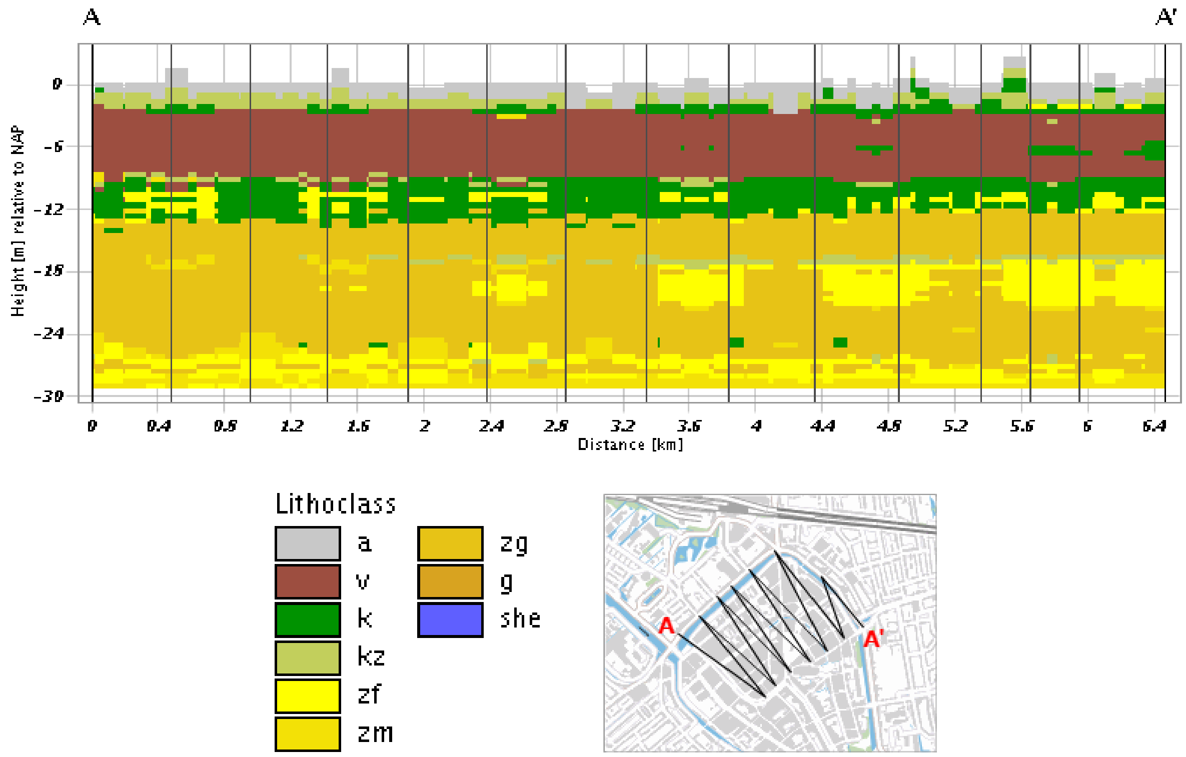


Figure 9 Vertical distribution of lithological class until -30 $\boldsymbol{m\ NAP}$ from GeoTOP v1.3 DINOlocket (https://www.dinoloket.nl/en/subsurface-models)

The information about foundation types in the research area is still a lack of sufficient investigation and comprehension. According to an inquiry conducted by *Den Nijs (2015)*, the foundation types of several buildings in the research area have already known about (see *Figure 10*). And for the wooden foundations have been identified on the map, it is also cognizant of the distances between the top of the wooden poles with respect to sea level or floors. In *Figure 10*, for example, “W_-0.9_SL” means the top of the wooden poles are at the elevation of -0.9 $m\ NAP$, “W_1.5_Flo” means the distance between the top of the wooden poles and the floors is 1.5 m), “S” refers to the foundation type “op staal” (see *Chapter 4.2.1*), and “N” refers to new foundations, which were probably made by concrete. As for crawl spaces, the comparison between groundwater level in front of the building and that in the backyard would be useful to indicate the inundation condition of the crawl spaces. Thus, wells located in the vicinity of houses and backyards were proposed.

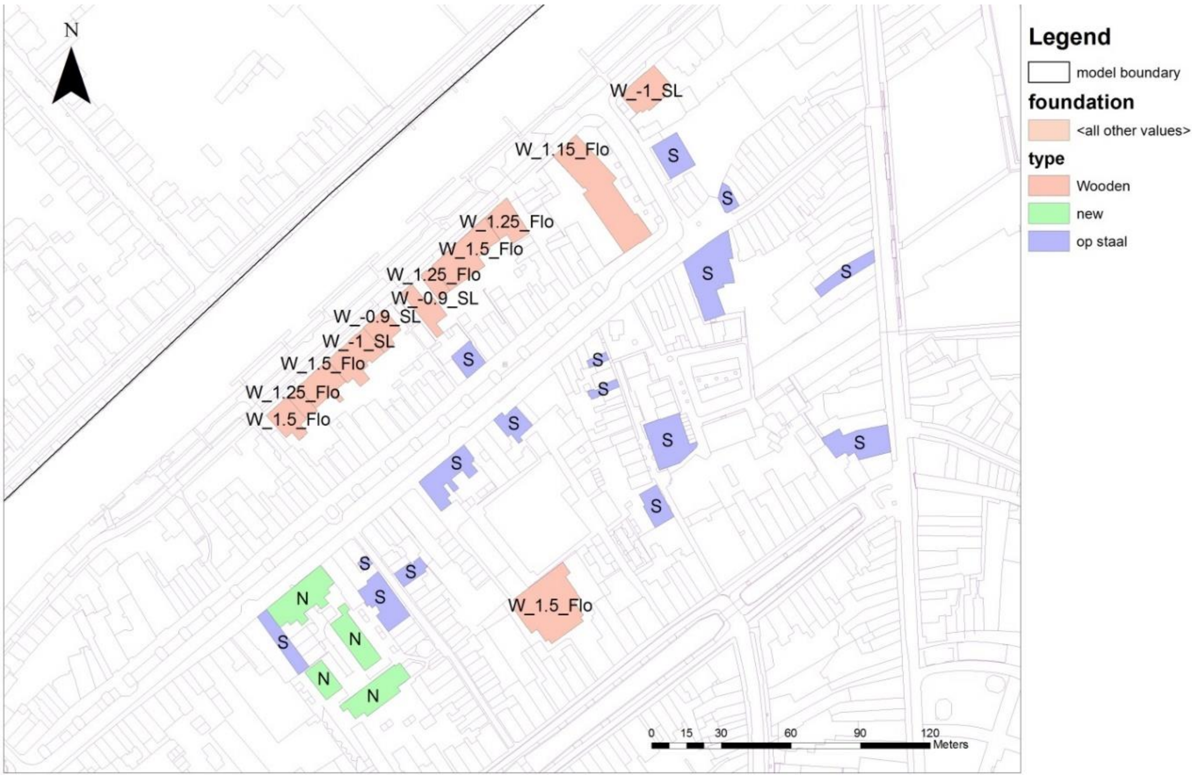


Figure 10 Already known foundation types in the research area (Den Nijs, 2015)

In summary, the choice for the well locations of the “no-regret” groundwater observation network takes in account of the land use, plant types, sewer system, surface water, foundation types, crawl spaces, and backyards. Ideally, each well should represent on factor and prevent the impact from other factors. To minimize the workload and financial cost, some wells are able to combine multiple functionalities as long as they do not have conflicts with each other. In spite of the adequate data, the success of the network design also depends on the cooperation and coordination among the scientific institutions, Gemeente Gouda, Wareco INGENIEURS, and local inhabitants. The final design results of the groundwater observation network are demonstrated in *Chapter 6.1*.

5.2 Field experiment

There are two kinds of field experiments implemented during the research period. One is slug test, which provides requisite data (e.g., hydraulic conductivity) for the groundwater flow model construction. Another one is to bring down the sewage water level by opening the weir, which is used to investigate the interactions between groundwater level and sewage water level. The objectives, requirements, and methods for each field experiment are explained in this chapter.

5.2.1 Slug test

The slug test is a controlled field experiment to determine the transmissivity or hydraulic conductivity of aquifers and aquitards. Usually, a small volume (or slug) of water is removed from or injected into a well, then the rate of water level rising or dropping back until the initial level reaches will be measured (*Kruseman & de Ridder, 1990*). The measured data – rate of water level change – is a function of the hydraulic conductivity and the geometry of the well or the screened interval (*Ohio EPA, 2006*). Therefore, the slug test requires a sudden change of the water level and rather frequent measurements of water level subsequently. In case the manual measurement is not accurate enough when the water level recovers too quickly, both the manual measurement technique and an automatic pressure transducer with a data logger will be applied for each measurement points. According to the vertical position of the well screens underneath the surface, 11 observation wells were selected to do the slug test. The detailed explanations and results of slug test will be in *Chapter 6.2.1*.

Comparably, slug test is easy to operate, inexpensive, time saving (usually several minutes), and only needs simple equipment. Thus, it is commonly used for a preliminary estimation of the aquifer conditions (*Kruseman & de Ridder, 1990*). Although it cannot be considered to replace the conventional pumping tests, *Moench & Hsieh (1985)* and *Ramey et al. (1975)* still thought the results from slug test were still accurate enough for the transmissivity estimation.

Since all the wells within the model boundary partially penetrate the unconfined aquifer, *Bouwer & Rice (1976)* method could be applied for data analysis. It is a method based on Thiem's equation and assumed that water level returns to the equilibrium level exponentially without the influences from inertial forces. The equations to calculate hydraulic conductivity K $[L/T]$ are showed below (*Kruseman & de Ridder, 1990*):

$$K = \frac{{r_c}^2 ln(R_e/r_w)}{2d} \frac{1}{t} ln \frac{h_0}{h_t}$$

Equation 4

For partially penetrating wells:

$$ln \frac{R_e}{r_w} = \left[\frac{1.1}{ln(b/r_w)} + \frac{A + Bln[(D-b)/r_w]}{d/r_w} \right]^{-1}$$

Equation 5

For fully penetrating wells:

$$ln \frac{R_e}{r_w} = \left[\frac{1.1}{ln(b/r_w)} + \frac{C}{d/r_w} \right]^{-1}$$

Equation 6

Where:

r_c = radius of the unscreened part of the well where the head is rising $[L]$ (see *Figure 11*);

r_w = horizontal distance from well center to undisturbed aquifer $[L]$ (see *Figure 11*);

R_e = radial distance over which the difference in head, h_0, is dissipated in the flow system of the aquifer $[L]$;

d = length of the well screen or open section of the well $[L]$ (see *Figure 11*);

t = time after starting the measurements $[T]$;

h_0 = head in the well at time $t_0 = 0$ $[L]$;

h_t = head in the well at time $t > t_0$ $[L]$;

A, B, C = dimensionless parameters, which are functions of d/r_w [-] (see *Figure 12*).

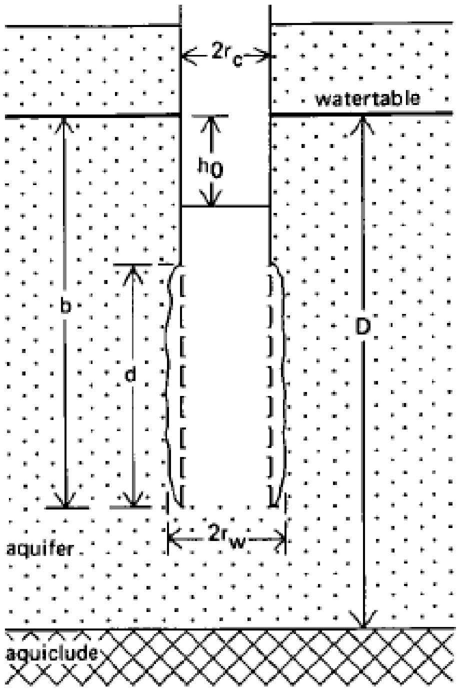


Figure 11 Illustration of geometrical parameters of a partially penetrated unconfined aquifer (Kruseman & de Ridder, 1990)

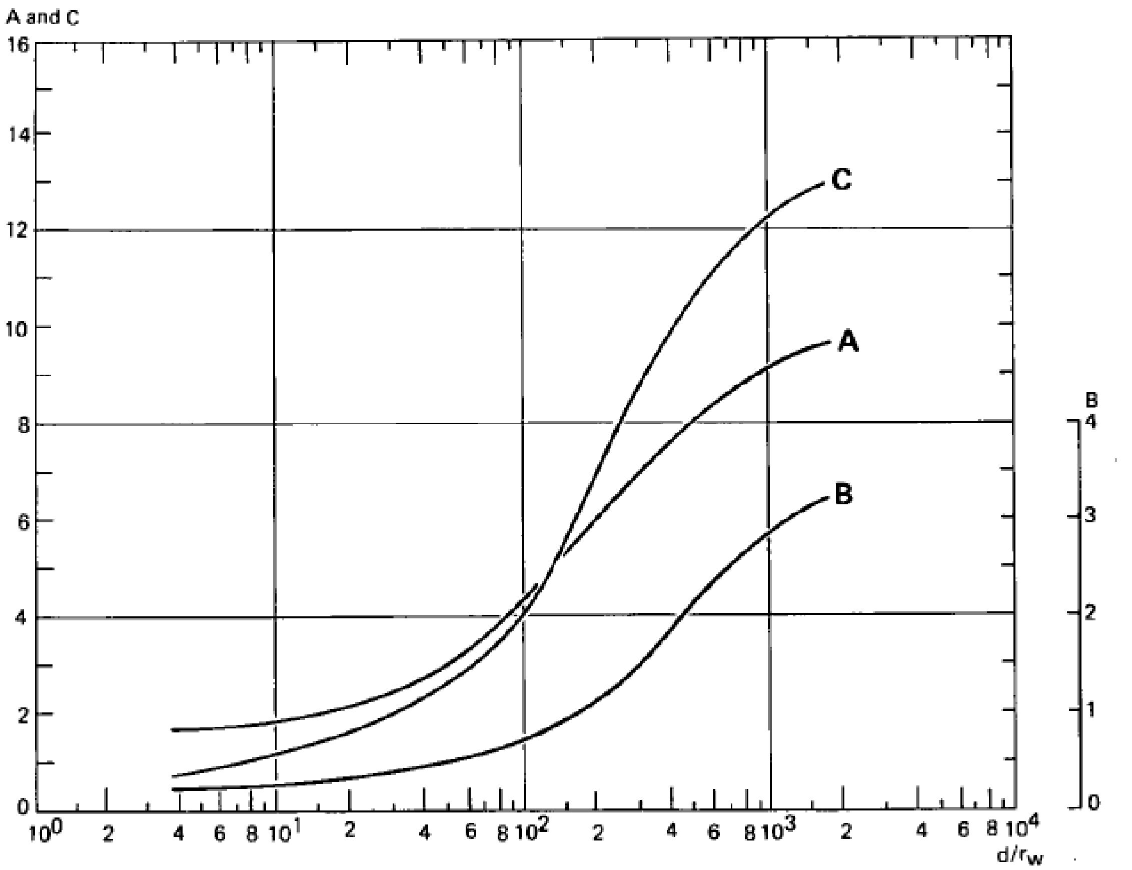


Figure 12 The Bouwer & Rice curves showing the relation between the parameters $\boldsymbol{A, B, C}$ and $\boldsymbol{d/r_w}$ (Kruseman & de Ridder, 1990)

5.2.2 Open the weir in the sewer system

There are four types of sewer systems in the research area, the back-stowed system ("Opgeboeid" in Dutch), pumping system ("Bemalen" in Dutch), storm water system (RWA, "Regenwaterriool" in Dutch), and flushing system ("Spoelleiding" in Dutch), respectively. The back-stowed system collects waste water from the houses and storm water from the streets then transfers to the pumping station via the pumping sewer system. Between the back-stowed system and pumping system, there are several weirs to maintain certain sewage water levels in the back-stowed system (see *Figure 2* and *Figure 13*). It has been proved that the back-stowed sewer pipes around existed groundwater observation well 1-1.04, 1-1.05, and 1-1.134 are leaky and have a more obvious impact on the groundwater level than precipitation and evapotranspiration. Whilst, the result was not convincing enough to indicate all the back-stowed system is leaky. The purpose to do the field experiment is to prove whether other parts of the back-stowed system are leaky as expected before, using the collected data from the more intensive groundwater observation network (see *Chapter 5.1* and *6.1*).

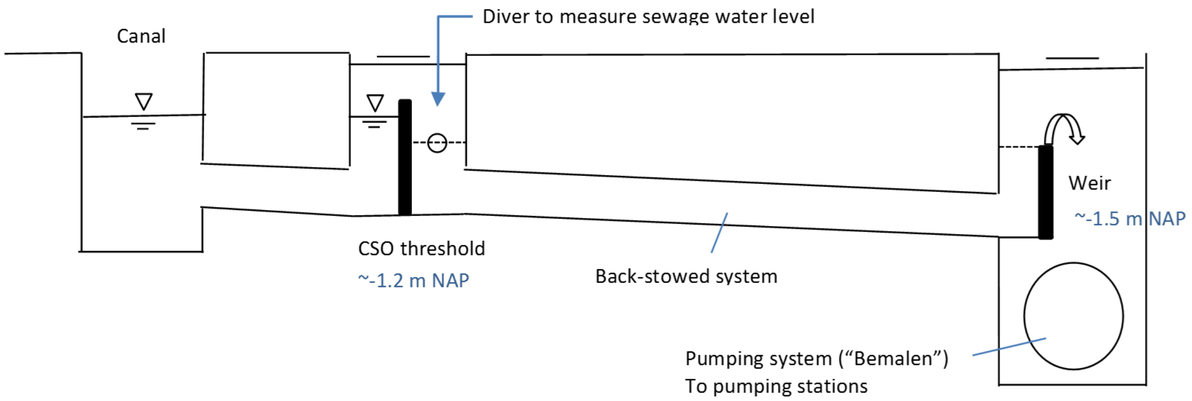


Figure 13 Illustration on the locations of weirs between the back-stowed system and pumping system, as well as the measurement location of sewage water level

The company Cyclus is responsible for opening or closing the weirs between the back-stowed and pumping systems every now and then, based on the requirements from Gemeente Gouda, local residents, and construction sites. Once the weir is open, the sewage water in the back-stowed system will flow into the pumping station, and the sewage water level will fall immediately. If the sewer pipes are leaky, the groundwater supposedly infiltrates into the sewer system, and groundwater level will drop correspondingly. It should be noticeable that the differences of the sewer bottom elevations at the CSO threshold and at the weir (see *Figure 13*). At the CSO threshold where the sewage water levels are measured, the bottom of the sewer pipe is at around -1.2 $m\ NAP$, yet, it is at around -1.5 $m\ NAP$ at the weir. It illustrates that when the weir is fully open and the sewage water drops down to the bottom at the weir (-1.5 $m\ NAP$), the measured sewage water level could only reach -1.2 $m\ NAP$. An additional data logger is suggested to install at the weir to have a more accurate database for the further investigation.

With the help from Cyclus and Gemeente Gouda, the weir can be opened for maximum two days. In the interest of studying the impact from sewage water on groundwater level alone, it is important to avoid other natural or anthropogenic factors that may affect groundwater level as much as possible. Hence, the field experiment was carried out during a dry period, at least there was no rain for more than two days. Cyclus, Gemeente Gouda, and Hoogheemraadschap van Rijnland checked the weather forecast constantly to select a suitable period, which could satisfy the experiment requirements. By comparing the sewage water levels measured at each CSO and groundwater data collected by observation wells, it will not be hard to tell whether all the sewer lines are leaky, where leaky pipes are located, how the extent of damage is, as well as how fast the groundwater responds to the change of the sewage water level.

Owing to the fact that no matter how accurate the weather forecast could be, it would still be possible to rain during the experiment. And it seems kind of effortless for Cyclus to open the weir. It is suggested to do this experiment several times to compare the results to prevent external influences, as long as it will not disturb normal life of local residents and working schedules of local companies.

5.3 Groundwater flow model

On account of the fact that groundwater flow pattern is rather invisible by human being yet evitable for urban water resources management, a groundwater flow model will be designed to take the first step for understanding urban geohydrology in the research area. A 2D layered finite-difference transient groundwater flow model was built with the help of iMODFLOW v2.6.37, which is an accelerated Deltares-version of MODFLOW (*Lambert et al., 2015; Ros, 2008; Vermeulen et al., 2016*). The main purposes of this model are: 1) to comprehend the groundwater flow behavior under the influence from different external stresses, especially to find out if the model could simulate what has been observed from the field experiment and cross-section analysis; 2) to indicate how well a regional groundwater flow model such as iMODFLOW is able to simulate the groundwater situation happened in a local area; and 3) to analyze possible interventions for future management and their potential consequences, which will be achieved in the further investigation. Since iMODFLOW has not been developed to support hourly model simulation so far, a daily model will be designed for the research area. Addition information about iMOD along with iMODFLOW can be found in *Appendix 4 Introduction of iMOD interface and iMODFLOW*.

5.3.1 Model conceptualization

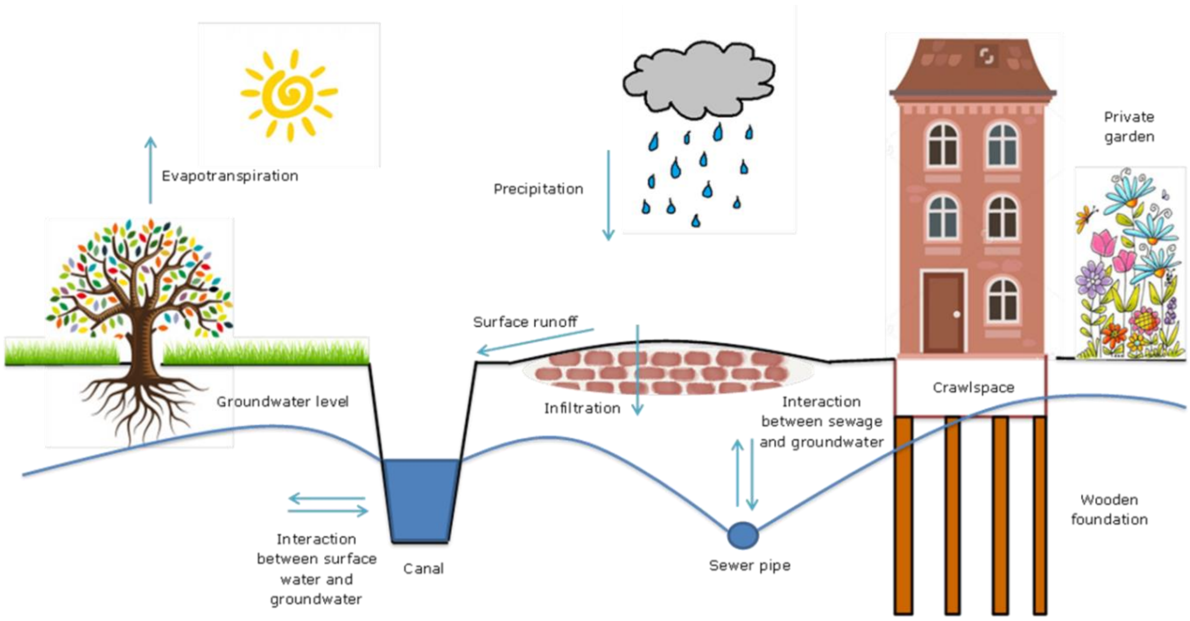


Figure 14 Scheme of conceptual model

The same as the groundwater level observation network design, the design of the groundwater flow model requires to consider the possible external factors on the groundwater level, which are recharge due to precipitation, evapotranspiration from vegetation and unsaturated zone, recharge or discharge via leaky sewer system, and interactions with surface water (see *Figure 14)*.

When it starts to rain, part of the water from precipitation will flow to surface water via surface runoff, part of water will be intercepted by leaves, roofs, asphalt, and so on then evaporate entirely, and the last part of the precipitation could infiltrate to the underground and contribute to the groundwater through the unpaved areas and paved areas with less impermeability, such as bricks. Vegetation will extract groundwater for their own growth, especially in the hot and dry period (at the end of the simulation period), the transpiration effect from those relatively big trees with deep roots would lead to a large amount of groundwater loss. The interactions between groundwater and surface water would be less obvious in the urban area than the natural environment since the canals are artificially built with concrete and bricks. In addition, if the sewer lines are leaky, the sewer system will be functioned as drainage system when the groundwater level is higher, and as recharge source when the groundwater is lower. From the previous investigation, sewage water level has a big influence on groundwater level.

5.3.2 Model design

In the iMODFLOW, a runfile is required to initiate the groundwater flow model simulation. It gives an overview of the model configuration, such as the location of the model, number of stress periods, grid size, model layers, parameters, and output variables (*Vermeulen et al., 2016*). Furthermore, the geographic information system (ArcGIS) and iMOD 3.4 interface were used to make sure the accurate spatial control of physical features and finite-difference model grids for the model input. And they were also applied for comparisons between model results and real measurement data, as well as results from sensitivity tests.

5.3.2.1 Assumptions and simplifications

Inevitably, assumptions and simplifications are required to transfer the complicated conceptual model into the numerical simulations which can be operated by the program (*Faunt et al., 2010*). The following simplifications and assumptions were applied in the model design:

1. The groundwater within the model boundary is assumed to have little interactions with the groundwater in the other polders nearby. Because the most parts of the model boundary are surrounded by canals (see *Figure 7*), and in this model simulation, the surface water levels in those canals are constant over time. Although this is not necessarily true, as those canals can be functioned as a General Head Boundary (GHB), which presumes an unlimited source of water to maintain the same water level all the time, this assumption was considered to be tenable in this case.
2. The vertical stratification of soil structure is assumed to be “course sand – peat and clay – fine sand” layers in order. From *Appendix 1 Borehole information*, for instance, there is around 35 cm of clay existing in the middle of the course sand layer at the Well 05, those part of clay would be neglected in the model layer division. And it would be the same case for other boreholes which have a similar composition.
3. Horizontal hydraulic conductivity (permeability) K_{xy} is assumed to be isotropic within a model cell. For the first model layer, the K_{xy} values for the whole area are interpolated using the Inverse Distance Weighted (IDW) technique in accordance with the results of the slug test (see *Chapter 6.2.1*). The vertical permeability K_z is calculated from K_{xy} by multiplying the vertical anisotropy (0.2 in this case). The

possible consequence of this assumption is that a sudden change of K value within a cell would contribute to being isotropy. However, the subsurface stratification is almost the same in the research area, and the cell size is rather small, the occasion like this is barely possible to occur.

4. The density and viscosity of groundwater are considered to be constant everywhere, in spite of that the contamination and temperature changes from the leaky sewer system would change them in space and time terms. Based on Darcy's law, both density ρ and viscosity μ have impacts on the flow rate per unit surface area q (*Mulligan & Charette, 2009*). In this case, the differences in density and viscosity in the terms of space and time would be neglected.
5. Since the previous investigations proved that the back-stowed system is leaky, in the model simulation, it is assumed that all the back-stowed pipes are leaky and have the same extent of the damage. As for other types of the sewer system, which were relatively new, they are assumed to be completely water-tight.
6. The surface water level will be assumed to maintain the same level during the simulation. Although the surface water level fluctuates all the time, it is still controlled nearly to -0.72 $m\,NAP$ by three pumping stations and only varies between a few centimeters (see *Figure 8*). In addition, there has not been a diver to measure the surface water level at the Turfmarkt canal, where the fluctuation of water level needs more attention. The data measured in front of the pumping stations cannot represent the real situation at Turfmarkt.
7. Besides precipitation, evapotranspiration, surface water, and sewage water, other potentially influential factors, such as human interventions, will not be taken into consideration for the model.

5.3.2.2 Spatial and temporal discretization

The groundwater flow model consists of 356 rows, 430 columns, and 3 layers, for a total of 459,240 cells covering an area of 0.61 km^2. Owing to the close distances between Well 01, 02, and 03 (between around 5 to 8 m, see *Figure 23*), uniformly sized model cells were set as 2 m × 2 m to prevent there would be two observation wells in one model cell. In the Amersfoort / RD New coordinate system, the model area spans from X_{min} = 108020 m to X_{max} = 108880 m and from Y_{min} = 446977.4 m to Y_{max} = 447689.44 m. On the basis of the vertical stratification of soil structure, the model would have three layers, course sand, peat and clay, and fine sand, separately (see *Figure 15*).

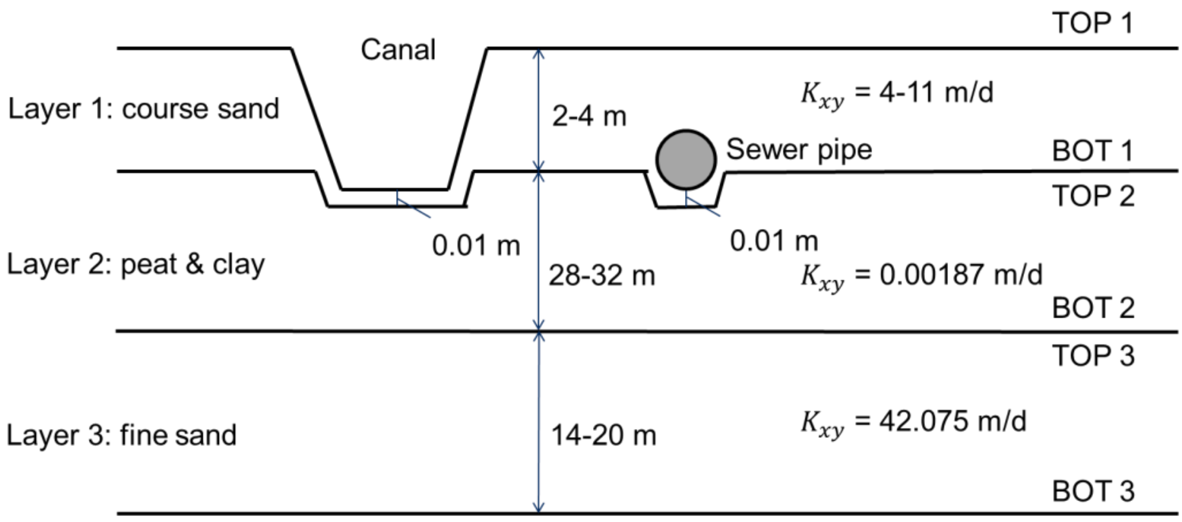


Figure 15 Sketch of layer division for the model

The top of the first layer (TOP1) is the combination of surface elevation interpolated based on AHN3_05m_dtm (see *Figure 1*) and the bottom of the canals, with a resolution of 0.5 m × 0.5 m. The bottom of the first layer (BOT1), which is also the top of the second layer (TOP2), is the interpolation in accordance with the available borehole information by the IDW technique in ArcGIS, with a resolution of 0.5 m × 0.5 m as well. Because the bottom of the canals is deeper than the interpolation result for TOP2, and the thickness of each layer should be at least 0.01 m, the elevation of TOP2 was adjusted accordingly. At the same time, some sewer pipes were built in the second layer, yet, there is a thin sandy layer beneath those pipes. Hence, the elevation of TOP2 was corrected to be lower than the bottom elevation of the sewer system. The bottom of the second layer (BOT2, the top of the third layer TOP3 as well) and the bottom of the third layer (BOT3) were collected from DINOlocket with a resolution of 100 m × 100 m.

Making use of the iMODFLOW, a daily groundwater flow model was designed for the research area from 16-04-2016 until 31-07-2016, in total 107 transient one-day stress periods. Nevertheless, all the new groundwater observation wells were finished installing equipment and started to measure since 20th of May. Hence, the model results from 20th of May until the end of July are the main simulations comparing to the observation values, which would be used for model calibration and verification.

5.3.2.3 Lateral boundary condition

It has been decided that the surface water level was assumed to be a constant head during the simulation, which provides a natural boundary condition for the model. Therefore, there would be three sides of the model boundary surrounded by canals (see *Figure 7*). On the other side of the model boundary, part of it is bounded by a canal, and another part of it is bounded by the street Lange Tiendeweg. At the same time, the Well 1-1.10 is located at this street, and it can be used to interpolate the boundary condition along this street.

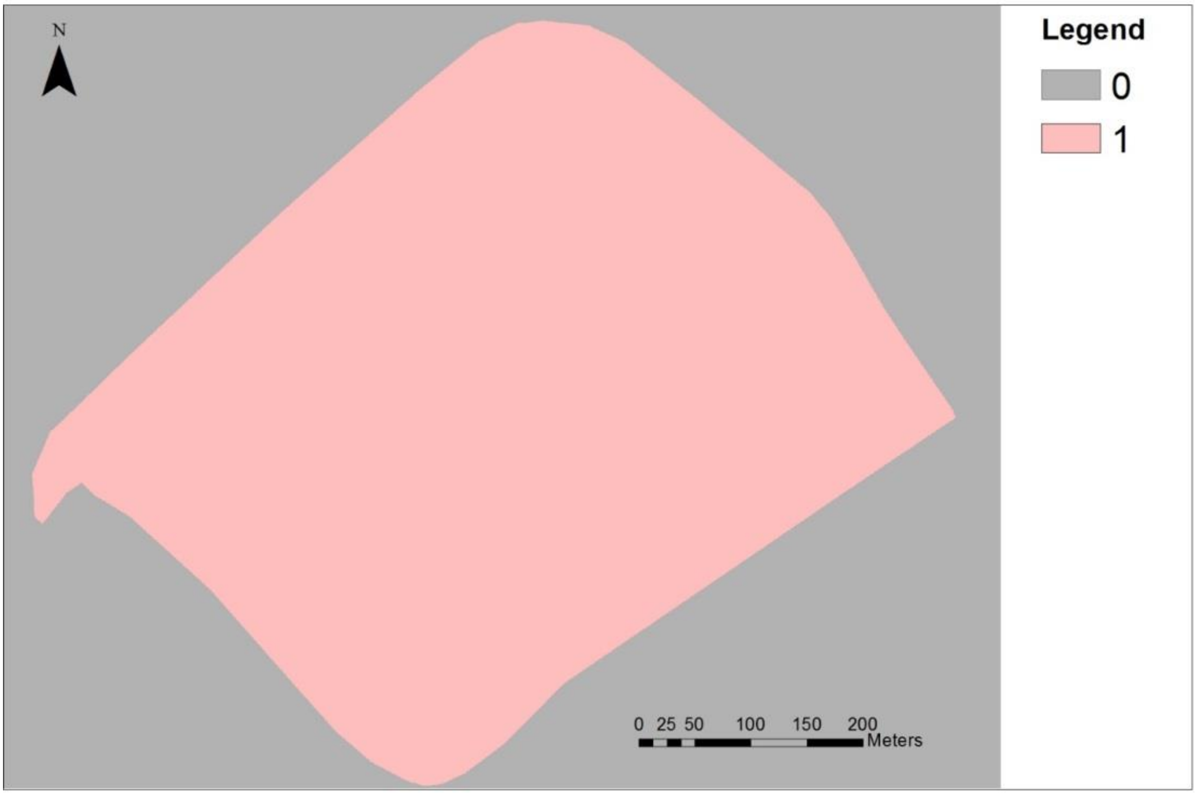


Figure 16 Boundary condition for the model simulation

According to the settings for the BND (Boundary Conditions) Module in the iMODFLOW (*Vermeulen et al., 2016*), the cells located outside of the model boundary were assigned to be “0”, which denotes they are excluded for the simulation, and no groundwater flow will go through those areas. And the cells inside of the boundary were assigned to be “1”, which means these cells will take part of the simulation and the groundwater head will be computed (see *Figure 16*). Though there is no fixed boundary condition connected to the activated area, the RIV (River) Package (see *Chapter 5.3.2.5*) makes sure the surface water level will not change during the simulation.

5.3.2.4 Initial condition

The initial head for the first layer is based on the measured groundwater level at each observation wells. All the new wells started to measure the groundwater since 20th of May, which was the starting time for the model results to be compared to. The model requires some time to adjust itself and start to simulate the real situations due to the influence of unsaturated zone, thus, the starting time of simulation should be at least one month in advance. From *Appendix 3 Hourly time series data of groundwater levels comparing to precipitation and sewage water levels*, the groundwater levels at the existed observation wells on 16th of April were similar to the groundwater levels around 20th of May. Thereby, the groundwater levels on 16th of April were selected to interpolate with groundwater levels on 20th of May, which generates the initial groundwater head. This, together with the surface water level, became the final initial head for the first layer (between -0.7 to -1 $m\ NAP$).

The initial head for the third layer is a clipping from a regional groundwater flow model built by Hoogheemraadschap van Rijnland, the groundwater level is between -3 to -5 $m\ NAP$. In addition, the initial head for the second layer is the average value between the first and the third initial heads. The initial heads for each layer correspond to the SHD (Starting Head) Module in the iMODFLOW.

5.3.2.5 Hydraulic properties (input modules and packages)

CAP Unsaturated zone Module

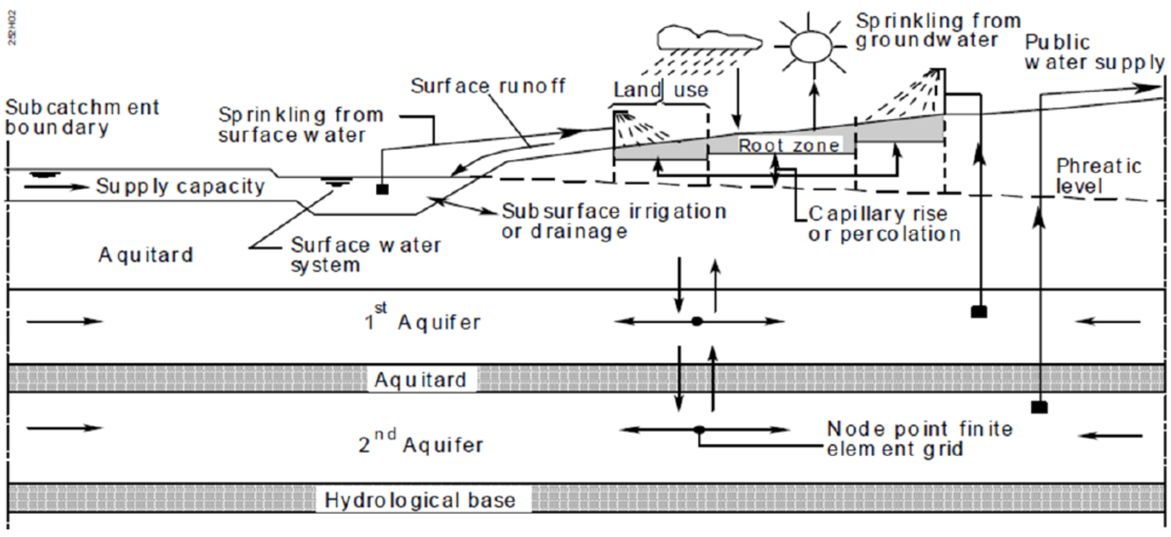


Figure 17 Overview of the processes and components in the SIMGRO model code (Vermeulen et al., 2016)

The CAP module is used for simulating the processes of groundwater recharge and discharge through the unsaturated zone with the help of MetaSWAP. MetaSWAP was developed as a sub-model in the SIMGRO (a dated acronym of SIMulation of GROundwater) code. The SIMGRO is an integrated model code intending for the regions with an undulating topography and unconsolidated sediments in the shallow subsoil. It can cover the whole system, comprising plant-atmosphere interactions, soil water, groundwater, and surface water (see *Figure 17*). As for the MetaSWAP, it models one “in-house” component of SEMGRO – SVAT (Soil Vegetation Atmosphere Transfer) process – for the unsaturated zone within vertical columns, which only comprises the plant-atmosphere interactions and soil water (see *Figure 18*). In the light of the simplification of “straight Richards”, MetaSWAP does not model any special processes like hysteresis, preferential flow, and bypass flow. Consequently, MetaSWAP excludes the influence of snow and frost on the soil water conductivity, the interflow groundwater and perched water table, and typical processes for steep slopes, etc. However, MetaSWAP is a suitable tool to simulate unsaturated zone for either shallow or deep groundwater levels (*Vermeulen et al., 2016*).

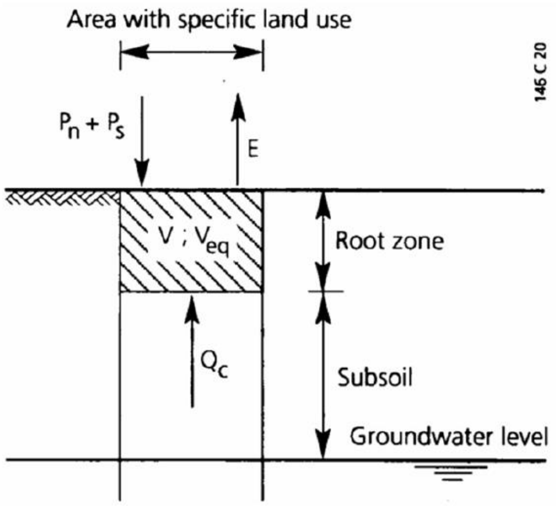


Figure 18 Unsaturated zone processed in the MetaSWAP. Where: $\boldsymbol{P_n}$ = net precipitation, $\boldsymbol{P_s}$ = irrigation, $\boldsymbol{E}$ = evapotranspiration, $\boldsymbol{V}$ = soil moisture at equilibrium, and $\boldsymbol{Q_c}$ = rising flux (Vermeulen et al., 2016)

The CAP module used in the model was on the basis of a previous groundwater flow model for the whole Netherlands built by Deltares. The required input data for MetaSWAP can be found in *Table 1*. Because the grid-cells are rather coarse, the BND, LGN, SEV, WTA, and UTA were altered in accordance with the finer geographic data in the model area. And precipitation and evapotranspiration data were contained in the file “mete_svat.inp”. Hence, there is no need to include RCH (Recharge) and EVT (Evapotranspiration) Packages in the model.

Table 1 Input data for MetaSWAP (Van Walsum, 2010; Vermeulen et al., 2016)

No.	Unit	Input data	Description
1	-	BND	Boundary settings
2	-	LGN	Landuse code, referring to the file “luse_svat.inp”
3	cm	RTZ	Rootzone thickness
4	-	SUF	Soil Physical Unit, referring to the file “fact_svat.inp”
5	-	MET	Meteo Station number, referring to the file “mete_svat.inp”
6	m NAP	SEV	Surface Elevation
7	-	ART	Artificial Recharge Type. 0 = no occurrence, 1 = present at current location from groundwater, 2 = present at current location from surface water

			extraction
8	-	ARL	Artificial Recharge Location, number of model layer from which water is extracted
9	mm/d	ARC	Artificial Recharge Capacity, depends on the duration of irrigation as specified in the file “luse_svat.inp”
10	m^2	WTA	Wetted area
11	m^2	UBA	Urban area
12	m	PD	Ponding Depth
13	m NAP	PWT	Depth of the Perched Water Table level below the surface
14	-	SFC	Soil Moisture Factor to adjust the soil moisture coefficient
15	-	CFC	Conductivity Factor to adjust the vertical conductivity
16	Files related to MetaSWAP	Fact_svat.inp	Values of vegetation factors and interception characteristics
17		Luse_svat.inp	Set of land use options and their characteristics
18		Thsat_svat.inp	Saturated water contents and saturated conductivities
19		Mete_grid.inp	Mete-information about the location and time parameters of the meteo-grids
20		Para_sim.inp	General input file
21		Sptu_svat.inp	Soil physical parameters for numerical calculation
22		Tiop_sim.inp	Specification of time-related output options
23		Beta2_svat.inp	Boesten parameter for soil evaporation
24		Init_svat.inp	Initial conditions of soil water
25		Sel_key_svat_per.inp	Specification of key variables for output to per bda files
26		Sel_key_svat_dtgw.inp	Specification of key variables for output to dtgw bda files
27		Metaswap.sim	Linking of SVAT units to MODFLOW cells

KHV Horizontal permeability Module

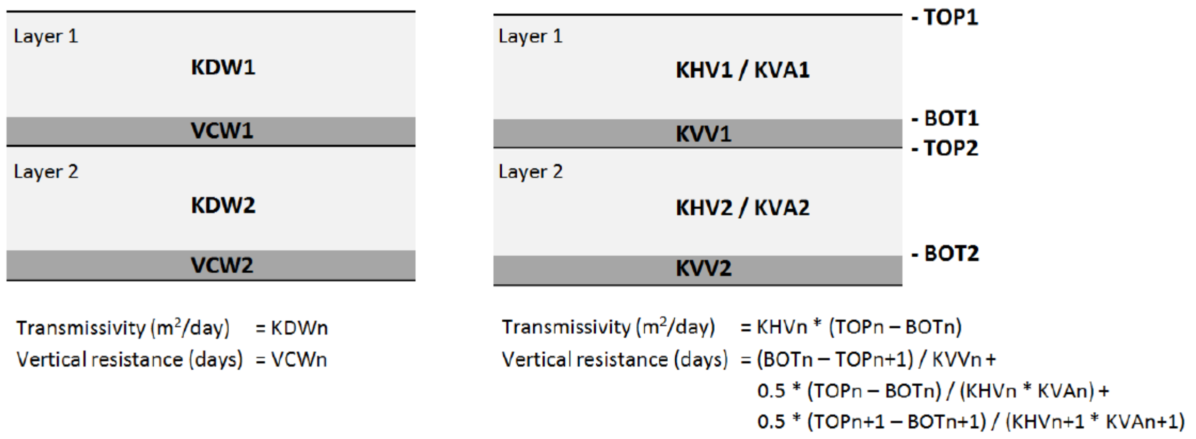


Figure 19 Hydraulic layer parameters used in iMODFLOW (Vermeulen et al., 2016)

The horizontal permeability KHV $[L/T]$ is a measure of a material’s capacity to transmit water. It is described as the rate of flow of water under a unit volume per unit time through a unit cross-sectional area of an aquifer (*Ferris et al., 1962*). It is used to calculate the transmissivity $[L^2/T]$ of each model layer in combination with the layer thickness (see *Figure 19*) (*Vermeulen et al., 2016*). The KHV value for the first layer was interpolated for the whole model area based on the results of the slug test (see *Chapter 6.2.1*). The KHV values for the second and third layers were retrieved from the groundwater flow model built for the company Croda by Royal HaskoningDHV (*Boleij in Dutch, 2013*). The model was constructed for the area Korte Akkeren, Gouda, which is just located on the left side of the inner city. The KHV values would be 0.00187 m/day for the second layer and 42.075 m/day for the third layer.

KVV Vertical permeability Module

The vertical permeability KVV $[L/T]$ is used to calculate the vertical resistance between two model layers in combination with the thickness of resistance layer (see *Figure 19*). It can be obtained by vertical anisotropy (KVA), which is the ratio of KHV and KVV (*Vermeulen et al., 2016*). Normally, the value of KVA ranges from 0.01 for clay to 0.5 for alluvium (*Todd & Mays, 2005*). In this case, in order to simplify the model, the vertical anisotropy was set to be 0.2, which indicates that KVV is five times smaller than KHV.

STO Storage coefficient Module

The storage coefficient STO $[-]$ is defined as the volume of water releases from or takes into storage per unit surface area of the aquifer per unit change in the hydraulic head (*Ferris et al., 1962*). The STO for each model layer is dependent on the lithology (*Vermeulen et al., 2016*). The STO value for the first aquifer can be calculated by *Equation 7*. And for both of the second and third aquifers, the STO was chosen to be 1×10^{-6}.

$$STO = S_s b$$

Equation 7

Where:

S_s = specific storage $[L^{-1}]$, which would be 0.27 in this case;

b = thickness of the first layer $[L]$.

RIV River Package

The RIV (River) Package was applied to simulate the effects of flow between the surface water and groundwater system, as well as between sewer and groundwater system. The reason for the sewer system to be imported into the model as RIV Package instead of DRN (Drainage) Package is that the sewage water level is not below groundwater level permanently. When the sewage water level is higher than groundwater level, and the sewer pipes are leaky, the sewage water will be the recharge source for groundwater, which means the sewer system is functioned as a river rather than drainage system. The flow between RIV and groundwater system for reach n is given by (*Harbaugh, 2005*):

$$QRIV_n = CRIV_n\left(HRIV_n - h_{i,j,k}\right)$$

Equation 8

Where:

$QRIV_n$ = flow between the river and the aquifer, taken as positive if it is directed into the aquifer $[L^3/T]$;

$CRIV_n$ = hydraulic conductance of the river-aquifer interconnection $[L^2/T]$;

$HRIV_n$ = water level in the river $[L]$;

$h_{i,j,k}$ = head at the node in the cell underlying the river reach $[L]$.

In the iMODFLOW, the source of water in the RIV Package is unlimited, which implies the water in the RIV Package will never dry out. The RIV Package requires four input data, which are water level $[L]$, bottom level $[L]$, $CRIV_n$ $[L^2/T]$, and infiltration factor $[-]$, respectively (*Vermeulen et al., 2016*). The water level for the canal areas will keep the same value at -0.72 $m\,NAP$, and that of the sewer area will be altered based on the measured sewage water level in each drainage area. *Equation 9* and *Figure 20* show the method to calculate $CRIV_n$ (*Harbaugh, 2005*). And infiltration factor would be "1" to indicate the infiltration is allowed.

$$CRIV_n = \frac{K_n L_n W_n}{M_n}$$

Equation 9

Where:

K_n = hydraulic conductivity of the riverbed material $[L/T]$, which is assumed to be 0.05 m/day in this case;

L_n = length of conductance block, which is the length of the river as it crosses the node $[L]$;

W_n = river width $[L]$;

M_n = thickness of the riverbed layer $[L]$, which is assumed to be 0.25 m as an initial value based on experience in this case, it may require further calibration.

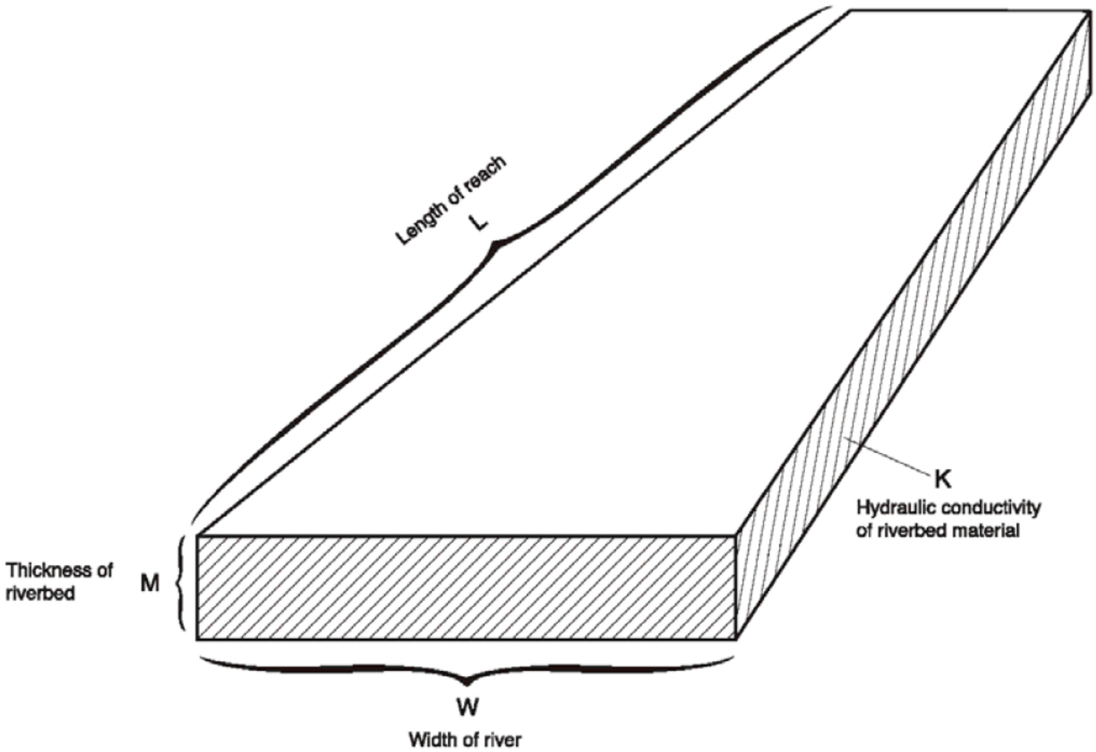


Figure 20 idealization of riverbed conductance in an individual cell (Harbaugh, 2005)

The summary of the active modules and packages in the model can be seen in *Table 2*.

Table 2 Active modules and packages used in the model

Module/Package	Assigned model layer	File number
CAP	1, 2, 3	27
BND	1, 2, 3	3
SHD	1, 2, 3	3
TOP	1, 2, 3	3
BOT	1, 2, 3	3
KHV	1, 2, 3	3
KVV	1, 2, 3	3
STO	1, 2, 3	3
RIV	1	4×107

5.3.3 Sensitivity analysis

Sensitivity analysis is applied to evaluate the impacts of different model designs and parameter values on the simulated groundwater head, in the interest of creating useful nonlinear regressions (*Hill & Tiedeman, 2002*). Parameter sensitivity, as a partial derivative of sensitivity analysis, is to compare the changes of the simulated head when the value of parameter changes, which will be used for the sensitivity analysis in the model (*Faunt et al. 2010*). The results of parameter sensitivity analysis could identify the extent of importance of different parameters to the model outcomes by comparing the observation data to the simulation results.

On the basis of the simulation results of the model without MetaSWAP (see *Chapter 6.4.1.2*), the hydraulic conductance of the sewer system $CRIV_{sew}$ requires being adjusted in order to get a better performance. Nevertheless, the suitable values of $CRIV_{sew}$ cannot be obtained directly through measurements, and they depend on both of the construction year and material of each sewer pipe. Thus, parameter analysis in this research concentrated on the analysis of a single parameter $CRIV_{sew}$. It will be multiplied by 5 and divided by 5, then compare the simulation heads to the observed groundwater levels. The differences between simulation head $[CRIV_{sew} \times 5]$ and head $[CRIV_{sew}/5]$ are more obvious, the groundwater level measured at that location is more sensitive to the parameter $CRIV_{sew}$.

5.4 Data collection – time series data

5.4.1 Precipitation

Two sources of precipitation data were used for analysis and comparison in this report. One source is from KNMI (Royal Netherlands Meteorological Institute, http://www.knmi.nl/home), which is the daily data used as input data in the groundwater flow model. Another source is from HydroNET.nl (http://portal.hydronet.nl/), which is radar-based information with a temporal resolution of one hour for the whole inner city polder. It was used to compare with groundwater level in the time series analysis.

The data from KNMI is measured by a manual rain gauge from the 8-8 observation network (*KNMI Climate Explorer, 2016*). The precipitation gauging station is located just outside of the inner city polder (see *Figure 2*), so it can represent for the daily precipitation in the model area. The daily data is over the period from 0800 UTC on a preceding day until 0800 UTC to the present day, which suggests the data, for example, on 16-04-2016, was actually the accumulation of precipitation from 15-04-2016 8:00 until 16-04-2016 8:00. In the model, the data on 16-04-2016 was used as the precipitation on 15-04-2016. Therefore, the "time from beginning of the year at 00:00:00" for 01-01-2016 in TIOP_SIM.INP and METE_GRID.INP (input files for SIMGRO in METASWAP) was set up as “0”. Deduced by analogy, the “time from beginning of the year at 00:00:00” for the starting time (16-04-2016) would be “106” instead of “107”. The data is validated every ten days, taking up to three weeks, and the validated data for each month can be downloaded from the website at the end of that month (*KNMI in Dutch, 2016*).

The data from HydroNET.nl (http://portal.hydronet.nl/default.aspx?page=6&appid=16&lang=1) is radar-based information with a spatial resolution of 1 × 1 km and a temporal resolution of 5 min. It is on the basis of KNMI radar data composite from De Bilt and Den Helder and will be corrected with the aid of SCOUT software, data from 33 KNMI automatic gauge stations, and manual measurement data from over 300 daily stations.

From *Figure 21*, the variation trends of KNMI and HydroNET.nl are nearly the same. But data from HydroNET.nl is always slightly higher (≤ 5 mm) than that from KNMI. Although HydroNET.nl data has been validated, but radar data still has its own shortcomings which are not suitable for hydrological analysis. Due to the fact that the point precipitation measurements from KNMI are accurate enough for academic purpose, and multiple types of research for HydroNET.nl used them as calibrations, it is reasonable to choose KNMI data as model input (*Einfalt et al., 2013; Lobbrecht et al., 2011; Reichard et al., 2014*). Furthermore, during the model simulation period, the maximal daily precipitation is 33.9 mm, which occurred on 23rd of June, 2016.

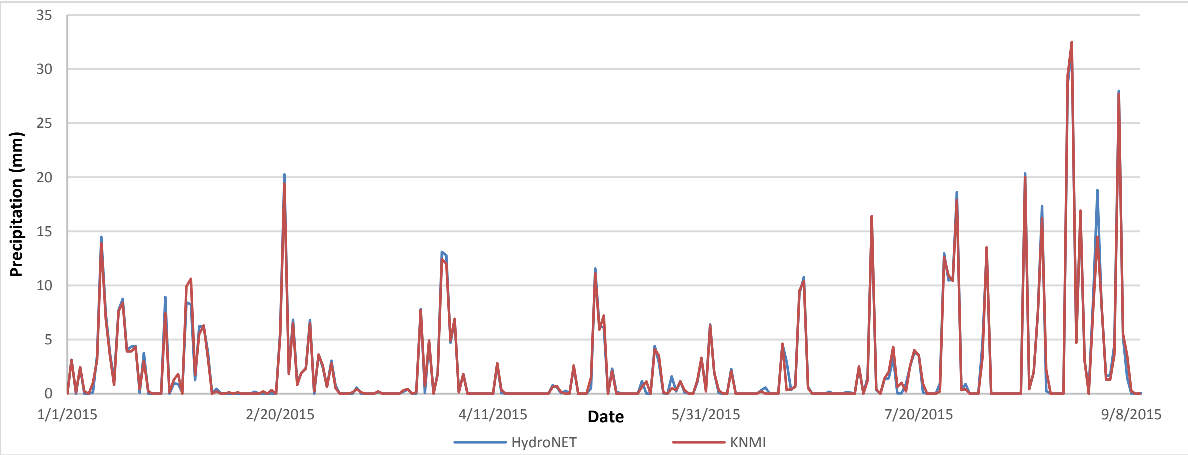


Figure 21 Comparison of daily precipitation between KNMI and HydroNET in 2015

5.4.2 Evapotranspiration

The potential evapotranspiration data was collected from KNMI as well (http://www.knmi.nl/nederland-nu/klimatologie/daggegevens). Since there is no measurement of potential evapotranspiration at the meteorological station in Gouda, in accordance with the principle of proximity, the data from Cabauw Station is selected as the substitution. The data is on the daily basis, and it is used as input for the groundwater flow model. As for the hourly evaporation data, it was calculated based on the proportion of the global radiation of one hour Q_h $[J/L^2]$ in the global radiation of one day Q_{24} $[J/L^2]$ (see *Equation 10*) and used for time series analysis.

$$EV_h = EV_{24} \times \frac{Q_h}{Q_{24}}$$

Equation 10

Where:

EV_h = potential evapotranspiration on hourly division $[L]$;

EV_{24} = potential evapotranspiration of a day $[L]$.

The daily potential evaporation varies from 0.9 mm (on 25th of April, 25th of May, 30th of May, and 20th of June, 2016) to 5.2 mm (on 5th of June, 6th of June, and 19th of July, 2016).

5.4.3 Groundwater level

The groundwater level data can be gathered via the WarecoWaterData.nl Portal (https://wareco-water2.munisense.net/). There are 22 observation wells distributed inside of the model boundary (see *Figure 23*). 7 of them have already existed and started the automatic measurements since the beginning of 2015. Another 15 new wells were installed in May based on the new groundwater observation network (see *Chapter 6.1*) and started to measure the groundwater level by automatic loggers simultaneously. All the automatic data is on the hourly basis.

The WarecoWaterData.nl could provide reliable groundwater data with high frequency. When the data is collected, manual procedures are reduced in order to avoid unnecessary artificial errors. All kinds of data (e.g., metadata, pressure, hydraulic head, etc.) is measured, validated, and calculated automatically. Field surveys are conducted by experts periodically for the sake of maintenance, manual control for automatic measurements, and adjustments with respect to NAP (Amsterdam Ordnance Datum). What is more, for the automatically measured data, the following rules are applied for validation: 1) unrealistically high: measured data is 0.15 m higher than the top of well; 2) unrealistically low: measured data is equal to the logger depths; 3) unrealistic fluctuation: the difference of measured data within an hour is more than 0.5 m; and 4) deviation from manual control measurement: distinction between automatic data and manual control data at the same time is more than 0.05 m (*Beets & Schuurman in Dutch, 2015*).

Since iMODFLOW has not been developed to support hourly model simulation yet, the average groundwater level of a day was used for comparison with the model results at each observation well. Nevertheless, the collected hourly data was still useful for time-series and cross-section analysis.

5.4.4 Sewage water level

The sewage water level data was obtained from the company Royal HaskoningDHV. There are six measurement points in the inner city, at two pumping stations and at four CSOs (see *Figure* **2**), separately. The data measured at the pumping stations is the sewage water level in the pumping system (“Bemalen” in Dutch) and storm water system (RWA, “Regenwaterriool” in Dutch), which have been regarded as watertight pipes. And the data measured at the CSOs is the sewage water level in the back-stowed system (“Opgeboeid” in Dutch), which has been considered to be leaky. Therefore, the data measured at the CSOs was used for data analysis and model simulation.

The whole inner city polder is divided into five drainage areas, Nieuwe Haven, Centrum, Markt, Tuinstraat, and Raam, respectively (see *Figure 22*). The area Nieuwe Haven and Tuinstraat go to different pumping stations (PS 103 and PS 104, see *Figure 2*), and Centrum and Markt discharge the sewage separately to them by gravity. Since Raam belongs to the drainage area Korte Akkeren, which is situated outside of the inner city, it does not connect to the rest of drainage areas (*Zandee in Dutch, 2009*). The model boundary contains the whole drainage areas like Nieuwe Haven, Centrum, and Markt, as well as part of the drainage area Tuinstraat. For each drainage area, there is one measurement point at the CSO to collect the sewage water level data (see *Figure 2*), and it is going to represent the sewage water level in each drainage area. Thus, the data measured at CSO 703 is for Nieuwe Haven, CSO 704 is for Centrum, CSO 724 is for Markt, and CSO 723 is for Tuinstraat.

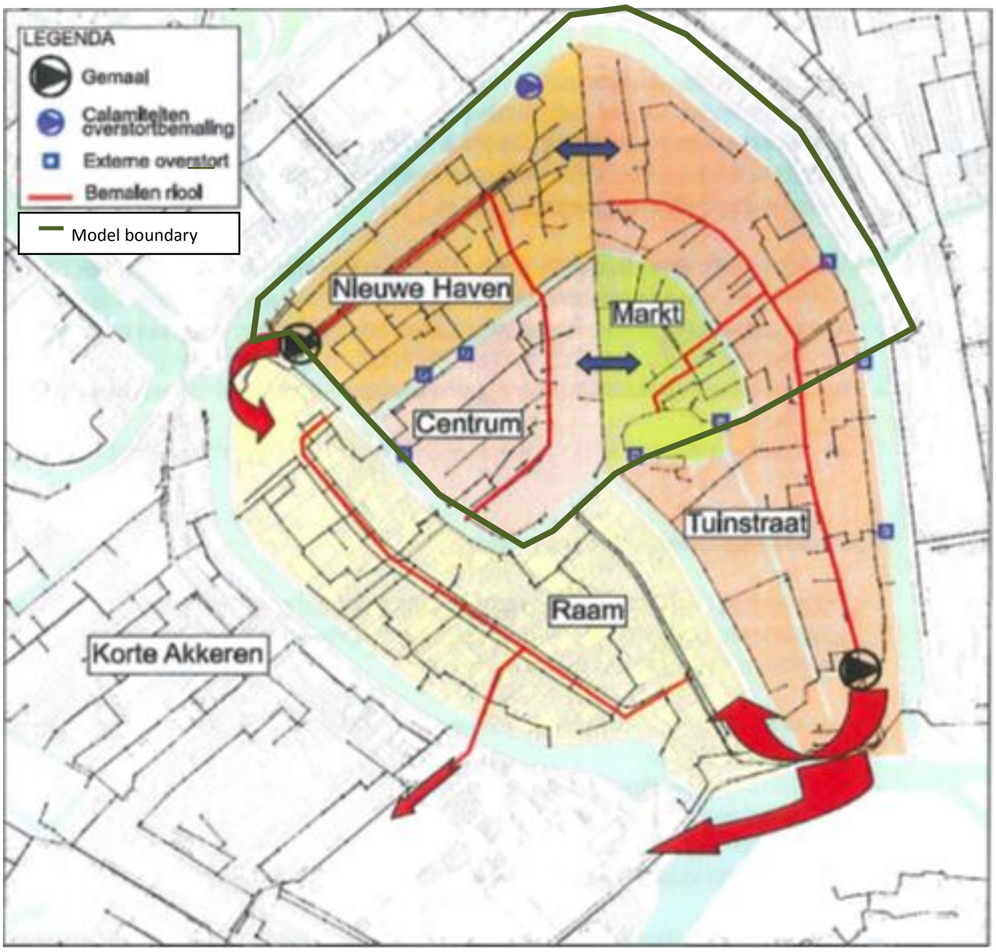


Figure 22 Drainage Areas in the Inner City of Gouda (Zandee in Dutch, 2009)

The data was measured every three minutes, which is not necessary for data analysis and model simulation. The hourly average values were calculated for data analysis, as the differences of sewage water level within an hour are small enough to neglect. And daily average values were used to import into the groundwater flow model. As far as known, the data is not validated regularly. However, the average values might minimize the influence from abnormal data along the time series. Anyway, it would still be better to have accurate data, further validation will be required.

6 Results

6.1 Local groundwater observation network

With the close cooperation and coordination among Deltares, Hoogheemraadschap van Rijnland, Gemeente Gouda, Wareco Ingenieurs, and local inhabitants, all the 15 new observation wells were finished installing on the 20th of May. The new wells were designed to focus on the groundwater fluctuation along the Street Nieuwe Haven and Turfmarkt (see *Figure 23*). In order to get sufficient access to the wells for data verification and maintenance, 13 wells were designed to be located in the public areas except for the Well 05 and 08, which are in the private gardens and backyards. On the whole, the new groundwater observation network takes into consideration of factors which have influence on or are influenced by groundwater level fluctuation as many as possible, such as land use (the wells are distributed in all kinds of land covers), plant types (Well 05, 08, 09, 10, and 13), sewer system (Well 01, 02, 03, 04, 1-1.04, 1-1.05, 1-1.08, and 1-1.134), surface water (Well 06, 07, 11, and 12), wooden foundations (Well 14 and 15), as well as crawl spaces and backyard (Well 05 and 08).

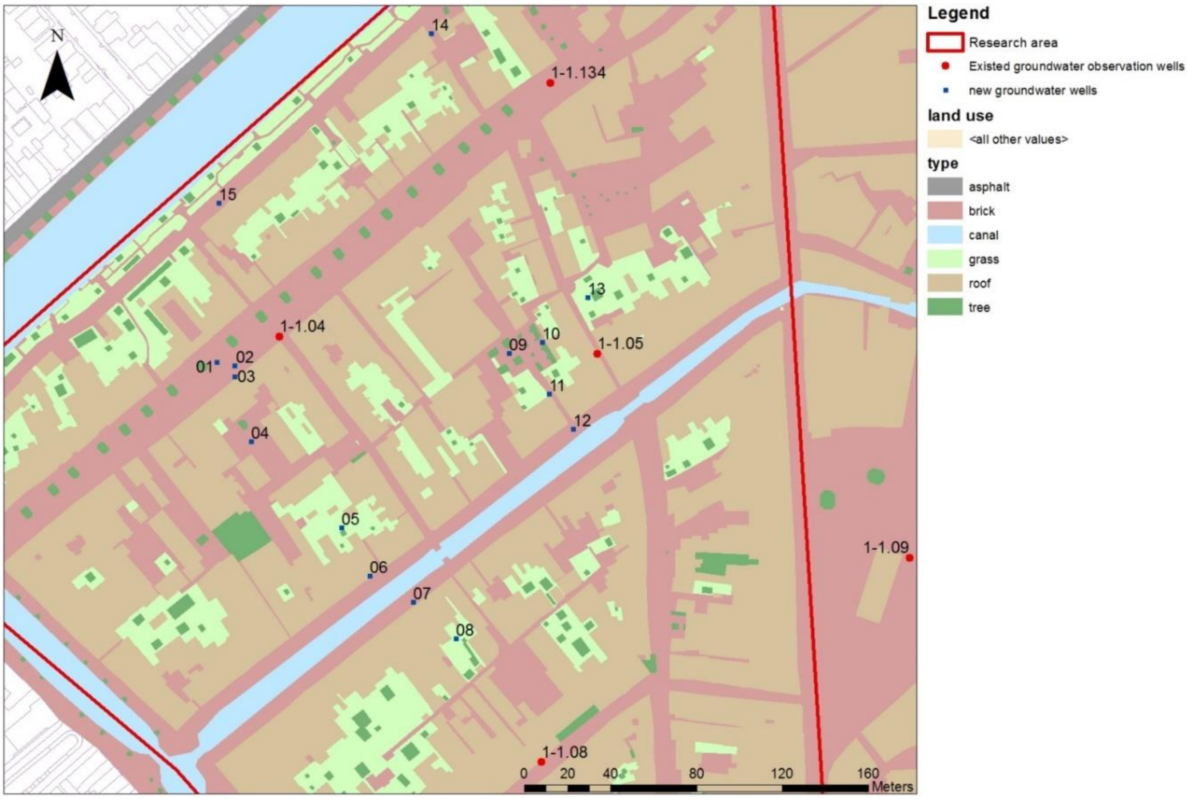


Figure 23 Locations of all the wells in the new groundwater observation network

The Well 01, 02, 03, and 04 are designed to analyze how fast and how far the fluctuation of sewage water level would have an impact on the groundwater level, which makes the Well 04 as a reference well. The complexity of underground infrastructure beneath the Street Nieuwe Haven makes it an ideal location for the investigation on groundwater flow behavior. The street is used to be a harbor, but the old watercourse was drained and filled with sand. Three types of sewer system are existed and parallel to each other along the street (see *Figure 24*). It has been assumed that all the back-stowed sewer pipes (orange lines in *Figure 24*) are leaky from the previous investigation results. In addition, the old harbor quays are still positioned underground, and the top of the quays is at the elevation of -1.5 $m\ NAP$ (*Den Nijs, 2015*). Between the Well 01 and 02, there are one part of the quay and a back-stowed pipe made by GVK constructed in 1995. And there is a back-stowed pipe made by concrete built in 1940 between the Well 02 and 03 (see *Figure 25*). The distances among these three wells in the vertical direction to the sewer pipes are no more than 10 m. As the reference well 04, it is seated nearly in the middle of two parallel back-stowed lines, in order to counteract the effects from each other (see *Figure 24*).

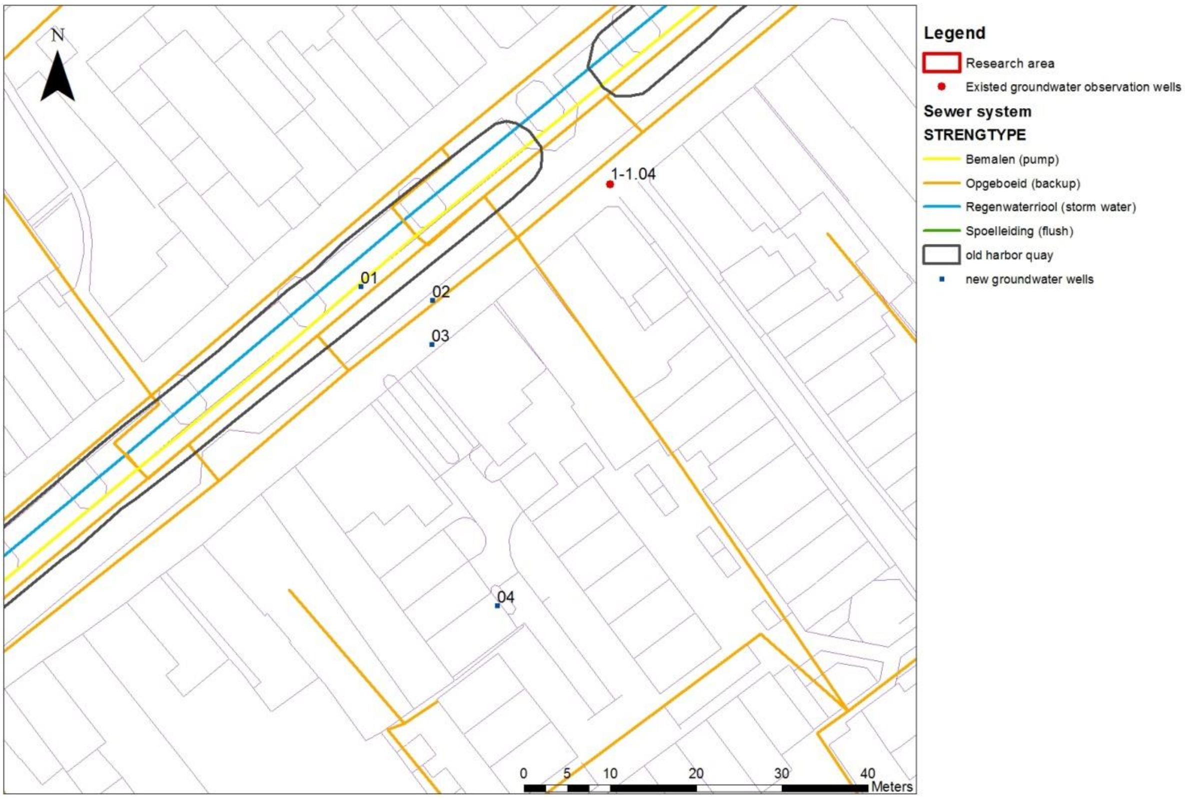


Figure 24 Underground infrastructures along the Street Nieuwe Haven

The Well 06, 07, and 12 are situated along the canal Turfmarkt to be used for explaining the influential from the surface water level, and the Well 11 is used as a reference well. The Well 05 and 08 are located in the private gardens, the primary vegetation in those backyards are grass and flowers. In addition, the Well 09, 10, 13 are installed in the vicinity of trees to indicate the influence of evapotranspiration, particularly in the summer time. Because the types of the trees near the Well 09, 10, and 13 are different (see *Figure 26*), together with the Well 05 and 08, the comparison could illustrate the distinct kinds of vegetation might have various impacts on groundwater levels in the urban areas. What is more, the Well 14 and 15 are located next to the premises with wooden foundations, which have been investigated and verified before (see *Figure 27*).

All the new wells for the observation network had been installed and started the automatic measurement since 20th of May, 2016. The borehole information and observation data can be retrieved from WarecoWaterData.nl Portal. Based on the observations along the simulation period (from 20th of May to 31st of July, 2016), the average groundwater levels in those observation wells are approximately from -0.8 $m\ NAP$ (at the Well 04, 05, 06, 07, 12, and 1-1.134) to -1 $m\ NAP$ (at the Well 01, 03, 1-1.04, and 1-1.05) (besides the Well 09). The minimal ad maximal measured groundwater levels vary from around -0.4 $m\ NAP$ (at the Well 10 and 13) to -1.5 $m\ NAP$ (at the Well 1-1.05) (except for the Well 09). Additionally, the fluctuation between minimal and maximal groundwater levels at each well range between

0.2 m (at the Well 06, 07, and 12) and 0.8 m (at the Well 13 and 1-1.05). More information about observation data of groundwater level can be found in *Appendix 3 Hourly time series data of groundwater levels comparing to precipitation and sewage water levels*.

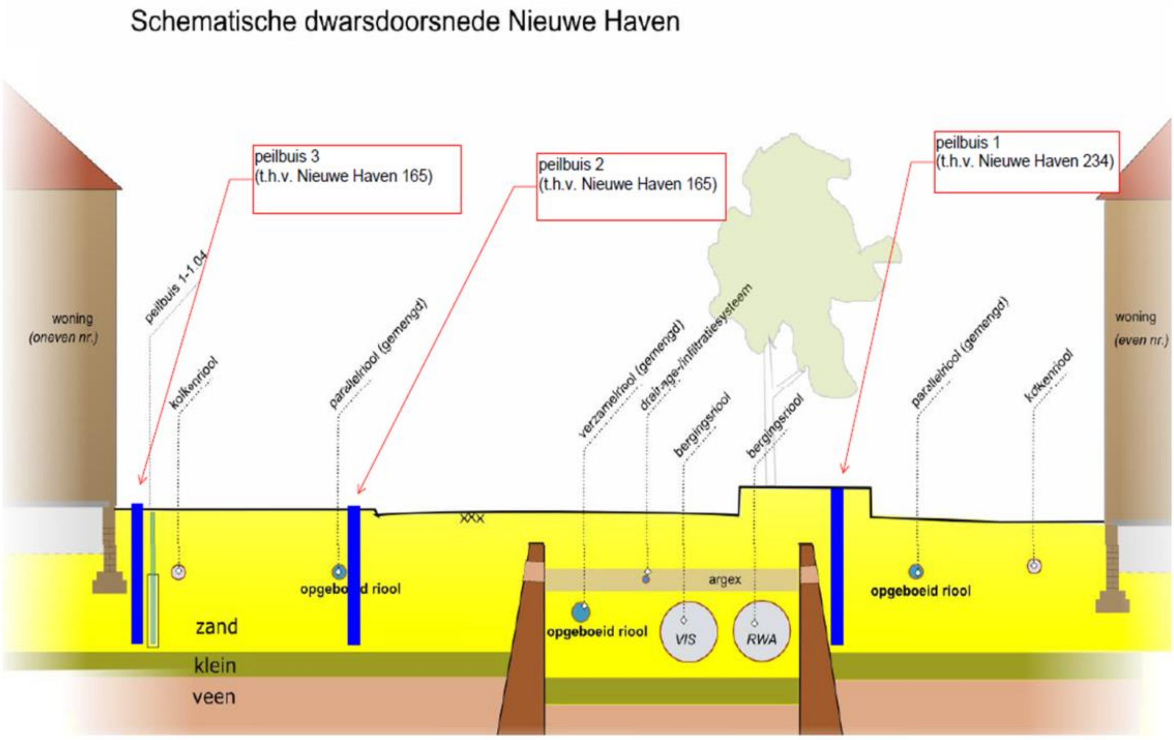


Figure 25 Position relationship of the Well 01, 02, and 03 in the visual angle of cross-section

Figure 26 Vegetation types near Well 09, 10, and 13

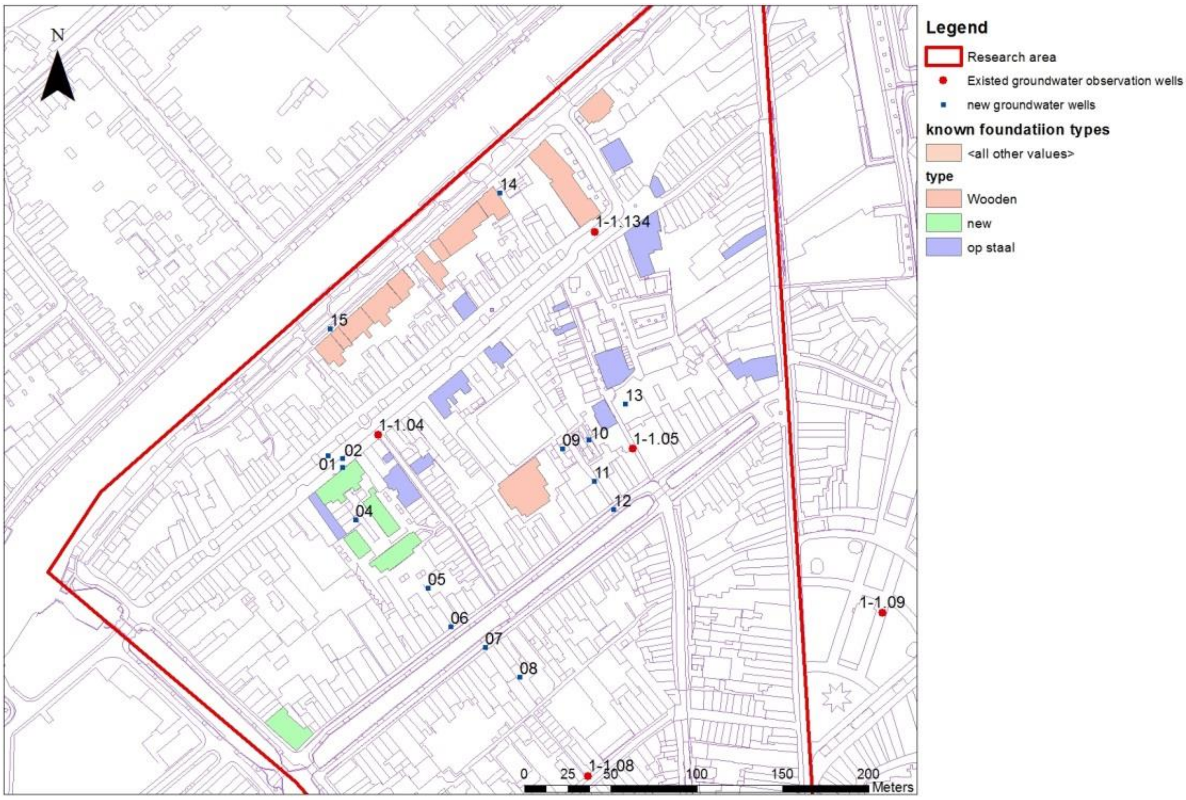


Figure 27 Buildings in the Area Nieuwe Haven with already known foundation types from previous investigations (Den Nijs, 2015)

6.2 Field experiment

6.2.1 Slug test

11 observation wells were selected to implement the slug test in the model area, namely, 02, 03, 05, 06, 07, 08, 12, 14, 15, 1-1.04, and 1-1.10 (see *Figure 23*). The reason for choosing these wells is that those well screens partially or fully penetrate the sandy layer. At the same time, the top of the screens is beneath the bottom of the sandy layer, or merely thin layers of sand are existed at the bottom part of the screens (see *Appendix 1 Borehole information*).

For each slug test, both manual and automatic measurements were applied. For the manual operation, the initial water depth from the top of the well would be measure, then additional water was poured into well as soon as possible, and the maximum head would be measured and continue the measurements after 5 seconds, 10 seconds, 20 seconds, 30 seconds, 1 minutes, 1.5 minutes and so on until the water level recovered to the initial head. The procedure of automatic measurement is similar. Rather than a manual measurement device, a data logger would be used to measure the pressure [$cm\ H_2O$] and temperature [℃] every 15 seconds. For wells where the hydraulic conductivities are comparatively high and took less time for operation, the manual and automatic measurements would do twice or three times. The calculation results of manual and automatic measurements would be intercompared, and an average value would be applied if the differences were not greatly big. Once the differences are not acceptable, the K_h values would be based on the results of the automatic measurements. The final results of slug test are showed in *Table 3*.

From the results of slug test, several calculated K_h values will not be included into the interpolation for the first layer KHV module. They are results of the Well 02, 06, 15, and 1-1.10 since they are considerably smaller than others. For example, the Well 02 is around 5 meters away from the Well 03, but the results have a great deal of differences from each other (5.49 m/d at the Well 03 and 0.166 m/d at the Well 02). Besides, both the Well 06 and 07 are situated along the canal Turfmarkt, the results of them also present a great difference. In addition, it should be noted that the comparison between the results of the Well 12 and 15. From the borehole information, the screen parts of both of the wells are in the clay layer. However, the value at Well 12 (7.92 m/d) is much higher than that at Well 15 (0.568 m/d). The possible explanation is that the length of the sandy layer on the top in the Well 12 is longer than that in the Well 15 (see *Appendix 1 Borehole information*), and the top of the screen in the Well 12 is located exactly the boundary between sandy and clayey layers. When the water was added, it is possible the water would flow upward to the sandy layer, thus, it might lead to the different flow rates in the wells. At the same time, the measured K_h value at the Well 15 seems obviously higher as well, comparing to the KHV value (0.00187 m/d) for the second layer in the model (see *Chapter 5.3.2.5*). It is possible that the data from the model built for the company Croda (*Boleij in Dutch, 2013*) may be not suitable for the research area. Further measurements and investigations are recommended to collect a reasonable hydraulic conductivity for the peat and clay layer. Or this can also be tested by the Parameter Estimation Module in the groundwater flow model.

Table 3 Results of slug test

Well	**Land cover**	**Number of times**	**Horizontal hydraulic conductivity K_h [m/d]**		
			Manual	Automatic	Average
02	Brick	1	-	0.166	0.166
03	Brick	1	5.91	5.32	5.49
		2	5.26	5.66	
05	Flowers and grass	1	3.10	3.48	3.29
06	Brick	1	0.756	0.778	0.767
07	Brick	1	5.19	12.7	11.2
		2	-	9.63	
		3	-	7.42	
08	Grass	1	4.03	1.81	3.85
		2	-	3.85	
12	Brick	1	8.16	7.67	7.92
14	Brick	1	8.30	6.09	8.56
		2	-	8.56	
15	Brick	1	1.31	0.568	0.568
1-1.04	Brick	1	9.90	9.22	8.62
		2	-	8.01	
1-1.10	Brick	1	0.328	0.253	0.291

In general, according to the slug test results, the horizontal conductivity $K_h$for the first sandy layer ranges from 4 to 11 m/d.

6.2.2 Open the weir in the sewer system

The field experiment was implemented twice, from 06-07-2016 7:00 to 08-07-2016 16:00, and from 21-07-2016 7:00 to 23-07-2016 16:00, respectively. For each experiment, the groundwater levels measured at the each well compared to the sewage water levels measured at two CSOs, which are used to represent the sewage water levels in each drainage area. The results from two experiments had little distinct, while during the first experiment, there was some rain during the last few hours before closing the weir. Therefore, in this chapter, the results of the second experiment will be explained. The results for both of the experiments can be found in *Appendix 3 Hourly time series data of groundwater levels comparing to precipitation and sewage water levels*.

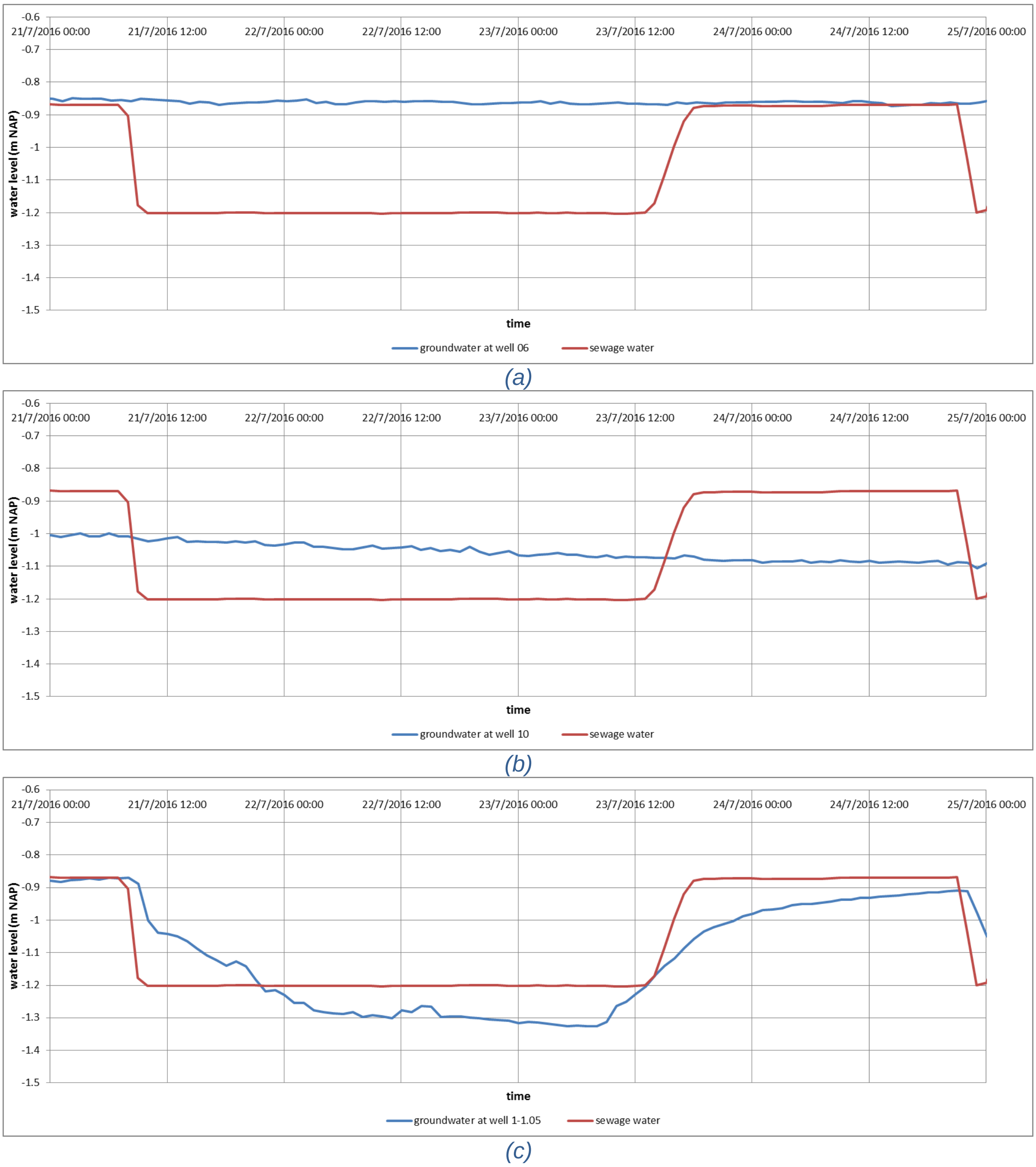


Figure 28 Comparison between groundwater level and sewage water level during the 2nd field experiment

In accordance with the decreasing levels of groundwater when the sewage water levels dropped down to the sewer bottom, the results can be separated into three groups, which are “barely change”, “slightly descend”, and “clear descend”. At five observation wells (Well 05, 06, 11, 12, and 1-1.08), there is basically no change in groundwater levels (< 0.05 m) when sewage water level falls. For example, *Figure 28 (a)* shows the groundwater levels in the Well 06 remained nearly steady when the sewage water went down about 35 cm. It is the same case for Well 05, 11, 12, and 1-1.08 (see *Appendix 3 Hourly time series data of groundwater levels comparing to precipitation and sewage water levels*). The possible explanations for this phenomenon are: 1) the back-stowed pipes near some of these wells (such as Well 05, 06, and 1-1.08) are made of PVC at the end of last century (see *Table 4*), which are highly unlikely to be leaky; and 2) the Well 11 and 12 are not in the vicinity of any potentially leaky sewer system (see *Figure 29*).

Table 4 Construction year and material of the back-stowed sewer system near to each groundwater observation well

Groundwater observation well	Back-stowed sewer system nearby	
	Construction year	Material
01	1995	GVK
02	1940	Beton (concrete)
03	1940	Beton (concrete)
04	-	-
05	1986	PVC
06	1991	PVC
07	1991	PVC
08	1870	Metselwerk (brick)
09	-	-
10	1869	Beton (concrete)
11	-	-
12	-	-
13	1869	Beton (concrete)
14	2004	PVC
15	2004	PVC
1-1.04	1940	Beton (concrete)
1-1.05	1869	Beton (concrete)
1-1.08	1970	PVC
1-1.134	1940	Beton (concrete)

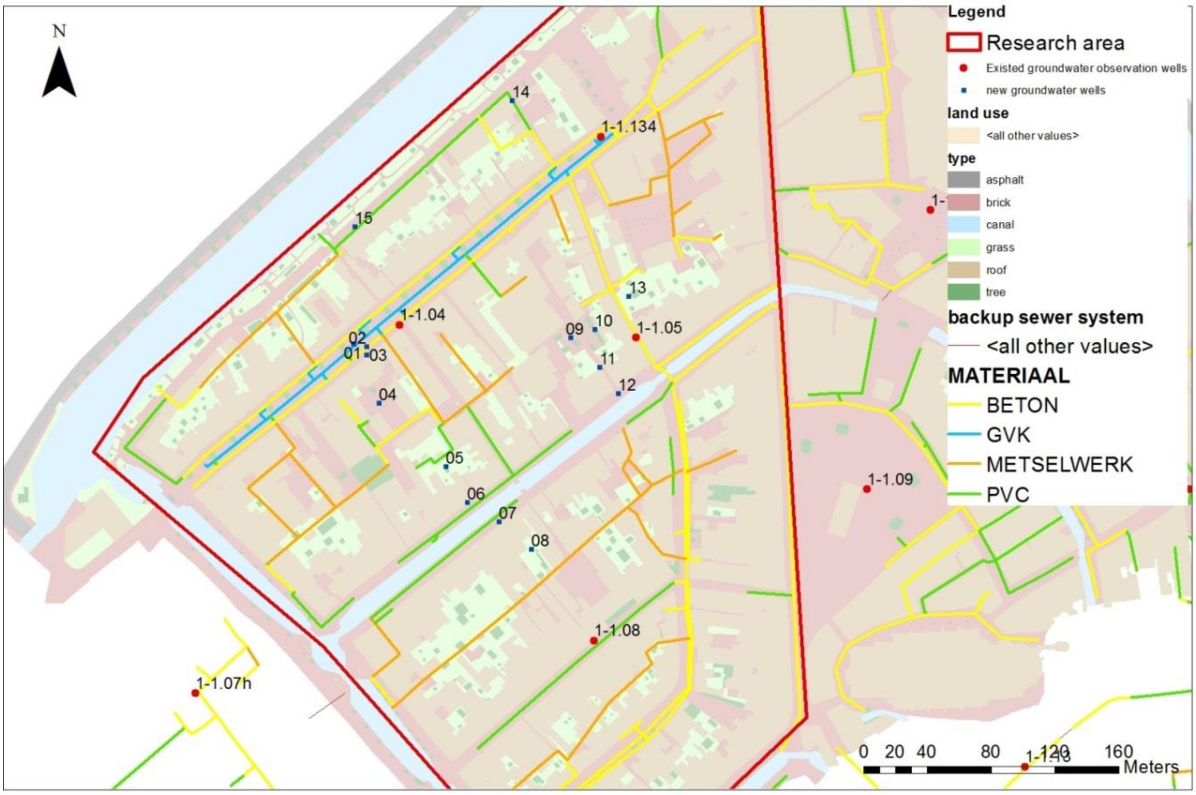
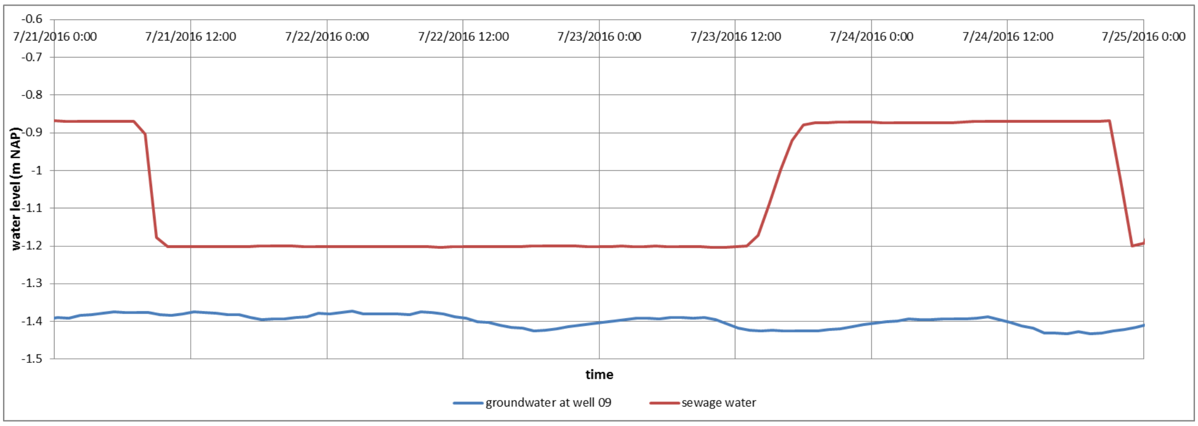
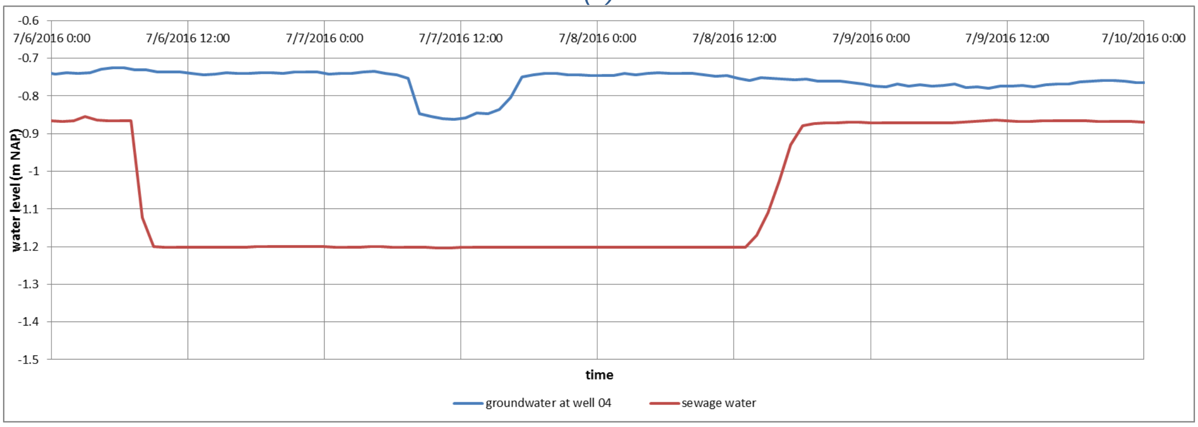


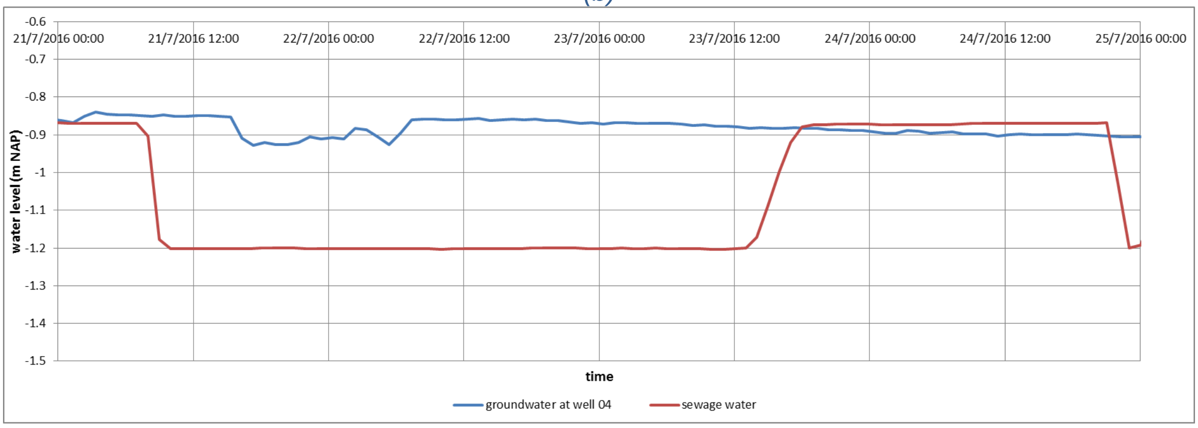
Figure 29 Material of the back-stowed sewer system near to each groundwater observation well



(a)



(b)



(c)

Figure 30 Abnormal phenomena happened at the Well 04 and 09 during the field experiments

Nevertheless, under the most circumstances, the groundwater would descend along with the sewage water. It is only a matter of extent. For instance, *Figure 28 (b)* and *(c)* indicate the different extents of groundwater declining when the sewage water level dropped to the same level. Both of them have the similar variation trending, which is when the weir was open, the groundwater decreased gradually until the weir was closed, then it bounced back to the original water level by degrees. Groundwater at the Well 01, 02, 03, 07, 10, 14, and 15 fell between 0.05 and 0.1 m during the experiment, while groundwater in the Well 08, 13, 1-1.04, 1-1.05, and 1-1.134 descended more than 0.1 m. The similarity among these wells is that the back-stowed pipes near them were made of concrete ("beton" in Dutch) or bricks ("metselwerk" in Dutch) (see *Table 4*), which have higher possibilities to leak. The potential reasons led to different extents of groundwater level dropping down might be that the different distances of the observation wells to the leaky sewer system (see *Figure 29*), or the distinct extent of damage in the sewer system. For example, both the Well 10 and 1-1.05 are located near the same sewer system built of concrete in 1869, the groundwater level in the Well 1-1.05 dropped deeper since the well is merely 1 m away from the sewer pipe, yet Well 10 is around 20 m away. It should be noted that in *Figure 28 (c)*, the groundwater measured at the Well 1-1.05 went beneath the sewage water level after opening the weir. It is reasonable since the bottom of the sewer system at the weir is deeper than that at the CSO, where the sewage water level data is collected (see *Figure 13*), as it has been explained in *Chapter 5.2.2*.

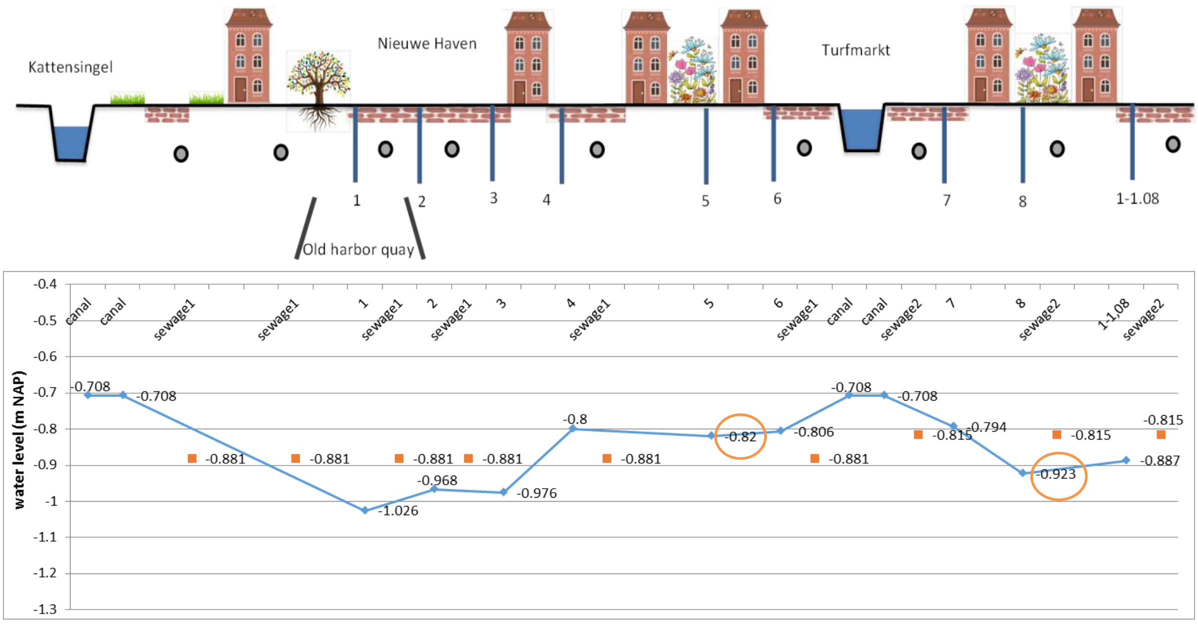
Through the comparison of groundwater levels and sewage water levels, some abnormal groundwater levels measured in the observation wells have been identified. For example, the groundwater at the Well 09 is considerably lower than that at the wells in the neighborhood (see *Figure 30 (a)*). Several manual verifications and device inspections prove that there is no error in the measurement. So far, there has not been found a reasonable explanation. Furthermore, during the two-time field experiments, the groundwater in the Well 04 fell and recovered back within less than a day (see *Figure 30 (b)* and *(c)*) when the weir was opened for two days. It is even more unusual because they only happened during the period the field experiments were implemented. In other normal occasions when the weir needed to be open, it did not display any abnormal behavior (see *Appendix 3 Hourly time series data of groundwater levels comparing to precipitation and sewage water level*). And there is no interpretation about it, either.

In summary, not all of the back-stowed sewer system are as leaky as assumed before. Both the year of the construction and the pipe material would affect the performance of the sewer system. Generally, the pipes made of concrete and bricks seem to be leakier than those made by plastic materials (such as GVK and PVC). At the same time, the older the pipes are, the leakier and more broken they can be. As for how fast and how far the leaky sewer system could influence the groundwater, it depends on the damage level of the sewer pipes.

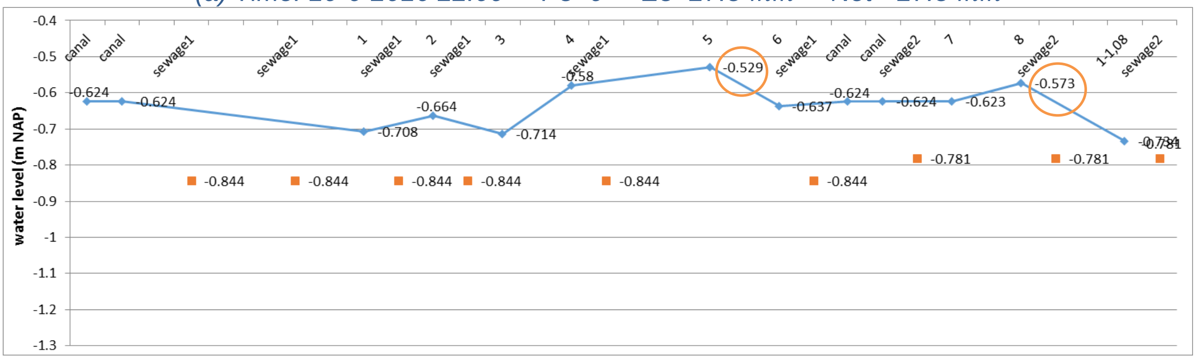
6.2.3 Cross-section analysis

From *Figure 23*, a cross-section line can be drawn from the canal Kattensingel until the Well 1-1.08 via the Well 01 to 08. This cross-section starts from the canal Kattensingle, via the street Nieuwe Haven, one private garden, and the canal Turfmarkt, and ends until the backyard and the street Lange Groenendaal on the other side of the canal Turfmarkt (see *Figure 31*). The complexity of the underground infrastructures along the Street Nieuwe Haven, high surface water levels along the Canal Turfmarkt, and the undiscovered situation in the private gardens make this cross-section line worthy to be analyzed. In this chapter, the most representative moments of the groundwater levels through the cross-section were selected to display the results, including the moment after the longest dry period during the research period (see *Figure 31 (a)*), the moment after the maximal daily rainfall event (see *Figure 31 (b)*), and the moment before and during the second field experiment (see *Figure 31 (c)*). The complete analysis along the cross section can be found in *Appendix 2 Cross section analysis*.

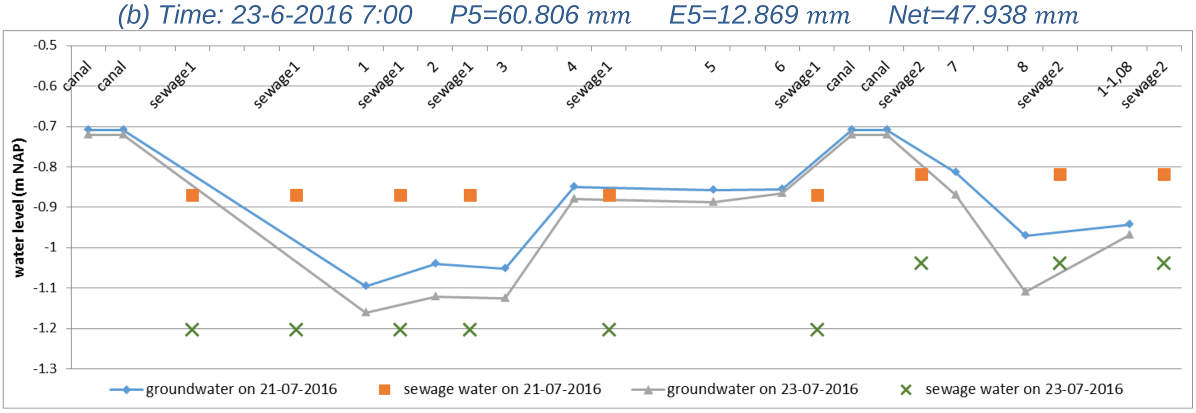
Figure 31 presents a simple sketch of the cross-section. The figure consists of land cover, the relative positions of groundwater observation wells, parallel back-stowed sewer system, and old harbor quay to each other in the visual angle of the cross section. The distances in the sketch do not represent the real situation. In *Figure 31 (a)* and *(b)*, the blue lines refer to the groundwater levels at the moment presented in the texts below the graphs, and the red dots refer to the sewage water levels in each drainage area at the corresponding time. In *Figure 31 (c)*, the blue and gray lines mean the groundwater levels at 7:00 on 21st of July and at 12:00 on 23rd of July, 2016, and the orange square dots and green cross dots mean the sewage water level at the respectively corresponding time. In the text below each graph, "P5" is the accumulated precipitation from the previous 5 days before the time of the measurement, “E5” is the accumulated potential evaporation from the previous 5 days, and “Net” is the difference between “P5” and “E5”, which subtracting “E5” from “P5”.



(a) Time: 10-6-2016 22:00   P5=0   E5=27.3 mm   Net=-27.3 mm



(b) Time: 23-6-2016 7:00   P5=60.806 mm   E5=12.869 mm   Net=47.938 mm



(c) Time: 21-7-2016 7:00   P5=0.855 mm   E5=21.713 mm   Net=-20.858 mm
Time: 23-7-2016 12:00   P5=0.855 mm   E5=24.505 mm   Net=-23.651 mm

Figure 31 groundwater level and sewage water level along the cross-section under different circumstances

Figure 31 (a) shows the groundwater levels and sewage water levels through the cross-section at 22:00 on 10th of June, 2016. Before 10th of June, there was a longest dry period (5 days) from 5th to 9th of June 2016. And on 5th and 6th of June, 2016, the maximum daily potential evaporation occurred. The groundwater levels at the Well 05 and 08, which are located in the private gardens, were comparatively lower than others. It indicates the impact of vegetation on the groundwater level due to transpiration in the dry weather. What is more, on 23rd of June, 2016, when the maximal daily precipitation happened, and the surface water and groundwater reached the highest levels, the groundwater levels at the Well 05 and 08 were distinctively higher than others, even about 0.1 m higher than surface water levels. It suggests that in the private gardens, groundwater responses faster to rainfall events than other types of land cover along the cross-section. *Figure 31 (a)* and *(b)* illustrate that the groundwater in the green areas is more sensitive to local weather conditions.

Figure 31 (c) compares the groundwater levels before opening the weir (blue line) and during the field experiment (gray line). It displays that the groundwater declined at different extents along the cross-section, which goes to a step further to indicate that not all of the back-stowed pipes are leaky, and even though some of them are leaky, the leakage circumstances and effects on groundwater level are in various degrees. For example, the range of groundwater dropping at the Well 08 is the most obvious one (0.14 m), because it is about 19 m away from a brick pipe built in 1870. One could notice that the groundwater at Well 08 measured at that moment was lower than the sewage water level measured at the same time as the bottom of the sewer pipe at the CSO 704 is higher than that at the weir, which has been mention in *Chapter 5.2.2*. Although the sewer pipes near the Well 07 and 1-1.08 were made by PVC, the 1870 brick pipe is so broken that the groundwater at those two locations is affected as well (see *Figure 29*). The distances from the Well 07 and 1-1.08 to the brick pipe are approximately 45 m and 50 m, severally. At the Well 01, 02, and 03, the decreasing levels were almost the same, 0.082 m at the Well 01, 0.073 m at the Well 02, and 0.065 m at the Well 03. There is a concrete pipe constructed in 1940 located between the Well 02 and 03. Since Well 02 is slightly closer to it and groundwater decreased a little bit more, the pipe in-between is identified as a leaky one. The Well 01 is basically situated in the middle of two pipes built in the same year (1940) and of the same material (concrete), it can be deducted that the sewer lines made of concrete seem to have a high possibility to be leaky. This also explains the descent of groundwater levels at the Well 04 and 05, between which there is a concrete pipe built in 1986. Moreover, the groundwater levels measured at the Well 06 barely changed (0.01 m) after the sewage water level was lowered, as the sewage water pipe near it is made of PVC and it is relatively far from any potentially leaky sewage pipes (more than 60 m away from leaky sewer line between the Well 04 and 05).

Combining the finding in *Chapter 6.2.2*, the leaking circumstances of the back-stowed sewer system are not to the same extent, and they depend on the material and construction year of the sewer pipes, as well as the damage locations along the sewer lines. In general, the back-stowed sewer system made of concrete and bricks seem to have a higher possibility to be leaky and influence the groundwater levels. The sewer system constructed between 1869 and 1940 has been proven to be leaky, and 1869 pipes are damaged more obviously. In addition, even along the sewer lines which were made of the same material and built in the same year, the location of breach point also affects how the groundwater responds to the changes of sewer water (comparing the results among the Well 01, 02, 03, and 1-1.04). As for the distance of the impact on groundwater from the leaky sewer system, it is up to the leaky extent of each pipe. At least from the observations in the research area, it can reach approximately 30 to 60 m.

In spite of the findings above, the cross-section analysis illustrates the impact of different land cover on the response of groundwater levels to precipitation and evapotranspiration, which is that unpaved areas are more sensitive to the local weather conditions. This will be explained in the next chapter as well.

6.3 Time-series analysis – precipitation and evapotranspiration

The groundwater level measured at each observation well was compared to precipitation, potential evaporation, and sewage water levels for the time-series analysis. The results can be found in *Appendix 3 Hourly time series data of groundwater levels comparing to precipitation and sewage water level*. The similar conclusions of the comparisons between groundwater levels with sewage water levels can be inquired from the previous chapters. In this chapter, the influence of precipitation and evapotranspiration on groundwater levels will be discussed. A dry period of five days (from 05-06-2016 to 09-06-2016) was selected to compare the groundwater levels to the potential evaporation. As mentioned before, the Well 05, 08, 09, 10, and 13 are located in private gardens or near different types of trees. Thereby, data measured at those wells will be used for the time-series analysis related to evapotranspiration (see *Figure 32*). In order to investigate the reaction of groundwater to precipitation, the maximal hourly precipitation (17.1 mm) occurred at 2:00 on 23rd of June, 2016 was selected, and the period for comparison is from 21st to 25th of June, 2016 (see *Figure 33* and *Table 5*). If the groundwater level responds relatively strong to the precipitation, it indicates that the hydraulic conductivity around the well is relatively low. It can be considered as an additional detection on the results of the slug test (see *Chapter 6.2.1*).

From *Figure 32*, it can be noticed that except from the Well 09, the groundwater levels at other wells all display a descending trend for five days during which there is no rainfall. The groundwater in the Well 05, 08, 10 and 13 dropped around 0.04 m, 0.06 m, 0.11 m, and 0.14 m, respectively. As the sewer water levels remained at about -0.88 $m\,NAP$ in these five days, the graphs illustrate the significant impact of the evapotranspiration on the groundwater levels in the green areas, and different types of vegetation have different extents on extracting groundwater. It is reasonable that the groundwater in the private gardens (at the Well 05 and 08) declined less since there are basically flowers, grasses, and even some pavements. At the same time, the groundwater measured at the Well 10 and 13 dropped more since they are located in the vicinity of big trees (7 Valse Acacia cv around the Well 10 and 7 Witte Paardekastanje around the Well 13), which have a better capacity of extracting groundwater. In addition, the groundwater levels at the Well 08 and 10 show an obvious resemblance to the daily variation pattern of the potential evaporation. Yet, the resemblance does not occur simultaneously. It can be observed that there are several hours of delay between the potential evaporation reaching the maximal value and groundwater level decreasing at the maximal rate. This is probably caused by the slow flow rate and reaction of groundwater to the extraction by vegetation for the transpiration. Although the groundwater at the Well 09 displays the similar pattern as well, it cannot explain why the groundwater at the Well 09 is nearly half a meter lower than others, and why the groundwater level drops and bounces back within a day. It is possible that it comes from the deeper aquifer replenish, or there are some underground infrastructures resisting the groundwater flow. Further investigation around the Well 09 is recommended.

From *Table 5*, it can be observed that when the precipitation reached the highest hourly value (17.1 mm), the groundwater at all the observation wells increased correspondingly. In most cases, the groundwater levels raised approximately 0.05 to 0.12 m within one hour after the precipitation (such as the Well 02), as shown in *Figure 33 (a)*. At some wells, the groundwater increase more than 0.4 m/h, for example, the Well 10 (see *Figure 33 (c)*) and 13, where trees are located. And the groundwater at the Well 15 increased only a little bit (0.035 m/h) (see *Figure 33 (d)*). In addition, the groundwater measured near the canal Turfmarkt (the Well 06, 07, and 12) responds rather fast to the precipitation (see *Figure 33 (b)* and *Appendix 3 Hourly time series data of groundwater levels comparing to precipitation and sewage water level*). It is probably caused by the shallow unsaturated zone in the riparian areas of the canal Turfmarkt. Generally, the groundwater in the unpaved and riparian areas is more sensitive to the precipitation.

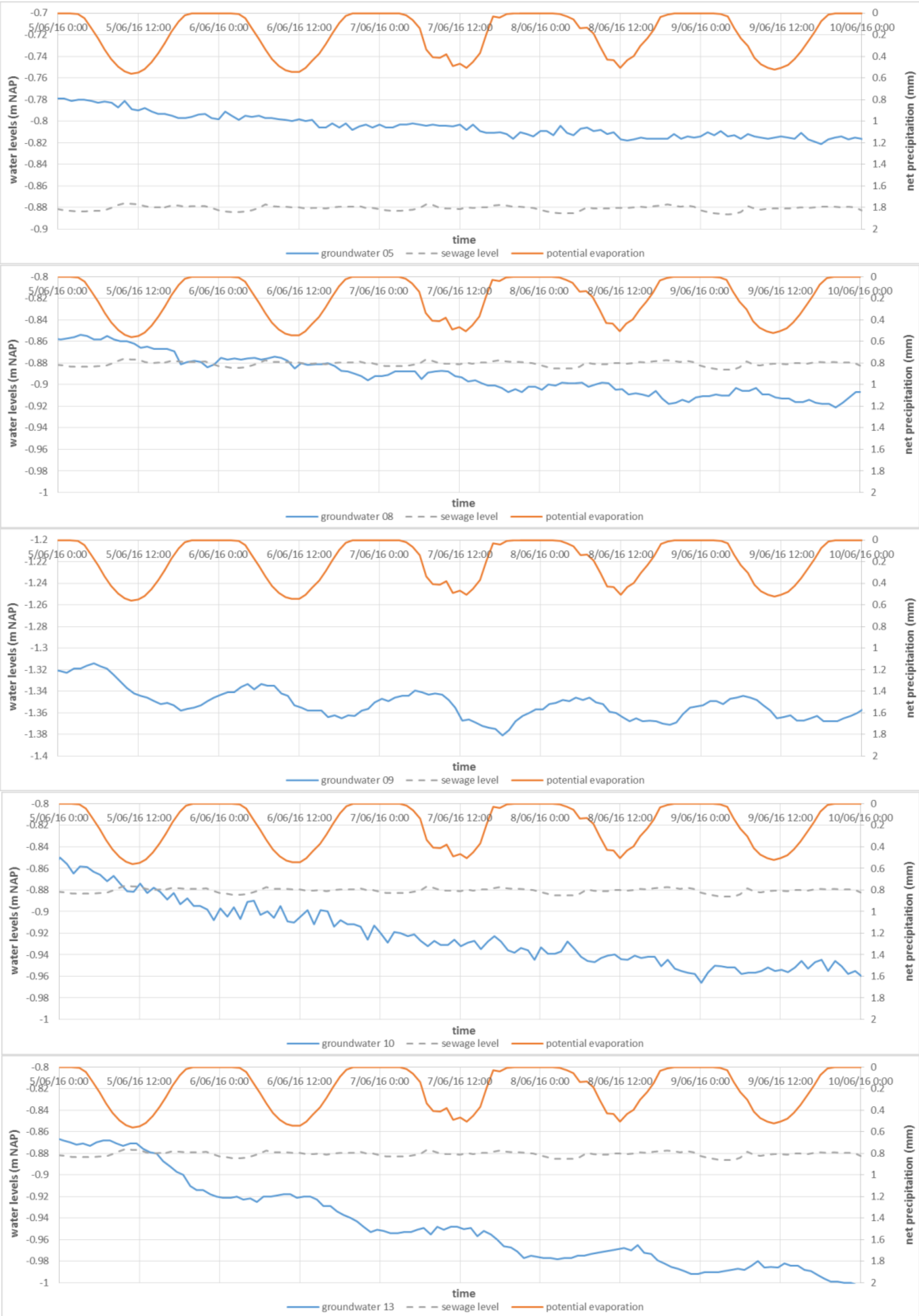
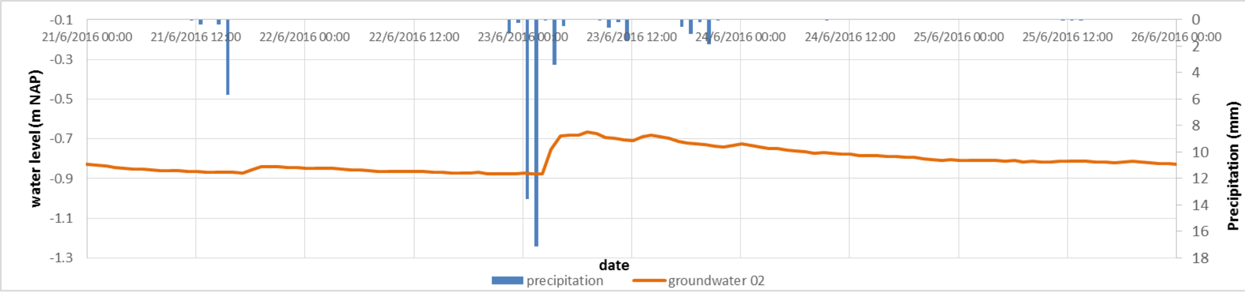
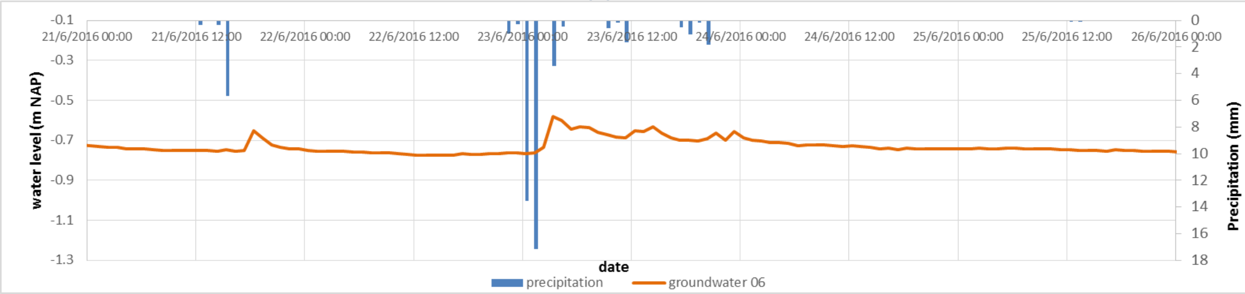


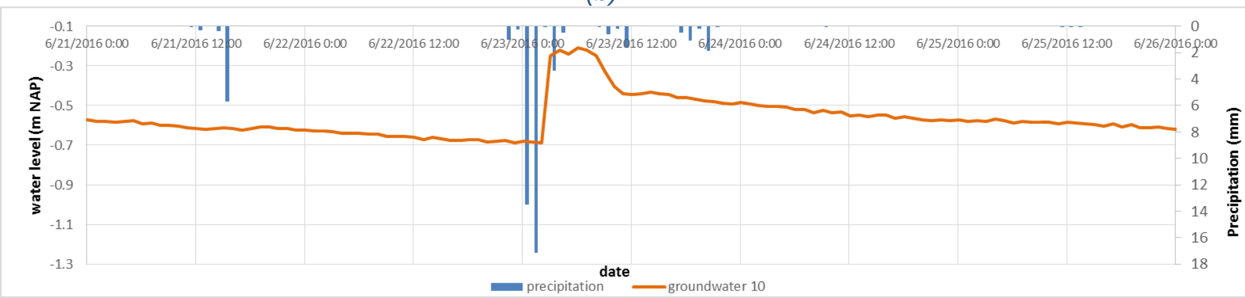
Figure 32 Comparison of groundwater levels at the Well 05, 08, 09, 10, and 13, potential evaporation and sewage water levels during a dry period (from 05-06-2016 to 09-05-2016)



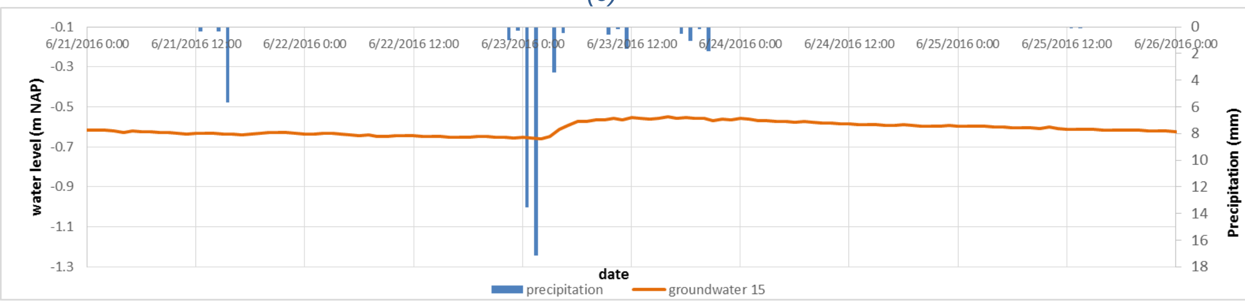
(a)



(b)



(c)



(d)

Figure 33 Comparison of groundwater levels and precipitation during a period when the maximal hourly precipitation occurred (from 21-06-2016 to 25-06-2016)

Furthermore, comparing to the results of slug test in *Chapter 6.2.1*, the results in *Table 5* may indicate the hydraulic conductivity measured at the Well 02, 03, 06, 07 may exist some errors. For instance, Well 02 is about 5 meters away from the Well 03, but the amount of groundwater levels increased in those two wells are quite similar (0.123 m/h at the Well 02 and 0.097 m/h at the Well 03). It is the same case for the Well 06 (rising 0.154 m/h) and 07 (0.18 m/h) since both of them are located along the canal Turfmarkt. It is not realistic that their hydraulic conductivities are so distinctive. Nevertheless, it still cannot explain the differences in the hydraulic conductivity measured at the Well 12 and 15. From *Table 5* and *Figure 33*, it would be expected that the hydraulic conductivity at the Well 15 would be higher than that at the Well 12. But based on the slug test (see *Table 3*), the results are exactly opposite. The results of the slug test may be not as accurate as expected, for the future research, it is recommended to study the hydraulic conductivity in the research area.

Table 5 Increasing level of groundwater levels within 1 hour after max hourly precipitation (17.1 mm) occurred

Well	Land cover	Increasing level (m/h)
01	Brick	0.129
02	Brick	0.123
03	Brick	0.097
04	Brick	0.057
05	Grass	0.180
06	Brick	0.154
07	Brick	0.180
08	Grass	0.186
09	Tree	0.077
10	Tree	0.437
11	Brick	0.150
12	Brick	0.161
13	Tree	0.432
14	Brick	0.175
15	Brick	0.035
1-1.04	Brick	0.114
1-1.05	Brick	0.118
1-1.08	Brick	0.152
1-1.134	Brick	0.114

Figure 34 Identify the most sensitive locations of the observation wells to sewage water, precipitation, and evapotranspiration

On the basis of the observations from the field experiment in *Chapter 6.2.2*, *6.2.3*, and *6.3*, the locations of the observation wells measured at which the groundwater levels are most sensitive to the changes of the sewage water levels, precipitation and evaporation have been identified (see *Figure 34*). The red circles indicate the groundwater measured at those wells is sensitive to the sewage water, the green circles indicate the groundwater measured at those wells is sensitive to evapotranspiration, and the blue circles indicate the groundwater over there is sensitive to precipitation. The Well 08, 13, 1-1.04, 1-1.05, 1-1.134

have been recognized the locations sensitive to the sewage water as they are close to the sewer pipes made of concrete or bricks and constructed in the period between 1869 and 1940. The Well 05, 08, 10 and 13 are the wells where the groundwater has the similar variation pattern with hourly potential evaporation. And the groundwater at the Well 05, 06, 07, 08, 10, 11, 12, and 13 is sensitive to precipitation. Hence, the groundwater at the Well 05 and 10 can be easily affected by weather conditions, and it at the Well 08 and 13 is affected by both sewer system and weather conditions.

6.4 Model simulation

Owing to the limitation of the time, the processes of model calibration and validation have not been completed. In this chapter, it would primarily focus on the attempt to calibrate the model and the parameter sensitivity analysis. However, the project will continue and further calibration on the groundwater flow model will be carried on. The current simulation results can be used to interpret the observations and findings from the field experiment (see *Chapter 6.2*), as well as point out the limitations and improvements for the further investigation (see *Chapter 7*).

6.4.1 Model results

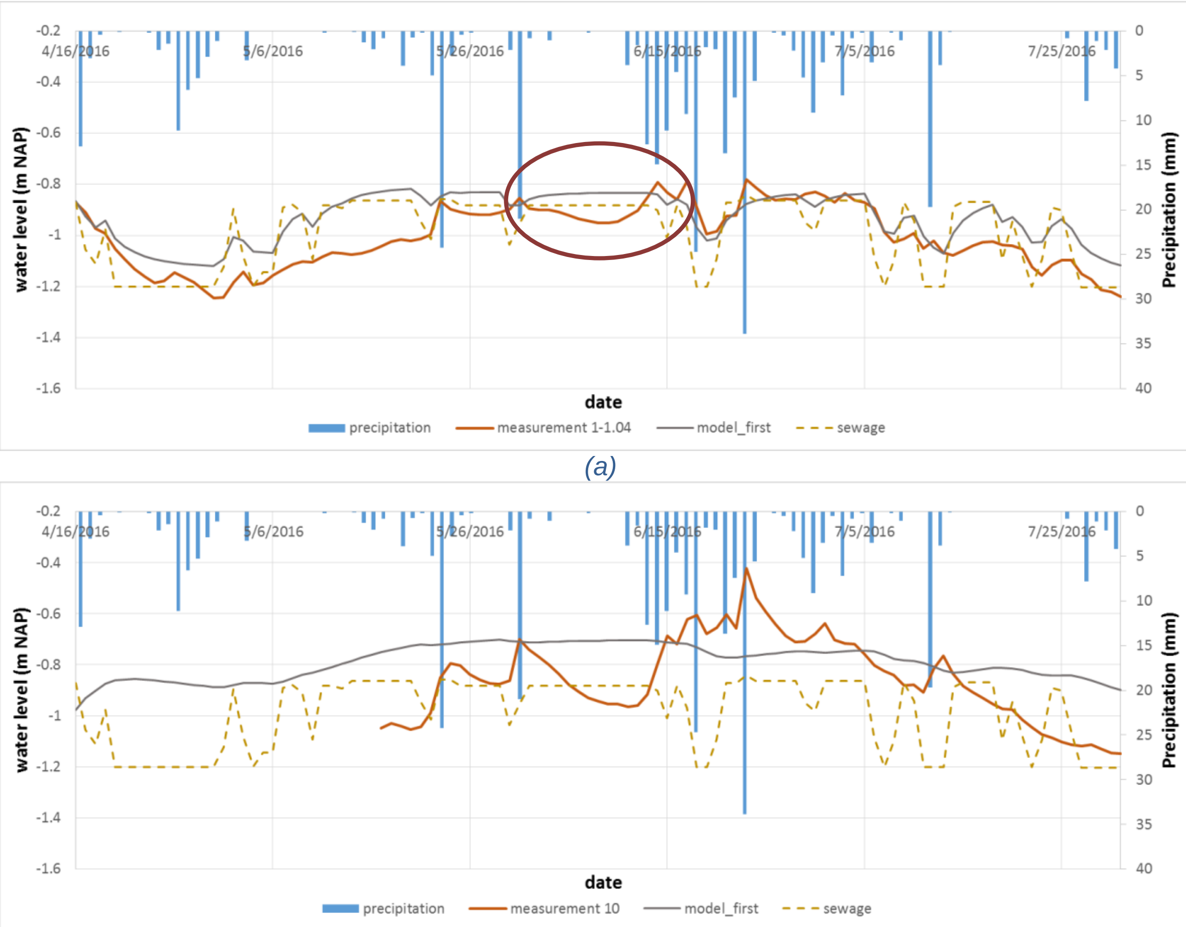
6.4.1.1 Simulation results of the initial model testing

For the first model test, two simulations were done and the results of them were compared. One simulation neglected the existence of old harbor quay along the street Nieuwe Haven, while another simulation took it into consideration. The maximum absolute differences between two simulations at groundwater observation wells were showed in *Table 6*. The differences are merely several millimeters, thereby, the old harbor quay has very little influence on the simulated groundwater levels. The top of the old quay is at the elevation of -1.5 $m\,NAP$, which is about 0.5 to 1 m below the groundwater levels. This might be the reason that it barely has an impact on the simulation heads. Therefore, for the following model calibration and sensitivity test, the existence of these quays will be neglected.

Table 6 Maximum absolute differences between simulation with and without horizontal flow barrier

Well	Maximum of absolute difference (m)
1	0.0036
2	0.0015
3	0.0012
4	0.0004
5	0.0017
6	0.004
7	0.0009
8	0.0005
9	0.0009
10	0.0015
11	0.0017
12	0.0016
13	0.0004
14	0.0006
15	0.0075
1-1.04	0.0014
1-1.05	0.0009
1-1.06	0.0005
1-1.08	0.0001
1-1.09	0.0005
1-1.10	0.0004
1-1.134	0.0002

Figure 35 gives examples of the initial simulation results at the Well 1-1.04 and 10. The initial simulation results at other wells can be found in *Appendix 5 Model simulation results*. In *Figure 35*, the red lines are the average daily groundwater levels, the gray lines are the simulation heads of the initial model with MetaSWAP, and the dotted orange lines are the average sewage water levels. During the period from 31st of May to 10th of June, 2016 (marked with the red circle in *Figure 35 (a)*), when there was no precipitation, the measured groundwater levels decreased due to the evaporation effect, while the modeled water levels basically remained the same. At the same time, the simulated heads at the Well 10, which is located in the vicinity of trees, did not display any variation pattern in accordance with precipitation or potential evaporation (see *Figure 35 (b)*). Both of the graphs suggest that the effects of precipitation and evapotranspiration on the groundwater were not simulated by the model, which indicates that the embedded program MetaSWAP might not perform properly in this case. For the purpose to testify this conjecture, the model will be simulated without MetaSWAP, and the results can be found in *Chapter 6.4.1.2* and *Appendix 5 Model simulation results*.



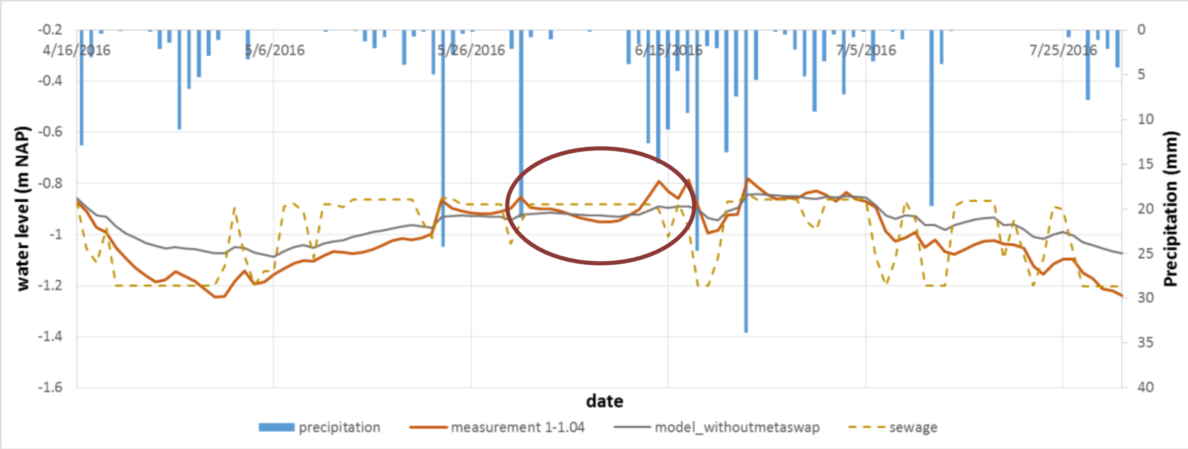
(a)

(b)

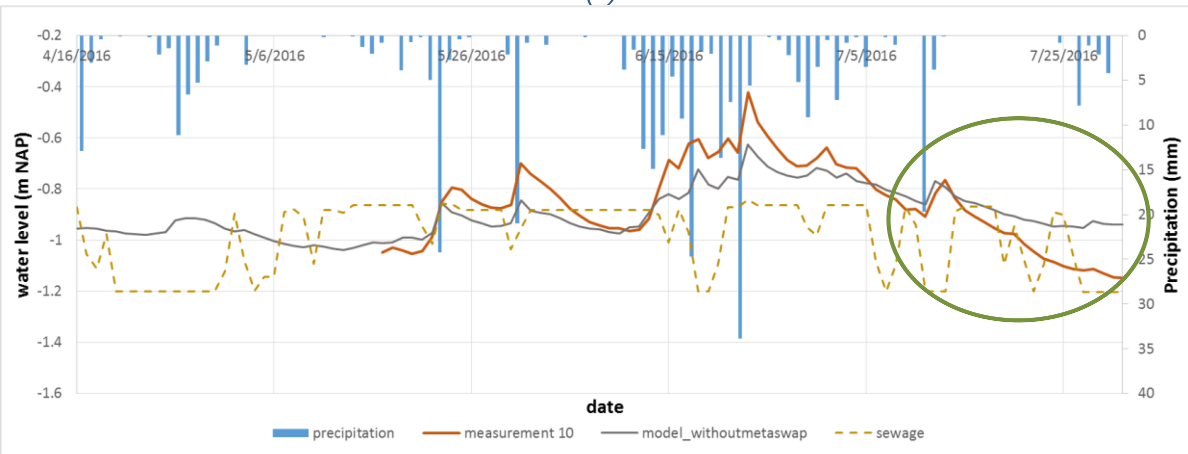
Figure 35 Simulation results of the initial model test at the Well 1-1.04 and 10

6.4.1.2 Simulation results of the model without MetaSWAP

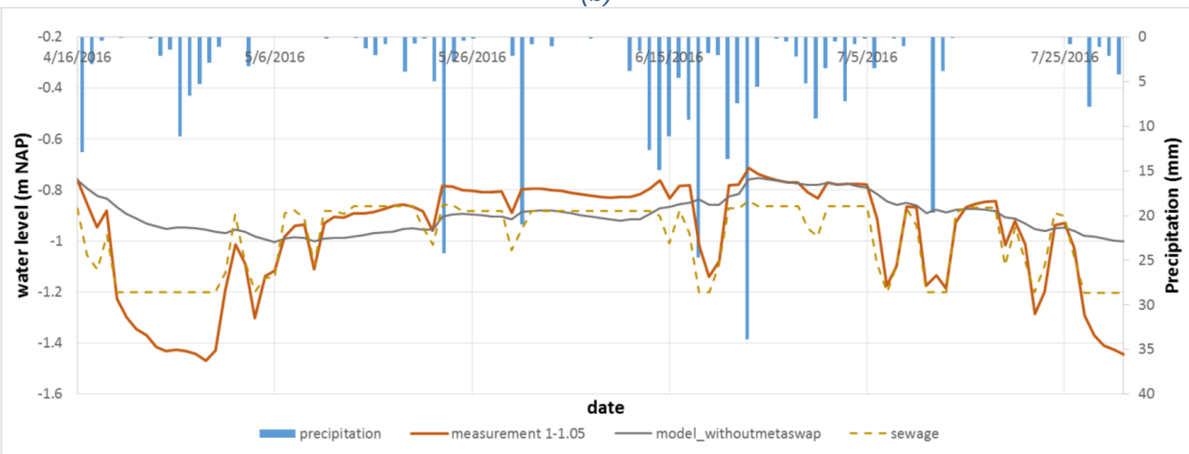
In order to test the conjecture mentioned in *Chapter 6.4.1.2* for a short time, the precipitation and the potential evaporation will be considered to be the same value for the whole model area in each time step, without regard for the influence of land use on the amount of precipitation infiltrating to the underground and amount of groundwater extracting to the atmosphere during the evapotranspiration.



(a)



(b)



(c)

Figure 36 Simulation results of the model without MetaSWAP at the Well 1-1.04, 10, and 1-1.05

Since the precipitation and potential evaporation were included in the CAP Module, where the MetaSWAP is embedded, once the MetaSWAP is disregarded, another two packages, namely, RCH (Recharge) and EVT (Evapotranspiration), should be involved in the model simulation to represent precipitation and potential evaporation. The RCH Package refers to the recharge strength (mm/day) from the precipitation that could infiltrate and recharge to the groundwater. And the EVT Package defines the evapotranspiration owing to the transpiration by vegetation and the evaporation from the unsaturated zone. It requires the evapotranspiration strength (mm/day), the top elevation for maximal evapotranspiration strength ($m + MSL$), and the thickness over which the evapotranspiration strength is reduced to zero (m) (*Vermeulen et al., 2016*). As there is not enough investigation and data available for the model area, in this case, the top elevation for maximal evapotranspiration strength was set as $1\ m$ below surface level, and the thickness over which the evapotranspiration strength is reduced to zero was chosen to be $1\ m$. These values were chosen at random since there are no investigation and understanding on the parameters. If the EVT package continues to be included in the model, these parameters will require calibration.

Figure 36 shows the examples of the model simulation without MetaSWAP at the Well 1-1.04, 10, and 1-1.05. The model results without MetaSWAP for other observation wells are in *Appendix 5 Model simulation results*. Comparing *Figure 35 (a)* and *Figure 36 (a)*, the simulation without MetaSWAP performs better during the dry period marked with the red circles in the graphs. And this model is able to simulate some of the variation of the groundwater levels caused by precipitation and evapotranspiration at the Well 10 (see *Figure 36 (b)*). The comparison of these four graphs proves that the conjecture that the MetaSWAP might not perform properly is plausible. It is understandable that the program MetaSWAP is complicated, and lots of additional input data required by MetaSWAP has not been measured or studied in the model area. Based on a clipping part for the model area from a national groundwater flow model, it is conceivable that such a regional model is not suitable for the simulation in such a small area. The potential solution is to involve the experts in the MetaSWAP and collect necessary data during the model construction. Moreover, because of the homogeneity assumed for the recharge from precipitation and evapotranspiration, as well as the assumptions made for EVT Package, the model results are not accurate enough to compare to the measurements. However, the simulated groundwater heads are able to indicate the variation degree from the model simulation, which can be used to be as a signal on the performance of the model by comparing to the variation degree from the observation data.

Although the model without the MetaSWAP performs better than the initial model on groundwater variation trend, from *Figure 36 (b)*, it cannot simulate the decreasing of the groundwater levels at the Well 10 from 13th of July, 2016 until the simulation ends (marked with a green circle). The groundwater dropped about $0.4\ m$ during a period of 20 days. It is possible that the transpiration rate at this location is way much higher due to the existence of trees. While, it is also possible that the groundwater could infiltrate to the deeper aquifer. Until now, it cannot be explained by the model results. At the same time, the hydraulic conductance of the sewer system $CRIV_{sew}$ near the Well 1-1.05 should be larger, since the groundwater levels show a palpable variation with the sewage water levels. This will be investigated in the *Chapter 6.4.2*.

6.4.2 Sensitivity analysis

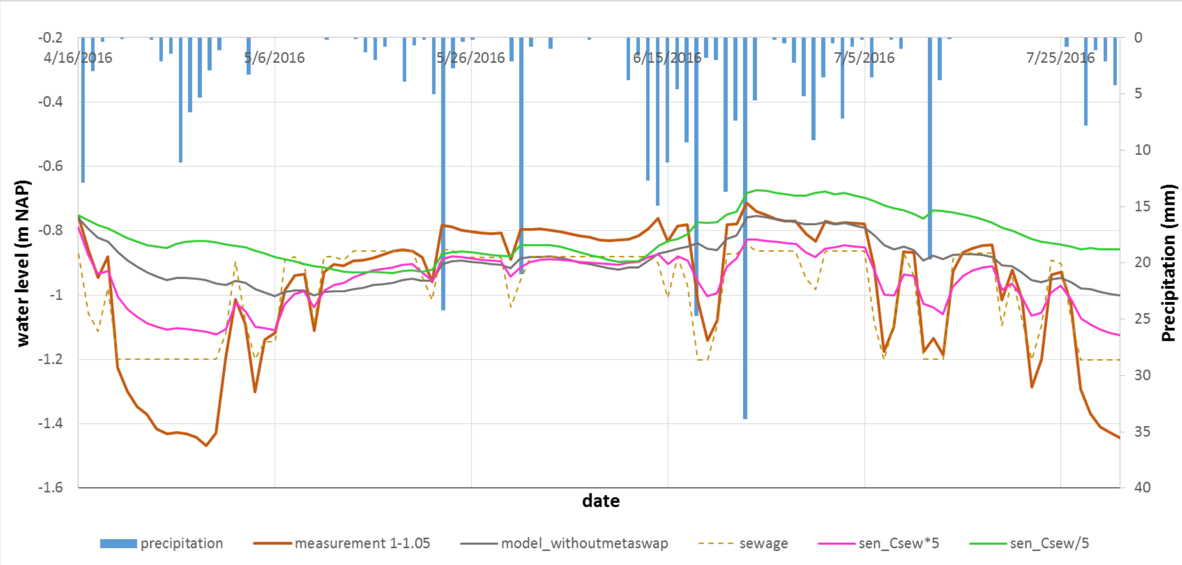
As mentioned in *Chapter 5.3.3*, the parameter sensitivity analysis will only focus on the parameter – hydraulic conductivity of the sewer system $CRIV_{sew}$. The initial $CRIV_{sew}$ value is the same value of $0.35\ m^2/day$ for all the pipes, it was multiplied by 5 and divided by 5 for the parameter analysis. Then the values of $CRIV_{sew}$ become $1.75\ m^2/day$ for $[CRIV_{sew} \times 5]$ and $0.07\ m^2/day$ for $[CRIV_{sew}/5]$, separately. The results of /the sensitivity analysis can indicate where the sensitive locations of groundwater levels to $CRIV_{sew}$ are, and give the first impression about the ranges of a suitable value of $CRIV_{sew}$ for each sewer pipe. The complete analysis results are showed in *Table 7* and *Appendix 5 Model simulation results*.

The evaluation about the sensitivity degree of the groundwater levels measured at each observation well is based on the comparison between the simulated heads of $[CRIV_{sew} \times 5]$ and simulated heads of $[CRIV_{sew}/5]$ (see examples shown in *Figure 37*). In *Figure 37*, the red lines are the measured groundwater levels, the gray lines are simulation results from the model without MetaSWAP, which are used as the reference model simulations in this case, the blue lines are the simulated heads of $[CRIV_{sew} \times 5]$, green lines are the simulated heads of $[CRIV_{sew}/5]$, and the orange dotted lines are the sewage water levels. If the shape between blue and green lines is distinct, it illustrates that the groundwater level measured at this well is comparatively sensitive to the leakage situation of the sewer system in the neighborhood, as displayed in *Figure 37 (a)*. If the shape of those two lines is similar, and the simulated heads have little differences (see *Figure 37 (b)*), it means the groundwater level measured at this point is basically not sensitive to the leakage situation of the sewer system in the neighborhood. And the sensitivity results shown in-between of these two situations are identified as medium sensitivity degree to the parameter $CRIV_{sew}$.

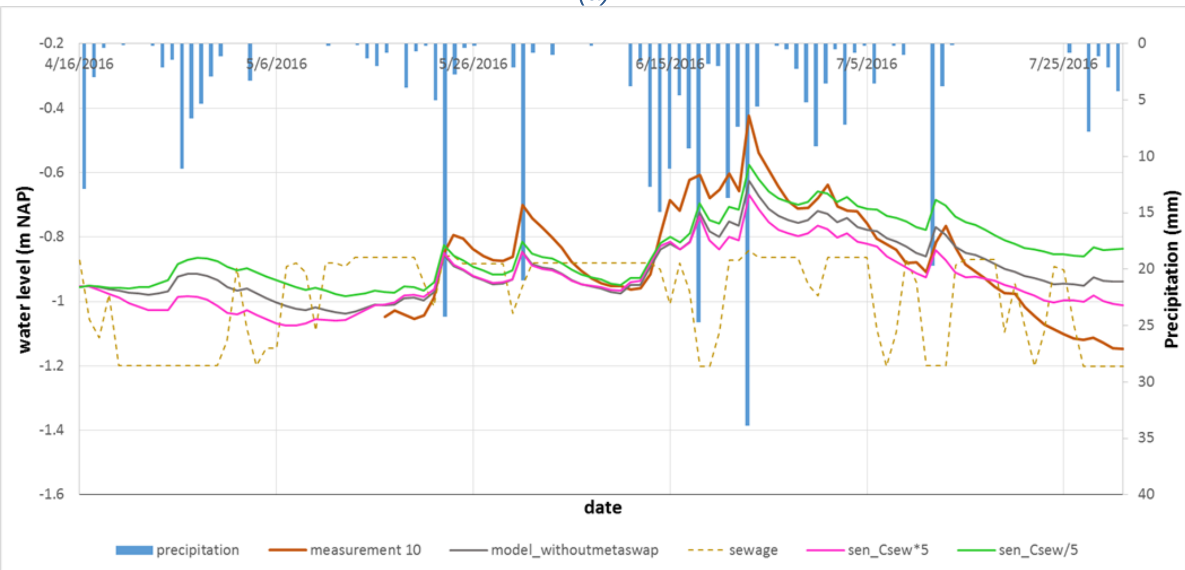
Table 7 Results of sensitivity analysis on the indication about the suitable $\boldsymbol{CRIV_{sew}}$ ranges

Groundwater observation well	Back-stowed sewer system nearby		Sensitivity degree	Suitable $\boldsymbol{CRIV_{sew}}$ range $(\boldsymbol{m^2/d})$
	Construction year	Material		
01	1995	GVK	High	0.07 ~ 0.35
02	1940	Beton (concrete)	High	0.35 ~ 1.75
03	1940	Beton (concrete)	High	0.35 ~ 1.75
04	-	-	Medium	-
05	1986	PVC	High	< 0.07
06	1991	PVC	High	< 0.07
07	1991	PVC	Medium	0.07 ~ 0.35
08	1870	Metselwerk (brick)	Medium	> 1.75
09	-	-	Low	-
10	1869	Beton (concrete)	Medium	0.07 ~ 0.35
11	-	-	Low	-
12	-	-	Low	-
13	1869	Beton (concrete)	High	0.35 ~ 1.75
14	2004	PVC	High	0.07 ~ 0.35
15	2004	PVC	High	0.07 ~ 0.35
1-1.04	1940	Beton (concrete)	High	0.35 ~ 1.75
1-1.05	1869	Beton (concrete)	High	> 1.75
1-1.08	1970	PVC	High	0.07 ~ 0.35
1-1.134	1940	Beton (concrete)	High	0.35 ~ 1.75

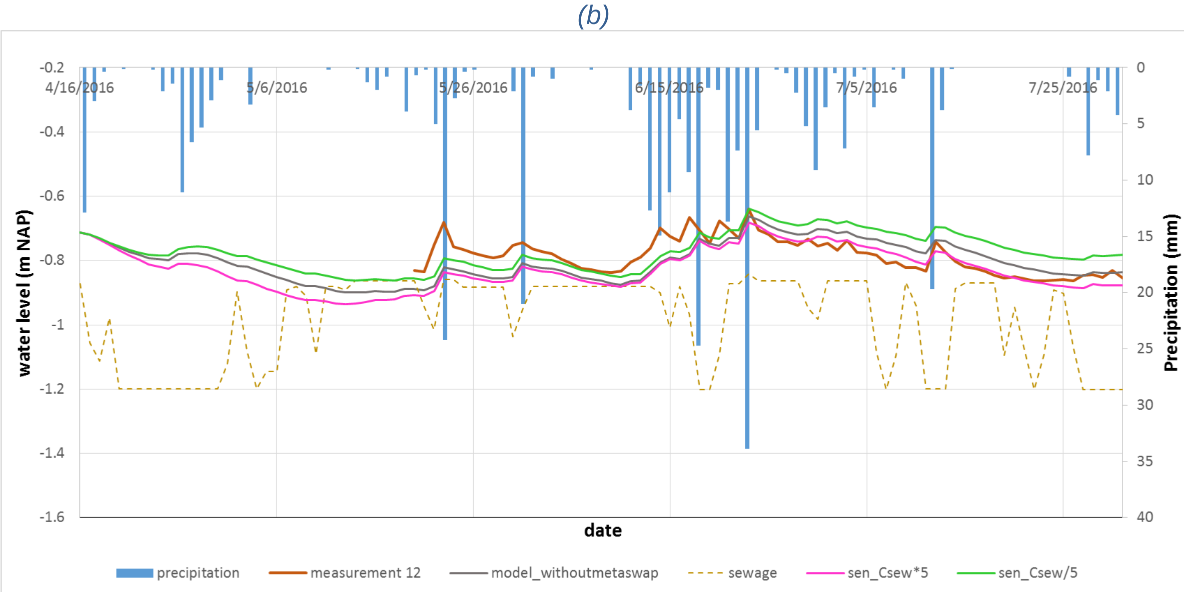
According to the results in *Table 7* and *Appendix 5 Model simulation results*, the locations of the observation wells, which have been recognized to have high sensitivity degree to the parameter $CRIV_{sew}$, are marked with red circles in *Figure 38*. There are 11 wells identified, and they are all situated close to the back-stowed system. Hence, the closer the groundwater to the potentially leaky back-stowed sewer system is, it is more influenced by the leaking situation of the sewer pipes. Nevertheless, the Well 07 is located in the vicinity of a sewer pipe, the model results do not identify the groundwater measured there is sensitive to $CRIV_{sew}$. The possible reason is that the observation well is also close to the canal, the impact of the stable surface water level might neutralize the influence from the leaky sewer pipe.



(a)



(b)



(c)

Figure 37 Parameter sensitivity results at the Well 01, 10, and 07

What is more, *Table 7* indicates suitable value ranges of the parameter $CRIV_{sew}$ for the sewer pipes near each observation well. Generally speaking, the sewer pipes built in the 19th century have been identified as considerably leaky, the $CRIV_{sew}$ values for them are suggested to be higher than 1.75 m^2/day in order to have a better simulation on the groundwater levels (see *Figure 37 (a)*). The sewer pipes built by concrete in the 20th century are identified to be relatively damaged, but not as terrible as those even older sewer pipes. Therefore, the $CRIV_{sew}$ values are recommended to be in the ranges of 0.35 to 1.75 m^2/day. In addition, the sewer pipes made of PVC and GVK seem to be water-tight or a little bit leaky, the $CRIV_{sew}$ values can be rather small (between 0.07 and 0.35 m^2/day, or even smaller than 0.07 m^2/day). On the whole, the findings from the sensitivity analysis are consistent with those from the field experiment.

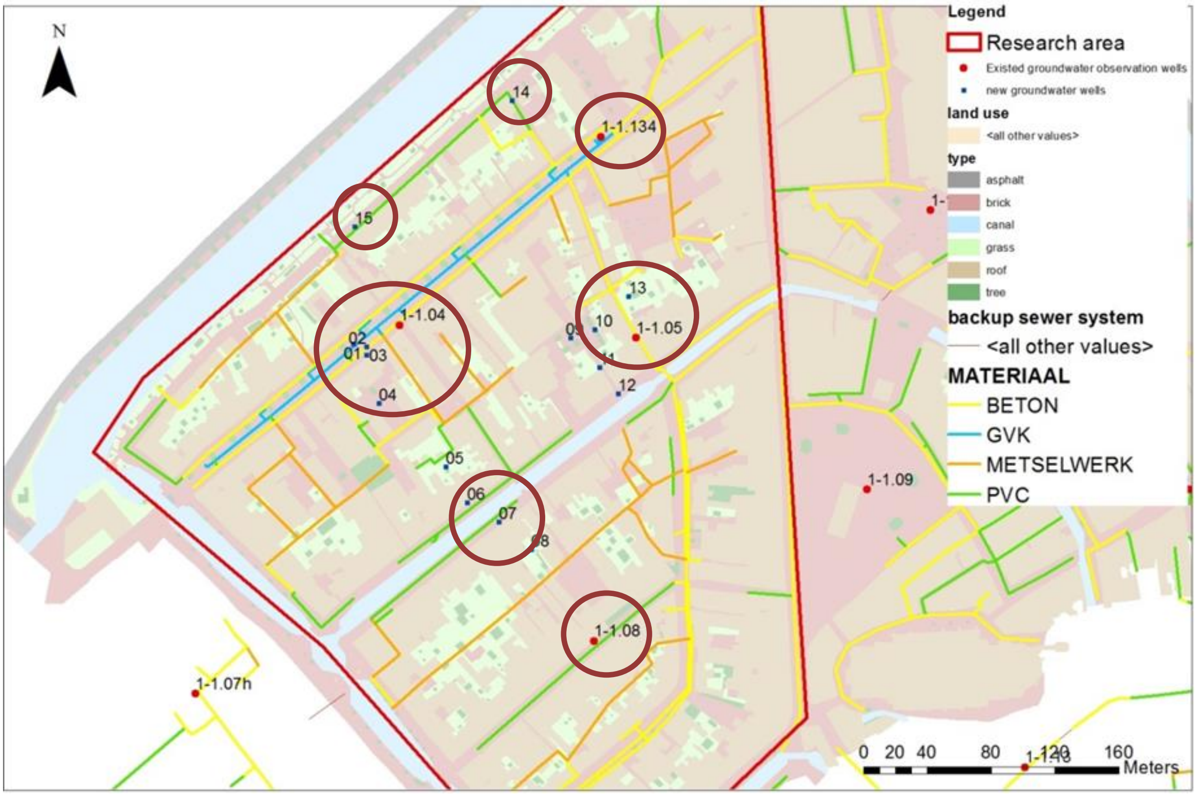


Figure 38 Locations of the observation wells which have a high sensitivity degree to $\boldsymbol{CRIV_{sew}}$

7 Conclusions and Recommendations

As the starting point of the Gouda project carried through by the Dutch Coalition “Stevige stad op slappe bodem”, this research concentrates on the investigation of the fluctuation characteristics of groundwater in the Nieuwe Haven and Centrum areas, in the interest of alleviating the damages brought by subsidence and constructing a climate-resilient city through controlling groundwater levels sufficiently and effectively in the future. Due to the fact that seldom research has focused on the urban groundwater management related to land subsidence so far, the methods, procedures, experience and lessons accumulated in this research can be considered as a guide or assistance for other areas or cities confronting the similar problems. In this chapter, it will summarize the procedures and methods taken during the research period, as well as their corresponding results. In addition, since the project is still in progress, based on the current discoveries, some recommendations are listed at the end of this chapter for further investigation and improvements.

The research initiates with the identification of the current vulnerabilities caused by climate change and land subsidence in the research area (see *Chapter 2*). Land subsidence is primarily the consequence of peat oxidation and clay compaction which occurred gradually since the last century in the Netherlands. It has led to the uneven settling between buildings or roads which use different technologies on the foundation construction. At the same time, it causes dysfunction of the underground infrastructures, such as the sewer system, which has been recognized as one of the major influential factors on groundwater levels in the research area. As for climate change, the most important aspects that could affect the research areas are extreme rainfall events, global warming, and sea level rise. It indicates the higher possibility of inundation during heavy rainfall events and exposure of wooden poles and peat because of low groundwater levels in hot summer days, which could go a step further to accelerate the subsidence rate.

In consideration of the vulnerabilities discussed above, it is self-evident that controlling groundwater within the desired regime is an efficient intervention to alleviate land subsidence indirectly, which requires a full assessment of groundwater fluctuation pattern temporally and spatially. Therefore, a more representative groundwater observation network with additional 15 new wells was designed and implemented on 20th of May, 2016 (see *Chapter 5.1* and *6.1*). The new observation network takes into account of the influential factors on groundwater (land use, vegetation types, sewer system, and surface water), susceptible areas to groundwater fluctuation (wooden foundations, crawl spaces, and backyards), and accessibility for subsequent validation and maintenance.

Once the new groundwater observation network started the measurements properly, the field experiment (see *Chapter 5.2.2* and *6.2.2*) was implemented twice to investigate the leaky extent of the back-stowed sewer system. The experiment was done by opening the weir between the back-stowed system and pumping system for two days, then the sewage water levels in the back-stowed system decrease correspondingly. If the sewer system is leaky, the groundwater levels in its neighborhood should decrease as well. The groundwater levels measured at the observation wells were used to compare to the sewage water levels measured at the CSOs. Meanwhile, the time series of hourly precipitation and potential evaporation were compared to the groundwater levels as well (see *Chapter 6.3*). Furthermore, the groundwater levels along the cross-section (see *Chapter 6.2.3*) from the canal Kattensingle, via the street Nieuwe Haven, the canal Turfmarkt, until street Lange

Groenendaal under different sewage water levels and weather conditions were drawn and analyzed as well.

At the end of the research, a 2D layered finite-difference transient daily groundwater model was built with the help of iMODFLOW v2.6.37 (see *Chapter 5.3* and *6.4*) for the research area. And the parameter analysis of the hydraulic conductance of the sewer system $CRIV_{sew}$ was carried on. The results of the parameter analysis could recognize the area where is sensitive to the leakage of the sewer system. At the same time, it would provide a suitable range of the conductance of the sewer pipes for the further calibration of the model.

The results of the field experiment and parameter analysis illustrate that the leaking extent of the back-stowed sewer system depends on both the year of the construction and pipe material. Generally speaking, the sewer pipes built in the 19th century by concrete or bricks have identified as considerably leaky, the $CRIV_{sew}$ values of which are suggested to be higher than 1.75 m^2/day; the sewer pipes built by concrete in 1940 are leaky as well (between 0.35 and 1.75 m^2/day;), but not as terrible as those older sewer pipes; and the sewer pipes made of PVC and GVK seem to be at a relatively low extent of leakage (between 0.07 and 0.35 m^2/day, or even smaller than 0.07 m^2/day). As regards the response of the groundwater levels to the fluctuation of the sewage water levels, it is up to the leakage level of the sewer pipes, as well as the distance to the leaky pipes. From the data analysis, the influential area of the leaky sewer system could reach as far as 30 to 50 m. At the same time, even though two sewer pipes were built in the same year using the same material, the extent of leakage would be different.

Moreover, in accordance with the comparisons among groundwater levels, sewage water levels, precipitations, and potential evaporation, the Well 08, 13, 1-1.04, 1-1.05, 1-1.134 have been identified the locations where is more sensitive to the sewage water levels because they are situated in the vicinity of the rather old and leaky sewer pipes; the Well 05, 08, 10 and 13 are the wells at which the groundwater has similar variation pattern as the potential evaporation within a day, suggesting that vegetation has an impact on the fluctuation of the groundwater due to the evapotranspiration; and the groundwater measured at the Well 05, 06, 07, 08, 10, 11, 12, and 13 reacts considerably fast to the intense rainfall events, where unpaved and riparian areas are located. Thereby, the groundwater levels at the Well 08 and 13 are rather sensitive to sewer water levels, precipitation, as well as evapotranspiration. Meanwhile, the Well 01, 02, 03, and 1-1.08 have not been identified to be sensitive to any influential factors to a great degree, and the abnormal phenomenon existed at the Well 04 during the field experiment and extremely low levels at the Well 09 have not been able to be explained by the current research yet.

The whole project is still in progress, and certain aspects of current investigation require further improvements. Based on this research, some recommendations are brought forward for the following procedures in order to consummate the project. These recommendations will be given in three respects, namely, data collection, model calibration, and suggestions on the investigation in the other areas of the inner city of Gouda.

First of all, several recommendations for data collection are given below. They are related to the measurements of surface water and sewage water levels, improvements of the current groundwater observation network, as well as additional information needed for the research about the relationship between groundwater level and foundation.

- It is suggested to install a data logger to measure the surface water levels in the canal Turfmarkt since the freeboard over there is merely several centimeters, the fluctuation of the water level is significant to study the possible interventions to prevent inundation.

- It is also suggested to install a data logger to measure the sewage water levels at the weir, where the sewage water in the back-stowed system is regulated. In *Chapter 5.2.2*, it has mentioned that the bottom elevation of the sewer system at the current sewage water measurement point (CSO) is higher than that at the weir. The data input in the model now is not representative enough for the reality.
- The Well 01, 02, and 03 in the current observation network do not fulfill their originally design functionality (to study the influence of the sewage water on the groundwater) because the sewer pipes in the neighbor are not as leaky as assumed. These wells can be moved near the Well 1-1.04, 1-1.05, or 08, where the sewer pipes have been identified to be leaky. Alternatively, they can be also relocated in the backyards or near buildings of wooden foundations.
- Moreover, the extremely low groundwater levels at the Well 09 have not been able to be explained, additional wells are recommended to install in the vicinity of the Well 09.
- Since there are no complete investigations on the foundation types in the research area and the presently available data is not adequate enough to study whether the top of the wooden foundation would emerge beyond the groundwater. More information is needed for the investigation at the Well 14 and 15.

Secondly, the current groundwater flow model is not complete and good enough to represent the reality, further calibration is required to improve the model performance. In this part, the recommendations are concentrated on the model calibration and extra scenarios can be simulated once the model can display a rather good result.

- The precipitation and evapotranspiration for the RCH and EVT packages should be altered according to the land cover. Yet, extra parameter calibration will be needed for the EVT package. If possible, it would be helpful to include MetaSWAP in the model as well, but it requires more knowledge and experience about the MetaSWAP program.
- The calibration of the hydraulic conductivity of the first sandy layer and the second peaty and clayey layer should be carried on. The hydraulic conductivity of the first layer is from the results of the slug test, which was believed to have errors. And the hydraulic conductivity in the second layer is from a groundwater flow model built for the company Croda, it is possible that the underground situation is different in the research area.
- The starting head should be corrected, as the value measured at the beginning was the groundwater level which had not bounced back to the normal level after the pumping during the installation.
- The calibration on the hydraulic conductivity of the sewer system should take into consideration of the construction year and pipe material. Different pipes should be given different values to improve the model performance.
- Once the model performs rather well, several management scenarios could be simulated by the model to analyze their possible consequences and provide insightful proposals on the interventions in the future.
    - Scenario 1: decrease the surface water level maintained by the pumping stations;
    - Scenario 2: renovate all the leaky back-stowed sewer system with new water-tight pipes;
    - Scenario 3: construct new water-tight pipes, but remain the old leaky sewer system as subsurface drainage system;
    - Scenario 4: replace the leaky pipes with new water-tight pipes, and construct a new drainage system.

At the end, for the other drainage areas in the inner city of Gouda, where had not been identified that the sewage water had a more significant impact on the groundwater levels, the priority should be put on the groundwater level observations close to the sewer pipes made of concrete or brick and built in the 19^{th} or 20^{th} century. In addition, there is no big unpaved area of vegetation in the research area, extra observation wells should be implemented in the city parks.

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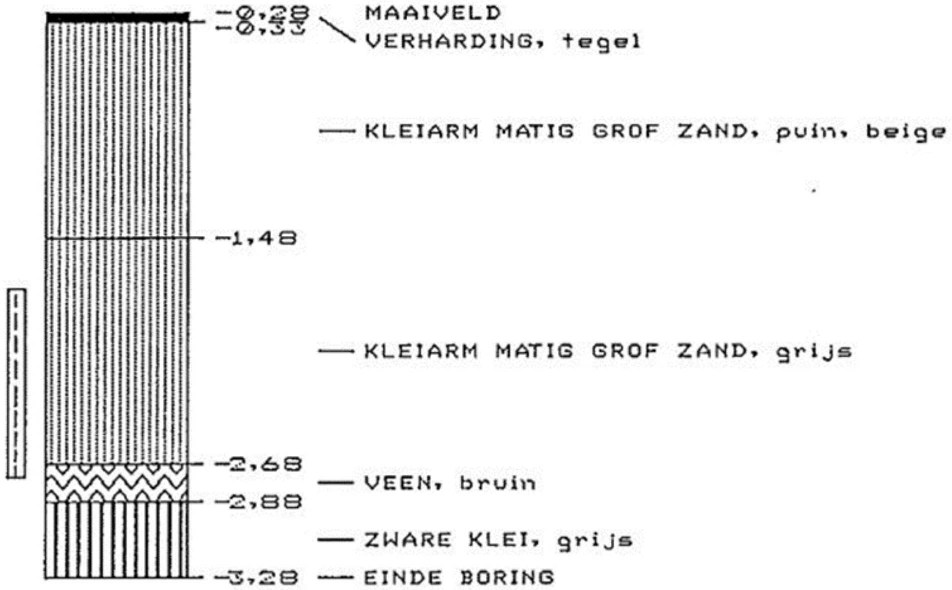
Appendix 1 Borehole information

Boringnummer: 1-1.04 Project: Nieuwe Haven e.o., Gouda

Locatie: Zie locatietekening.

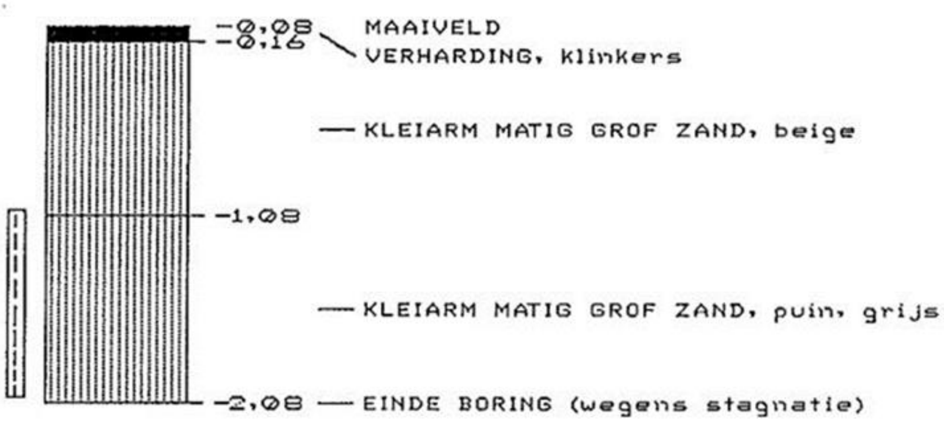
Datum: 30-06-1993 Projectcode: 767



Boringnummer: 1-1.05 Project: Nieuwe Haven e.o., Gouda

Locatie: Zie locatietekening.

Datum: 01-07-1993 Projectcode: 767



Boringnummer: 1-1.06 Project: grondwatermeetnet, Gouda

Locatie: Zie locatietekening.

Datum: 21-06-1994 Projectcode: 768

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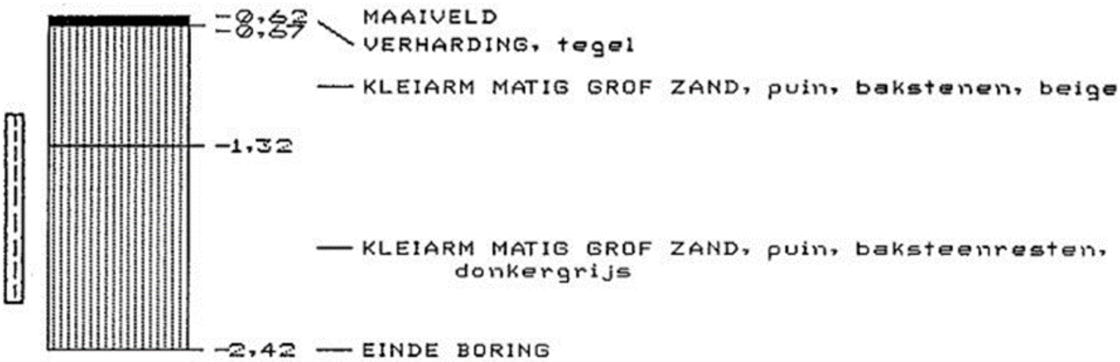
Locatie: Zie locatietekening.

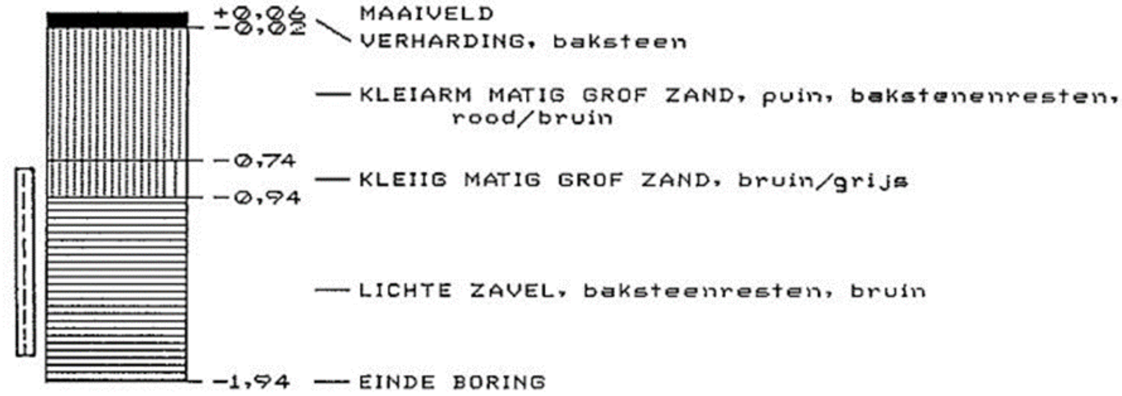
Datum: 20-06-1994 Projectcode: 768



Boringnummer: 1-1.09 Project: grondwatermeetnet, Gouda

Locatie: Zie locatietekening.

Datum: 21-06-1994 Projectcode: 768



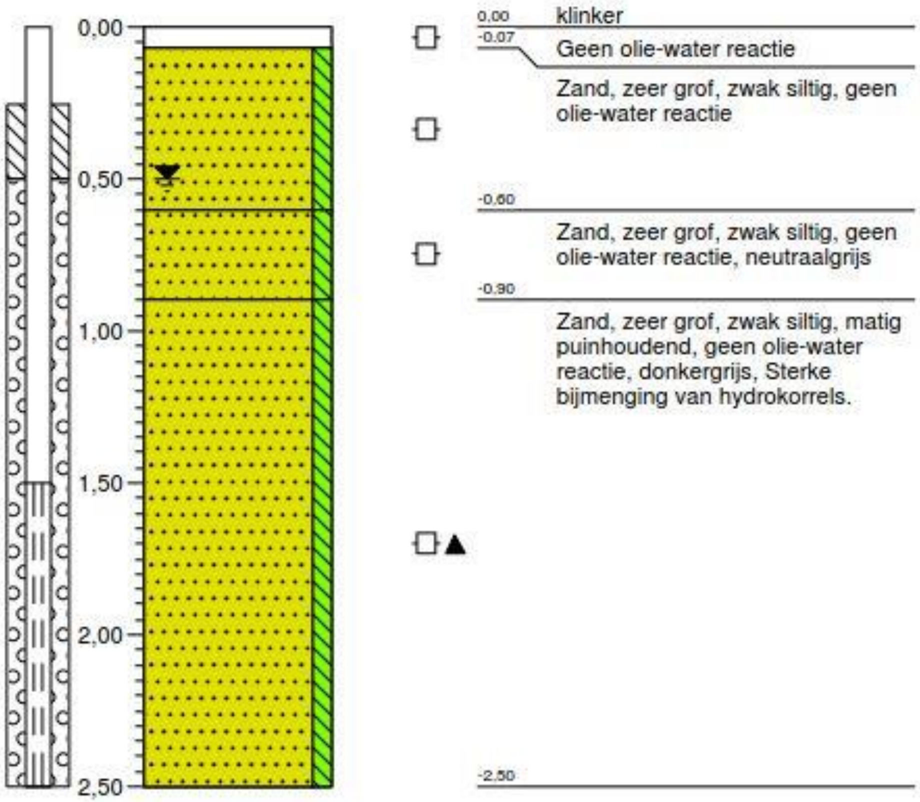
Boringnummer: 1-1.10 Project: grondwatermeetnet, Gouda

Locatie: Zie locatietekening.

Datum: 21-06-1994 Projectcode: 768

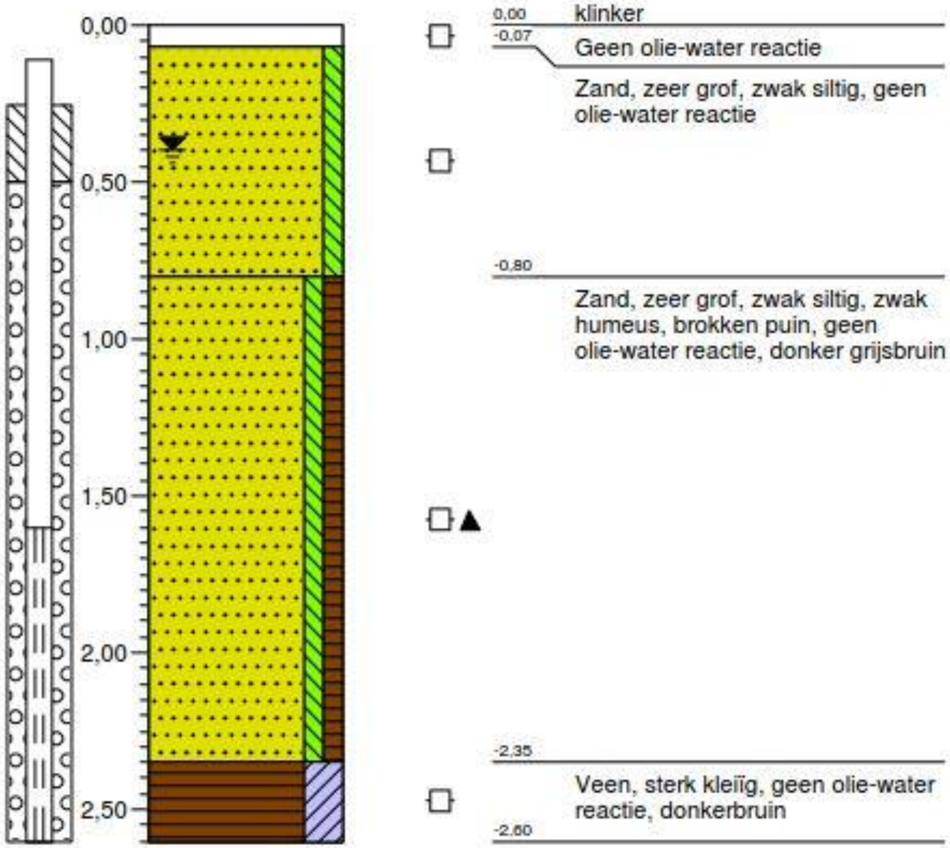
Meetpunt: 01

Datum: 18-05-2016
X: 108238,50
Y: 447349,97



Meetpunt: 02

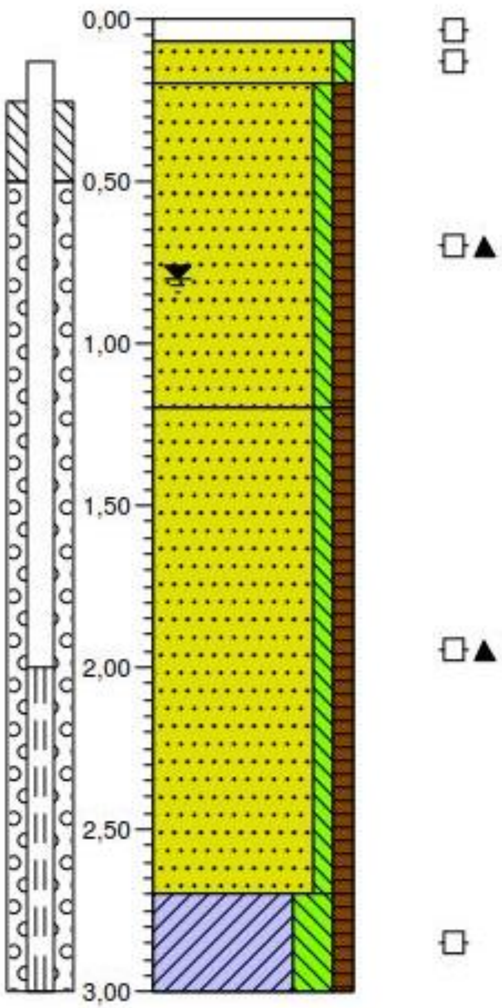
Datum: 18-05-2016
X: 108245,60
Y: 447342,22



5

Meetpunt: 03

Datum:	18-05-2016
X:	108245,73
Y:	447342,60



0,00 klinker

-0,07 Geen olie-water reactie

-0,20 Zand, zeer grof, zwak siltig, geen olie-water reactie

Zand, zeer grof, zwak siltig, zwak humeus, brokken puin, geen olie-water reactie

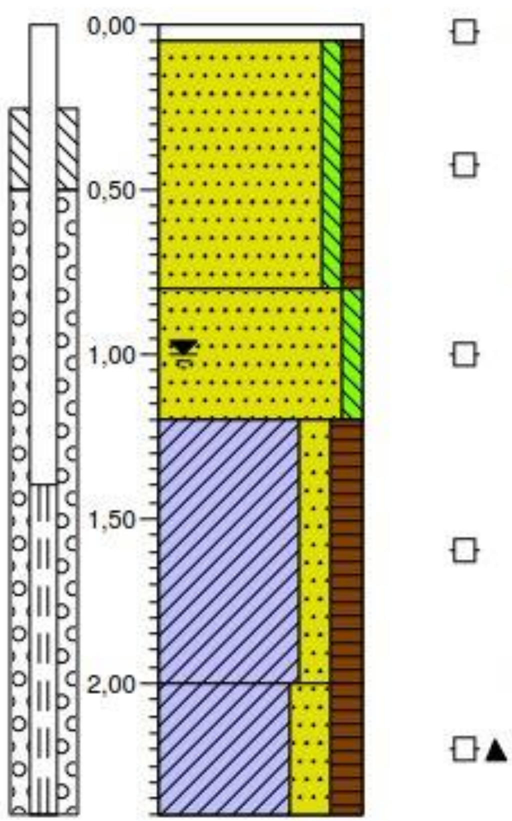
-1,20 Zand, zeer grof, zwak siltig, zwak humeus, brokken puin, geen olie-water reactie, neutraal bruingrijs

-2,70 Klei, sterk siltig, zwak humeus, geen olie-water reactie, neutraal grijsbruin

-3,00

Meetpunt: 04

Datum:	17-05-2016
X:	108252,86
Y:	447315,06



0,00 tegel

-0,05 Geen olie-water reactie

Zand, zeer grof, zwak siltig, zwak humeus, geen olie-water reactie, lichtbruin

-0,80 Zand, zeer grof, zwak siltig, geen olie-water reactie, neutraalgrijs

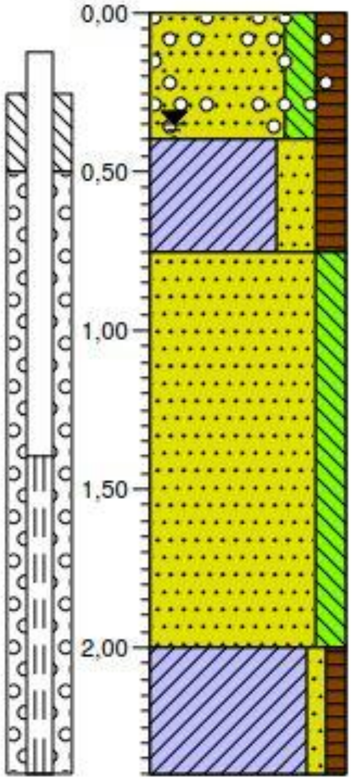
-1,20 Klei, matig zandig, matig humeus, geen olie-water reactie, neutraal grijsbruin

-2,00 Klei, sterk zandig, matig humeus, matig zandhoudend, geen olie-water reactie, neutraal bruingrijs

-2,40

Meetpunt: 05

Datum: 17-05-2016
X: 108295,09
Y: 447275,28



0,00 grind

Zand, zeer grof, matig siltig, matig humeus, sterk grindhoudend, geen olie-water reactie, donkerbruin

-0,40

Klei, sterk zandig, matig humeus, matig puinhoudend, geen olie-water reactie, donkerbruin

-0,75

Zand, matig grof, matig siltig, uiterst puinhoudend, geen olie-water reactie, neutraalgrijs

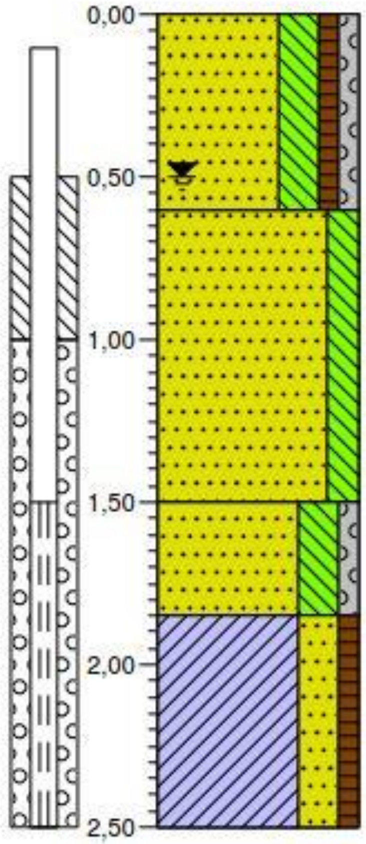
-2,00

Klei, zwak zandig, zwak humeus, zwak puinhoudend, geen olie-water reactie, licht bruingrijs

-2,40

Meetpunt: 06

Datum: 12-05-2016
X: 108308,23
Y: 447252,78



0,00 klinker

Zand, matig fijn, sterk siltig, zwak humeus, zwak grindig, matig puinhoudend, neutraal grijscreme

-0,60

Zand, matig fijn, matig siltig, neutraalgrijs

-1,50

Zand, matig fijn, sterk siltig, zwak grindig, sterk puinhoudend, neutraalgrijs

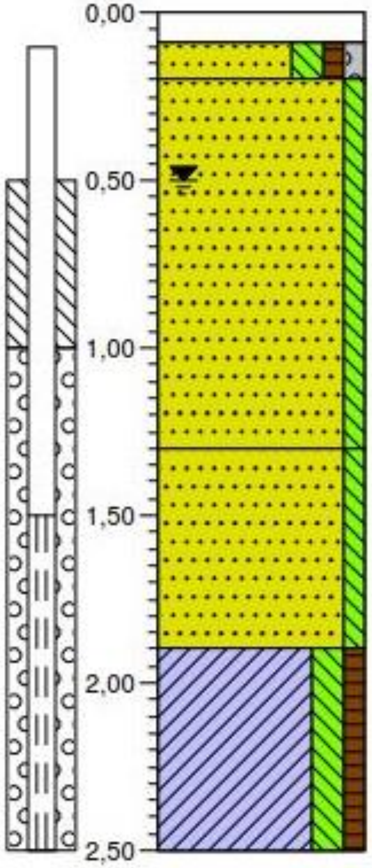
-1,85

Klei, sterk zandig, zwak humeus, neutraalgrijs

-2,50

Meetpunt: 07

Datum: 12-05-2016
X: 108328,33
Y: 447240,62



0,00 klinker

-0,09 Zand, matig fijn, matig siltig, zwak humeus, zwak grindig, lichtbruin

-0,20 Zand, matig fijn, zwak siltig, laagjes puin, neutraalgrijs

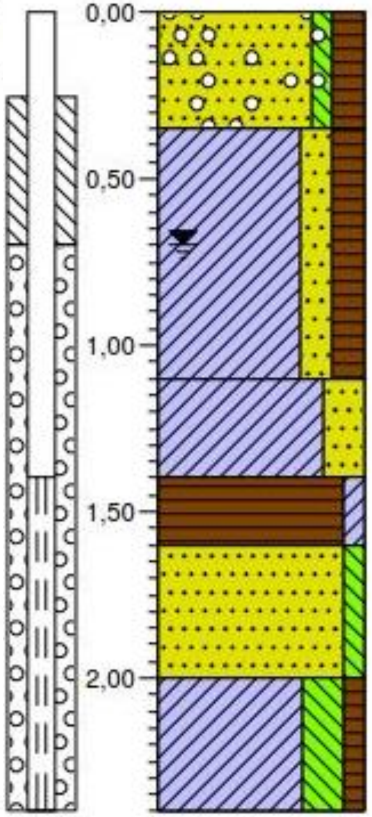
-1,30 Zand, matig fijn, zwak siltig, laagjes veen, neutraal bruingrijs

-1,90 Klei, matig siltig, zwak humeus, laagjes veen, neutraal bruingrijs

-2,50

Meetpunt: 08

Datum: 17-05-2016
X: 108348,33
Y: 447223,50



0,00 tuin

Zand, zeer grof, zwak siltig, matig humeus, matig grindhoudend, geen olie-water reactie, donkerbruin

-0,35 Klei, matig zandig, matig humeus, zwak puinhoudend, geen olie-water reactie, donkerbruin

-1,10 Klei, sterk zandig, geen olie-water reactie, lichtgrijs

-1,40 Veen, zwak kleiïg, geen olie-water reactie, neutraalbruin

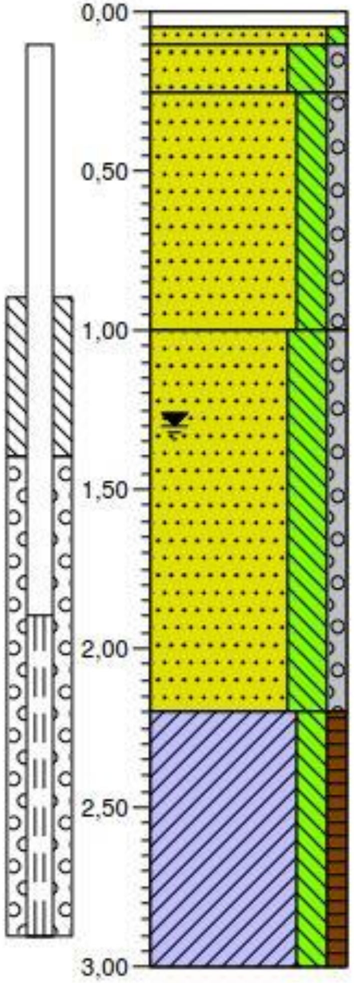
-1,60 Zand, matig grof, zwak siltig, geen olie-water reactie, neutraalgrijs

-2,00 Klei, sterk siltig, zwak humeus, laagjes veen, geen olie-water reactie, neutraal grijsbruin

-2,40

Meetpunt: 09

Datum: 12-05-2016
X: 108372,98
Y: 447356,30



0,00 tegel

-0,05

-0,10 Zand, matig fijn, zwak siltig, neutraal geelcreme

-0,25 Zand, matig fijn, sterk siltig, zwak grindig, sterk puinhoudend, neutraal bruinrood

Zand, matig fijn, matig siltig, zwak grindig, neutraal geelcreme

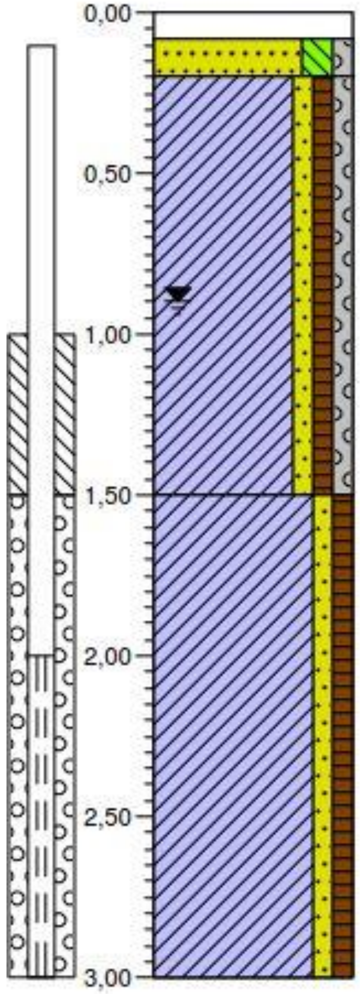
-1,00 Zand, matig fijn, sterk siltig, zwak grindig, brokken puin, neutraal cremegrijs

-2,20 Klei, matig siltig, zwak humeus, neutraalgrijs

-3,00

Meetpunt: 10

Datum: 10-05-2016
X: 108388,18
Y: 447361,34



0,00 klinker

-0,08

-0,20 Zand, matig grof, matig siltig, zwak grindig, neutraal bruincreme

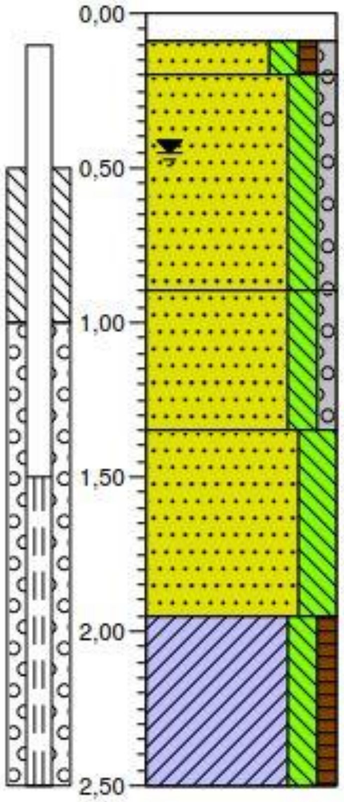
Klei, zwak zandig, zwak humeus, zwak grindig, sporen puin, neutraalbruin

-1,50 Klei, zwak zandig, zwak humeus, sporen puin, neutraal bruingrijs

-3,00

Meetpunt: 11

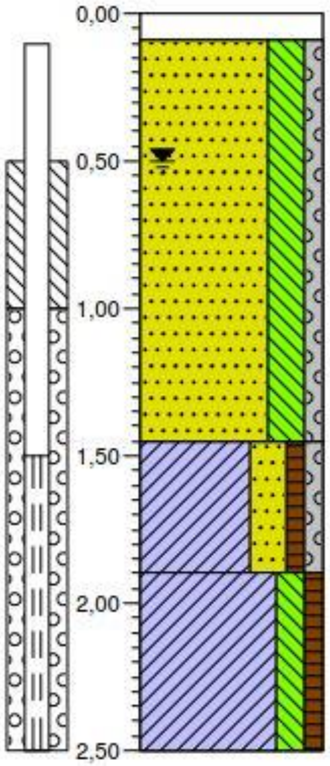
Datum: 10-05-2016
X: 108391,41
Y: 447337,40



0,00 klinker
-0,09
-0,20 Zand, matig fijn, matig siltig, zwak humeus, zwak grindig, neutraal cremebruin
Zand, matig grof, matig siltig, zwak grindig, neutraalgrijs
-0,90
Zand, matig grof, matig siltig, zwak grindig, sterk puinhoudend, neutraalgrijs
-1,35
Zand, matig fijn, sterk siltig, neutraalbruin
-1,95
Klei, matig siltig, zwak humeus, sporen puin, neutraalgrijs
-2,50

Meetpunt: 12

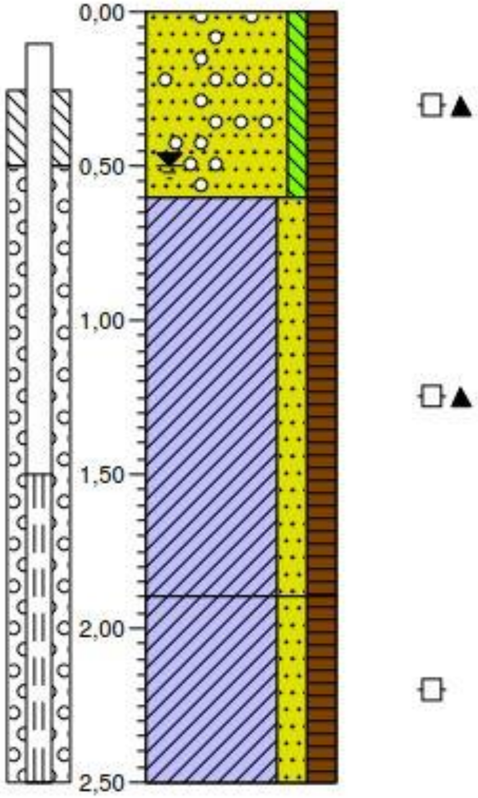
Datum: 10-05-2016
X: 108402,82
Y: 447321,00



0,00 klinker
-0,09
Zand, matig fijn, sterk siltig, zwak grindig, sterk puinhoudend, neutraal grijscreme
-1,45
Klei, sterk zandig, zwak humeus, zwak grindig, sterk puinhoudend, neutraal grijsbruin
-1,90
Klei, matig siltig, zwak humeus, neutraalgrijs
-2,50

Meetpunt: 13

Datum:	17-05-2016
X:	108409,56
Y:	447382,30



0,00 tuin

Zand, zeer grof, zwak siltig, matig humeus, zwak schelphoudend, zwak grindhoudend, geen olie-water reactie

-0,60

Klei, matig zandig, matig humeus, matig zandhoudend, sterk puinhoudend, zwak houthoudend, geen olie-water reactie, neutraalbruin

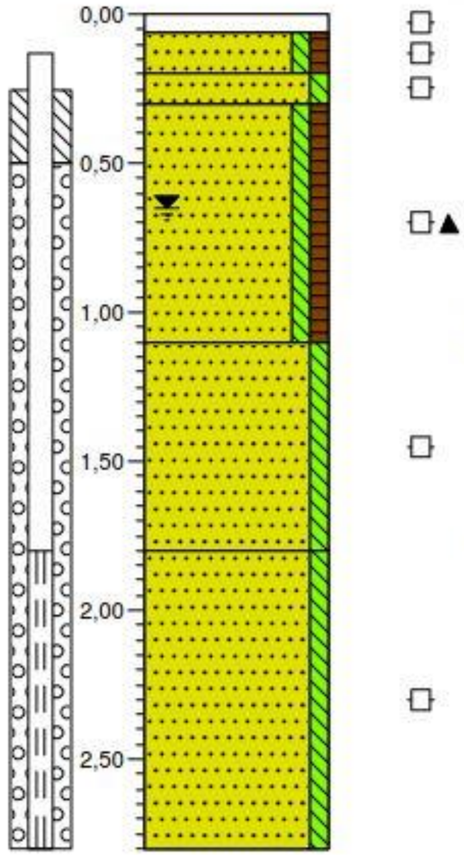
-1,90

Klei, matig zandig, matig humeus, geen olie-water reactie, donkerbruin

-2,50

Meetpunt: 14

Datum:	17-05-2016
X:	108336,54
Y:	447505,00



0,00 klinker

-0,06 Geen olie-water reactie

-0,20 Zand, zeer grof, zwak siltig, zwak humeus, geen olie-water reactie, licht bruingrijs

-0,30 Zand, uiterst grof, zwak siltig, geen olie-water reactie

Zand, zeer grof, zwak siltig, zwak humeus, brokken puin, geen olie-water reactie, donkergrijs

-1,10

Zand, matig grof, zwak siltig, geen olie-water reactie, neutraalgrijs

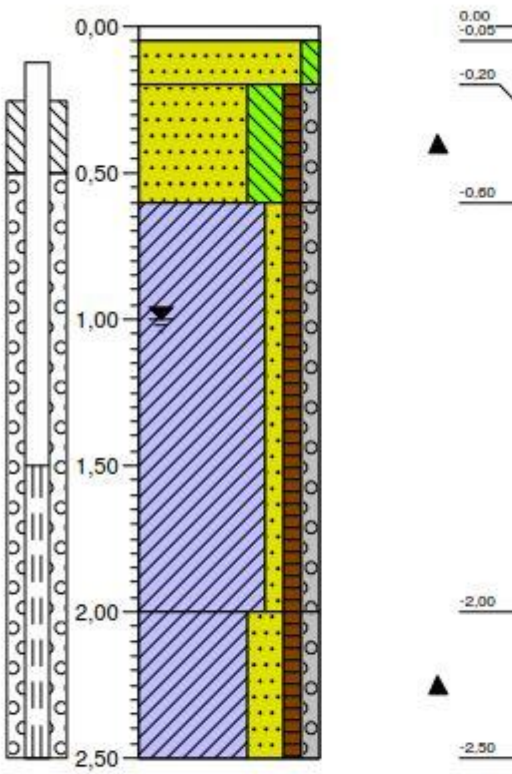
-1,80

Zand, matig grof, zwak siltig, geen olie-water reactie, lichtgrijs

-2,80

Meetpunt: 15

Datum: 12-05-2016
X: 0,00
Y: 0,00



tegel

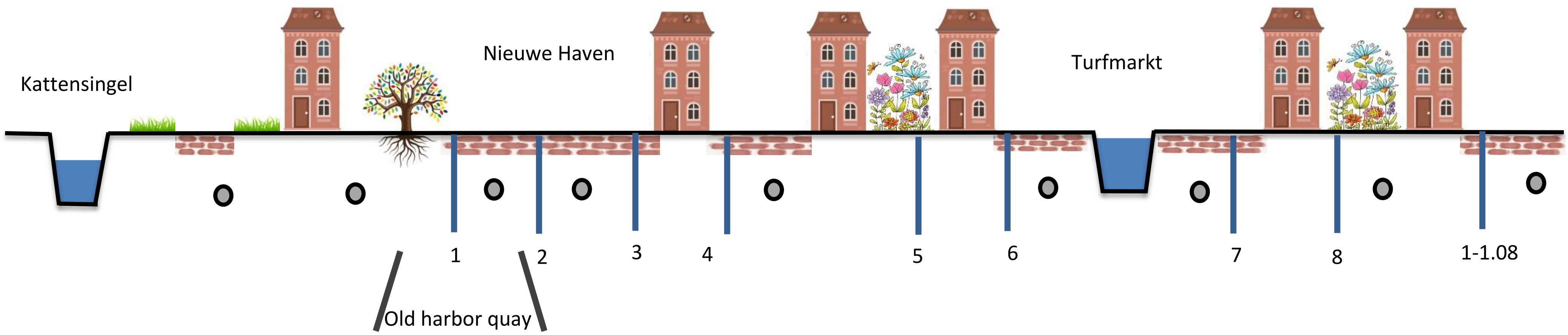
Zand, matig fijn, zwak siltig, neutraal geelcreme

Zand, matig fijn, sterk siltig, zwak humeus, zwak grindig, matig puinhoudend, neutraal cremebruin

Klei, zwak zandig, zwak humeus, zwak grindig, neutraal zwartbruin

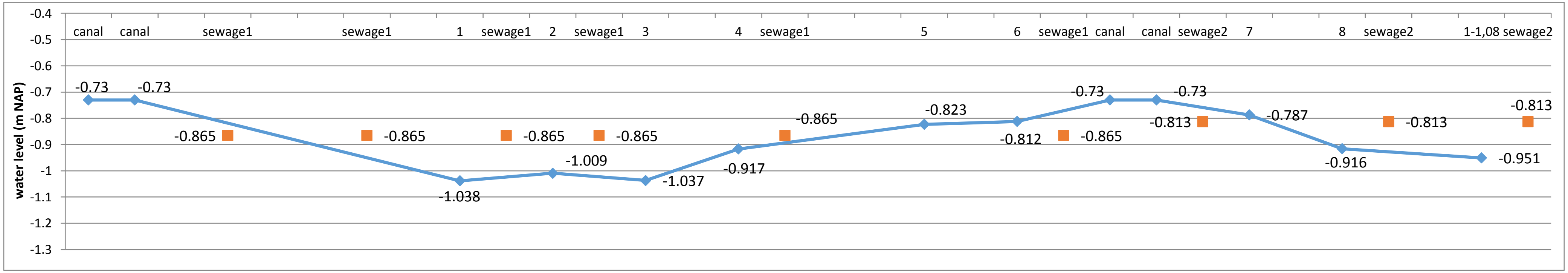
Klei, sterk zandig, zwak humeus, zwak grindig, sterk puinhoudend, neutraal grijsbruin

Appendix 2 Cross section analysis



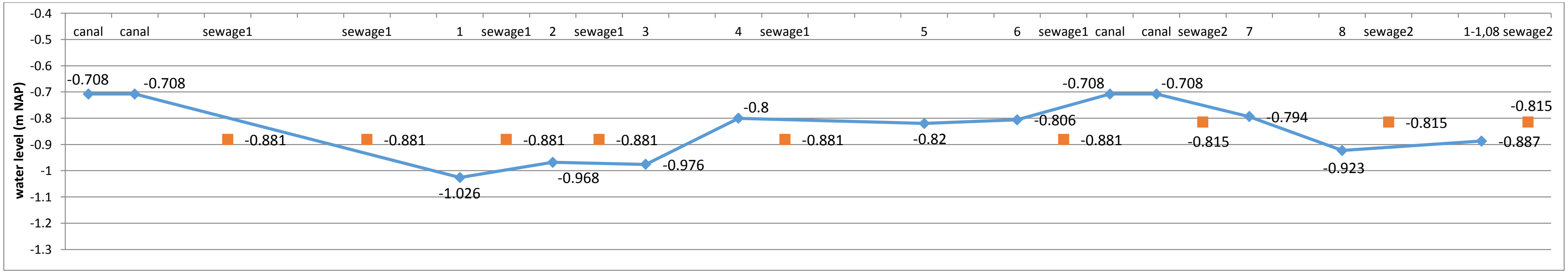
Time: 20-5-2016 15:00 P5 (accumulated precipitation from previous 5 days)=8.3 mm E5 (accumulated evaporation from previous 5 days)= 13.706 mm Net (P5-E5)=-5.406 mm

Note: the moment all wells were installed and started to measure



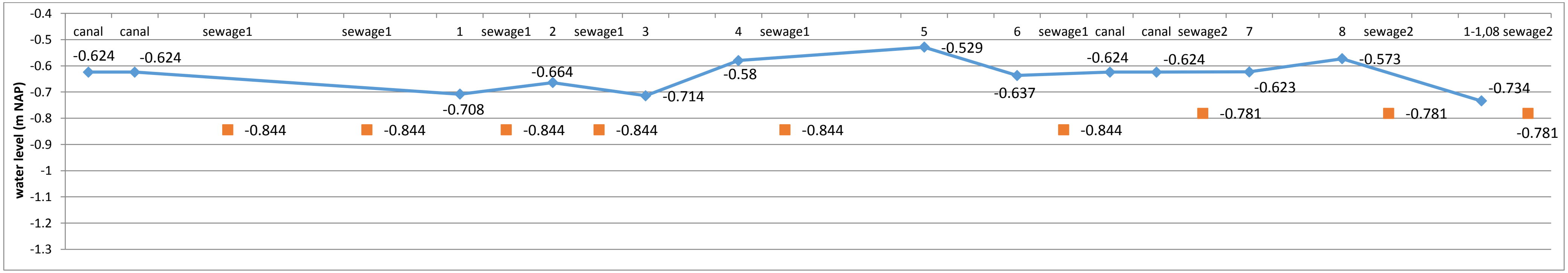
Time: 10-6-2016 22:00 P5=0 E5=27.3 mm Net=-27.3 mm

Note: max daily evaporation (5.2 mm) on 5-6-2016 and 6-6-2016; longest dry period, 5 days from 5-6-2016 to 9-6-2016



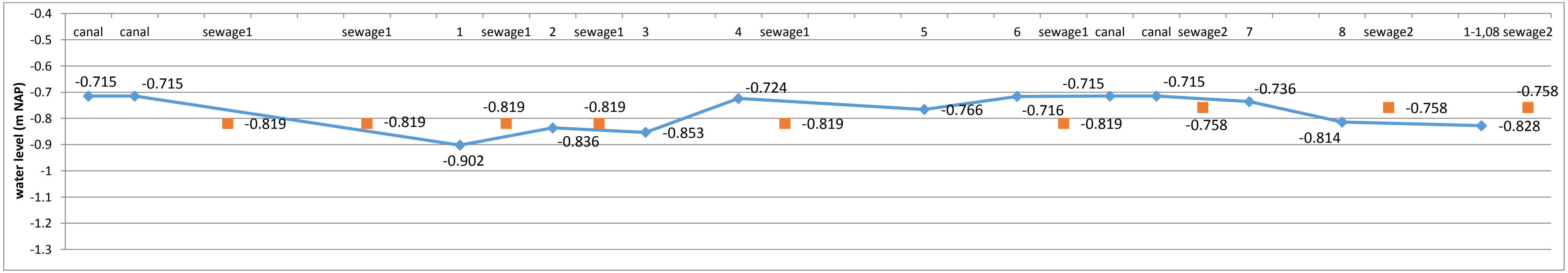
Time: 23-6-2016 7:00 P5=60.806 mm E5=12.869 mm Net=47.938 mm

Note: max daily precipitation (40.696 mm), highest surface water level (-0.621 m NAP) and groundwater level on 23-6-2016; min daily evaporation (0.9 mm) on 20-6-2016



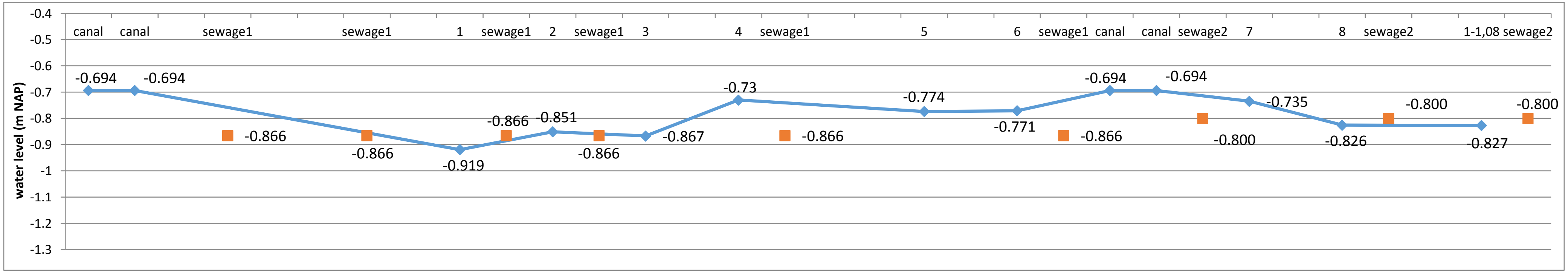
Time: 3-7-2016 9:00 P14=97.729 mm E14=32.705 mm Net=65.024 mm

Note: longest rain period, 14 days from 20-6-2016 to 3-7-2016



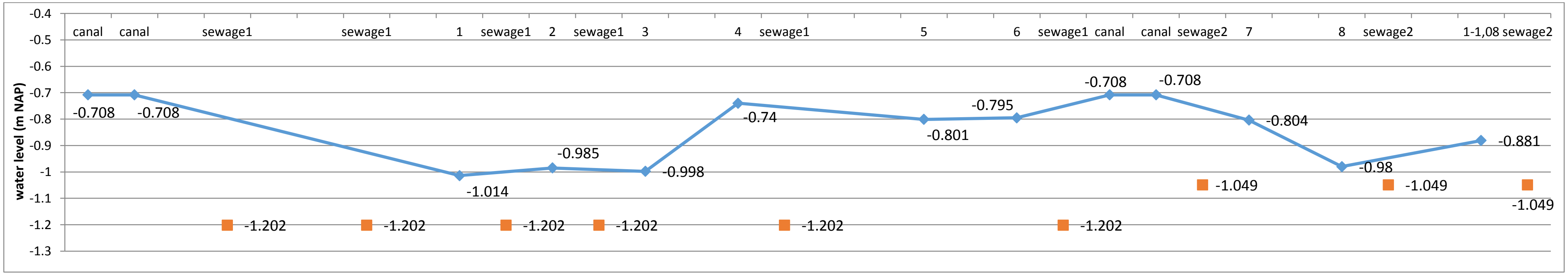
Time: 6-7-2016 7:00 P5=14.807 mm E5=16.319 mm Net=-1.512 mm

Note: 1st field test, before opening weir (at 7:30)



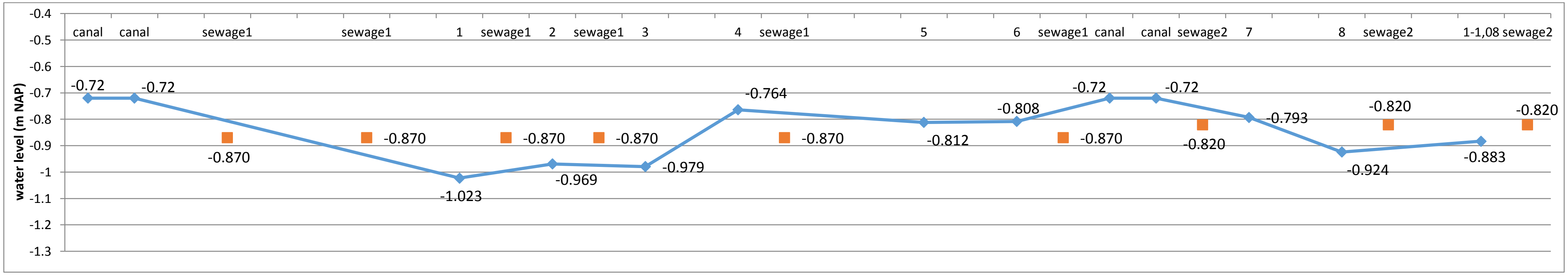
Time: 8-7-2016 8:00 P5=10.565 mm E5=19.215 mm Net=-8.65 mm

Note: 1st field test, after closing weir (at 7:45)



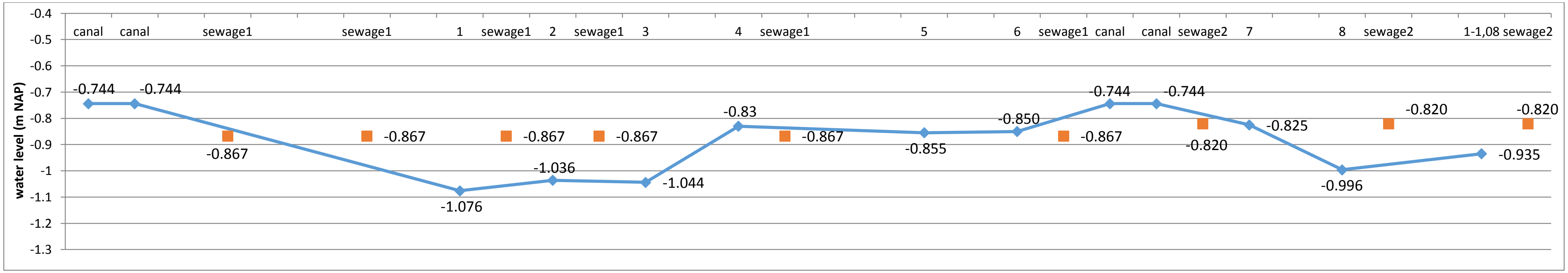
Time: 10-7-2016 8:00 P5=4.775 mm E5=12.298 mm Net=-12.523 mm

Note: 1st field test, 2 days after closing weir



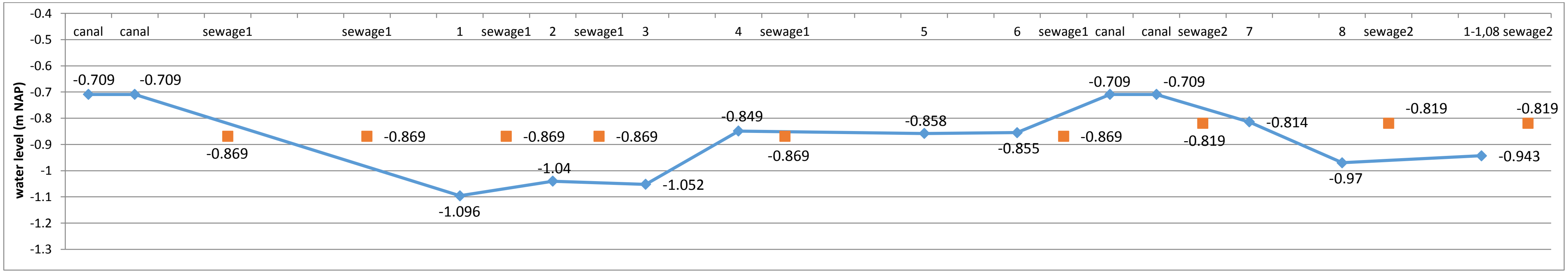
Time: 20-7-2016 16:00 P5=0.855 mm E5=24.459 mm Net=-23.605 mm

Note: lowest surface water level (-0.75 m NAP at 16:20 on 20-7-2016)



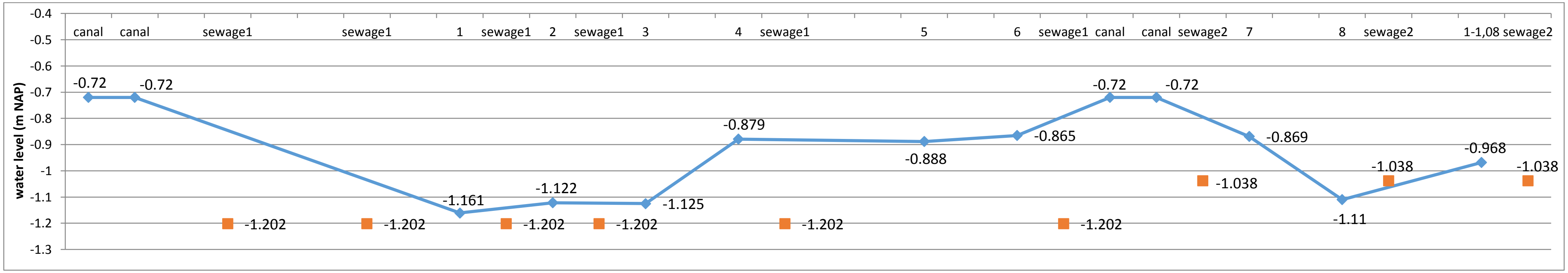
Time: 21-7-2016 7:00 P5=0.855 mm E5=21.713 mm Net=-20.858 mm

Note: 2nd field test, 1 hour before opening weir (at 8:00); max daily evaporation (5.2 mm) on 19-7-2016



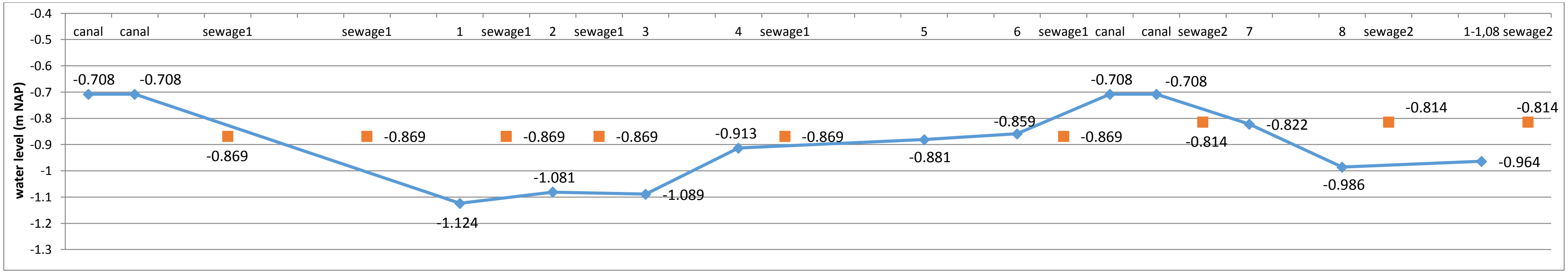
Time: 23-7-2016 12:00 P5=0.855 mm E5=24.505 mm Net=-23.651 mm

Note: 2nd field test, lowest sewage level before closing weir (at 20:00); longest dry period (5 days from 19-7-2016 to 23-7-2016)



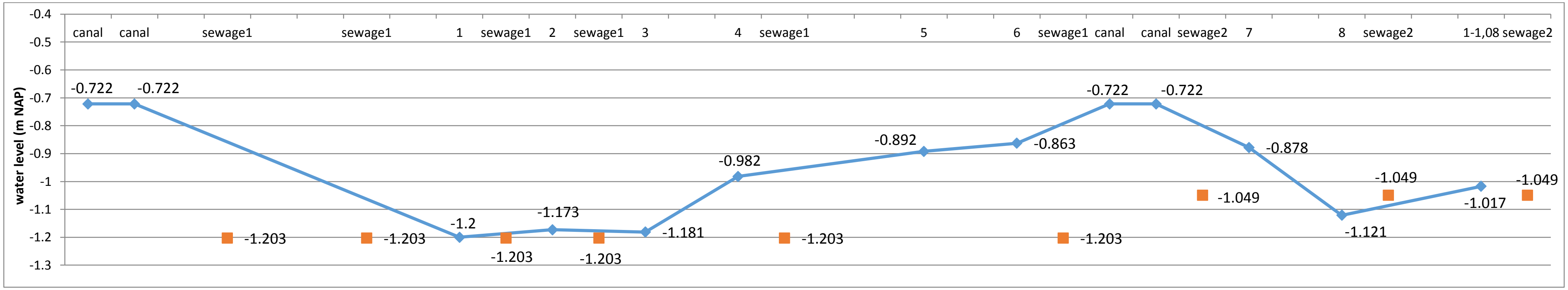
Time: 25-7-2016 20:00 P5=0.005 mm E5=21.2 mm Net=-21.195 mm

Note: 2nd field test, 2 days after closing weir

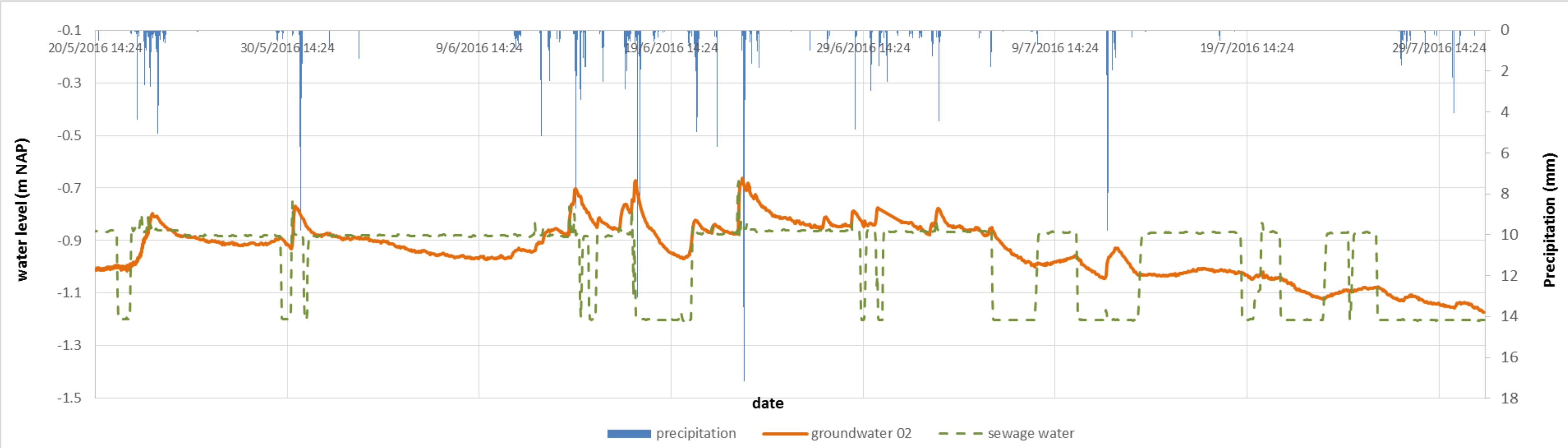


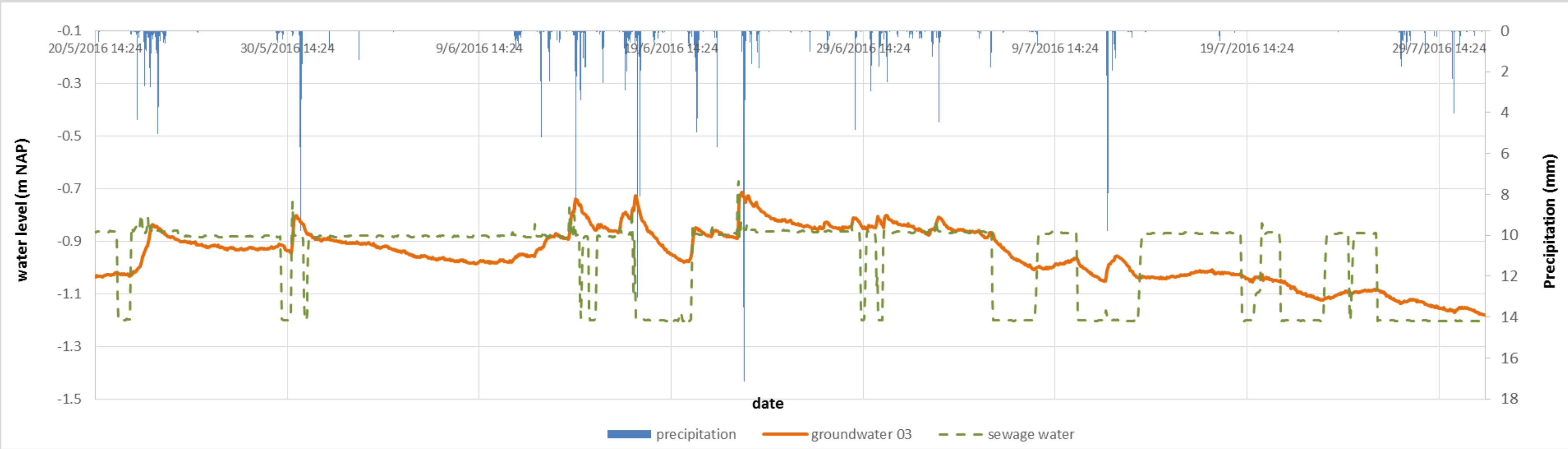
Time: 31-7-2016 23:00   P5=16.568 mm   E5=14.7 mm   Net=1.868 mm

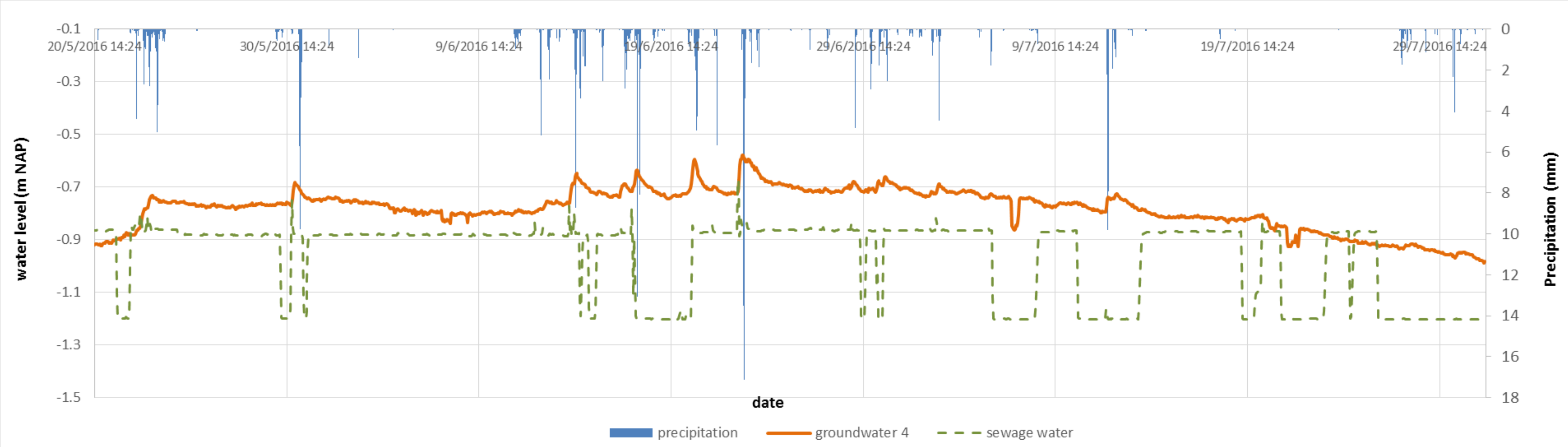
Note: lowest groundwater level



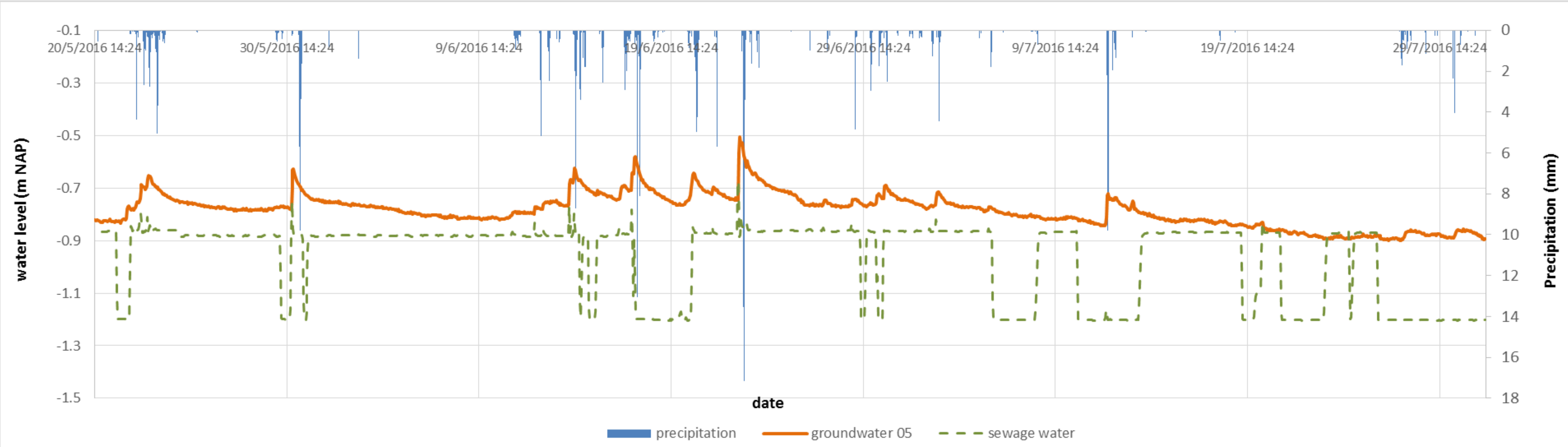
Appendix 3 Hourly time series data of groundwater levels comparing to precipitation and sewage water levels


water level (m NAP)
Precipitation (mm)
date
precipitation
groundwater 02
sewage water
20/5/2016 14:24
30/5/2016 14:24
9/6/2016 14:24
19/6/2016 14:24
29/6/2016 14:24
9/7/2016 14:24
19/7/2016 14:24
29/7/2016 14:24

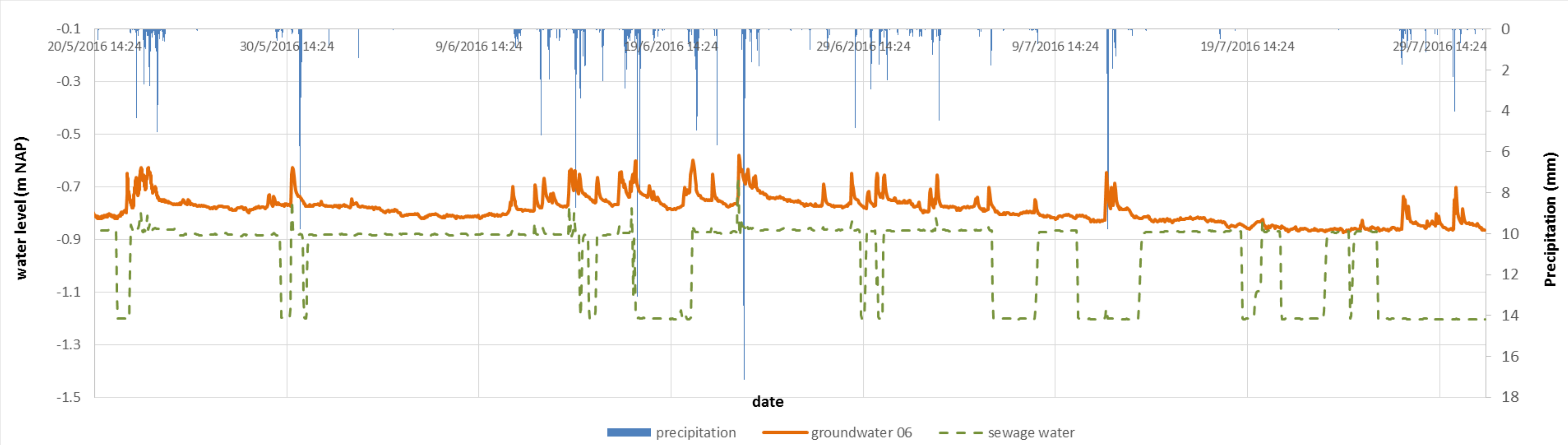

water level (m NAP)
Precipitation (mm)
date
precipitation
groundwater 03
sewage water
20/5/2016 14:24
30/5/2016 14:24
9/6/2016 14:24
19/6/2016 14:24
29/6/2016 14:24
9/7/2016 14:24
19/7/2016 14:24
29/7/2016 14:24

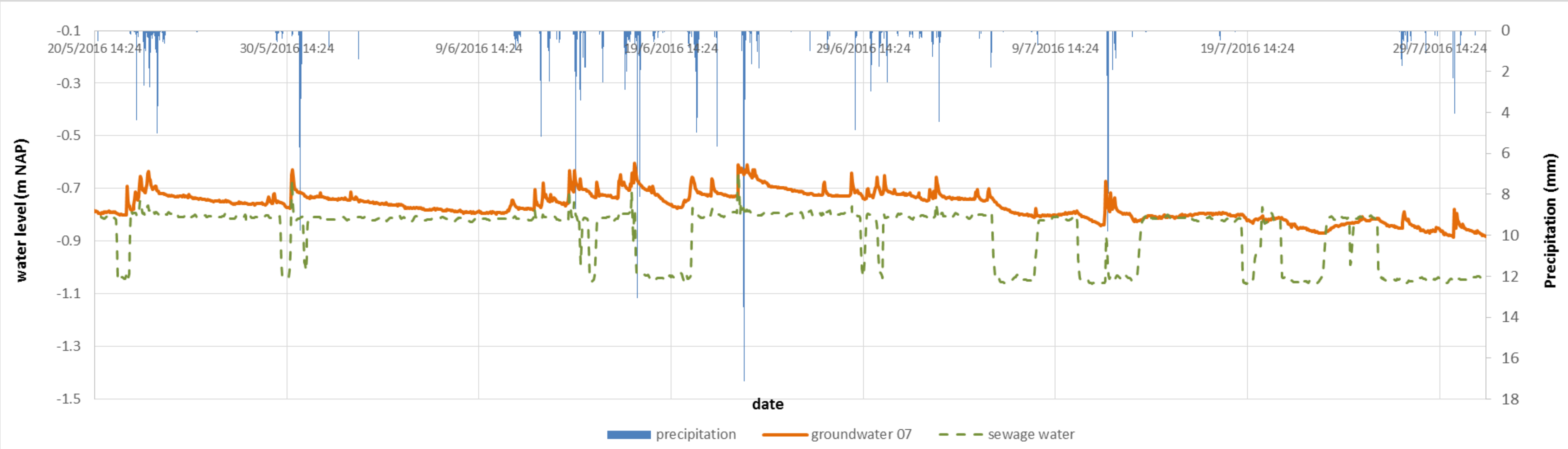


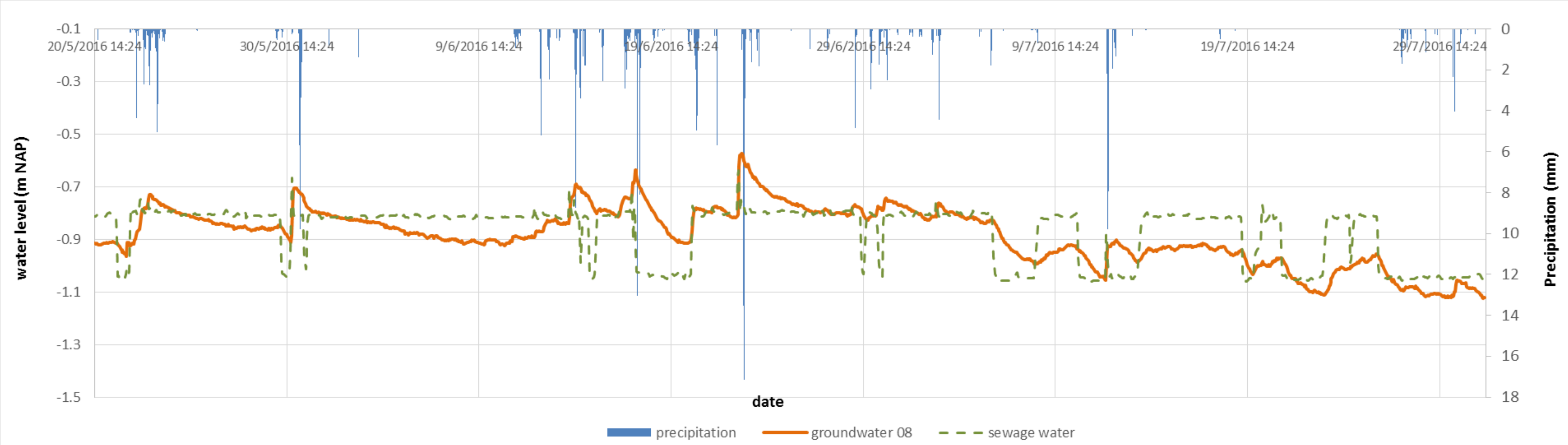
water level (m NAP)
Precipitation (mm)
date
20/5/2016 14:24
30/5/2016 14:24
9/6/2016 14:24
19/6/2016 14:24
29/6/2016 14:24
9/7/2016 14:24
19/7/2016 14:24
29/7/2016 14:24
precipitation
groundwater 4
sewage water



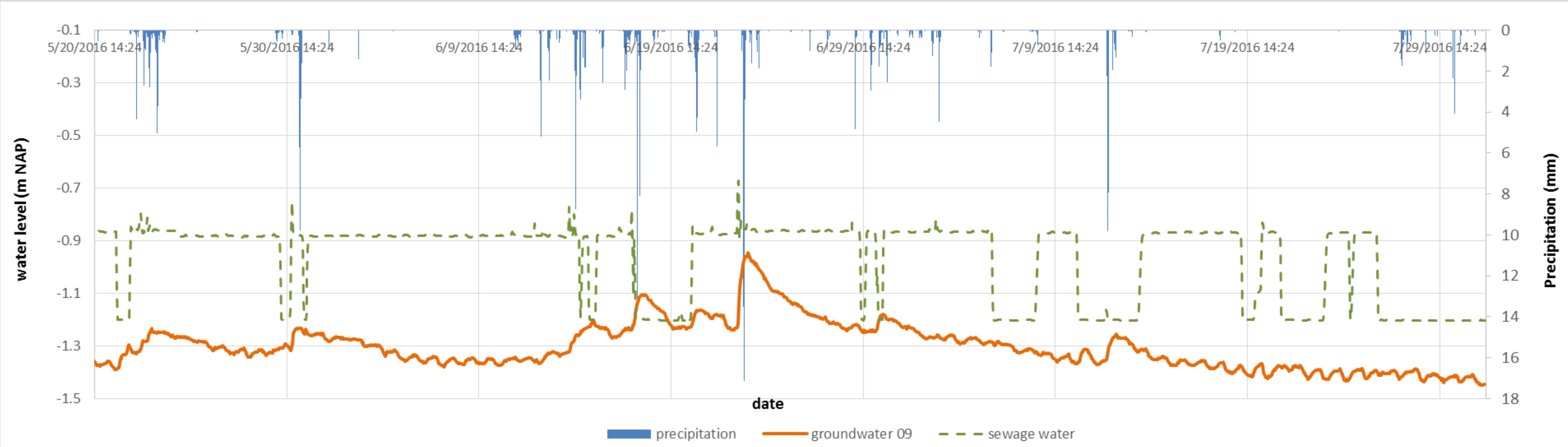
water level (m NAP)
Precipitation (mm)
date
20/5/2016 14:24
30/5/2016 14:24
9/6/2016 14:24
19/6/2016 14:24
29/6/2016 14:24
9/7/2016 14:24
19/7/2016 14:24
29/7/2016 14:24
precipitation
groundwater 05
sewage water


water level (m NAP)
Precipitation (mm)
date
20/5/2016 14:24
30/5/2016 14:24
9/6/2016 14:24
19/6/2016 14:24
29/6/2016 14:24
9/7/2016 14:24
19/7/2016 14:24
29/7/2016 14:24
precipitation
groundwater 06
sewage water

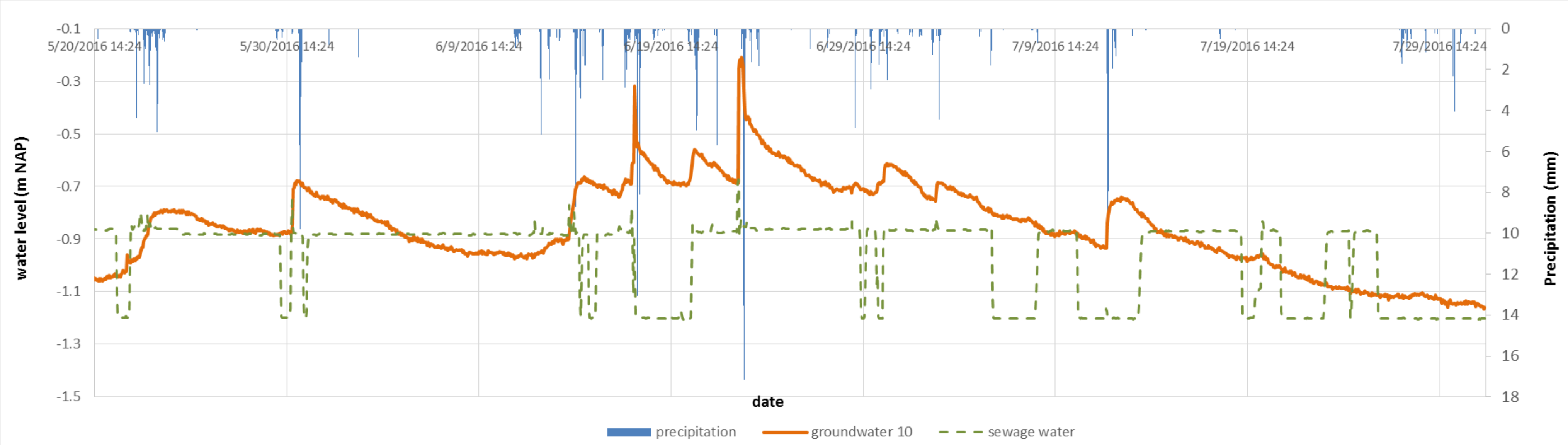

water level (m NAP)
Precipitation (mm)
date
20/5/2016 14:24
30/5/2016 14:24
9/6/2016 14:24
19/6/2016 14:24
29/6/2016 14:24
9/7/2016 14:24
19/7/2016 14:24
29/7/2016 14:24
precipitation
groundwater 07
sewage water



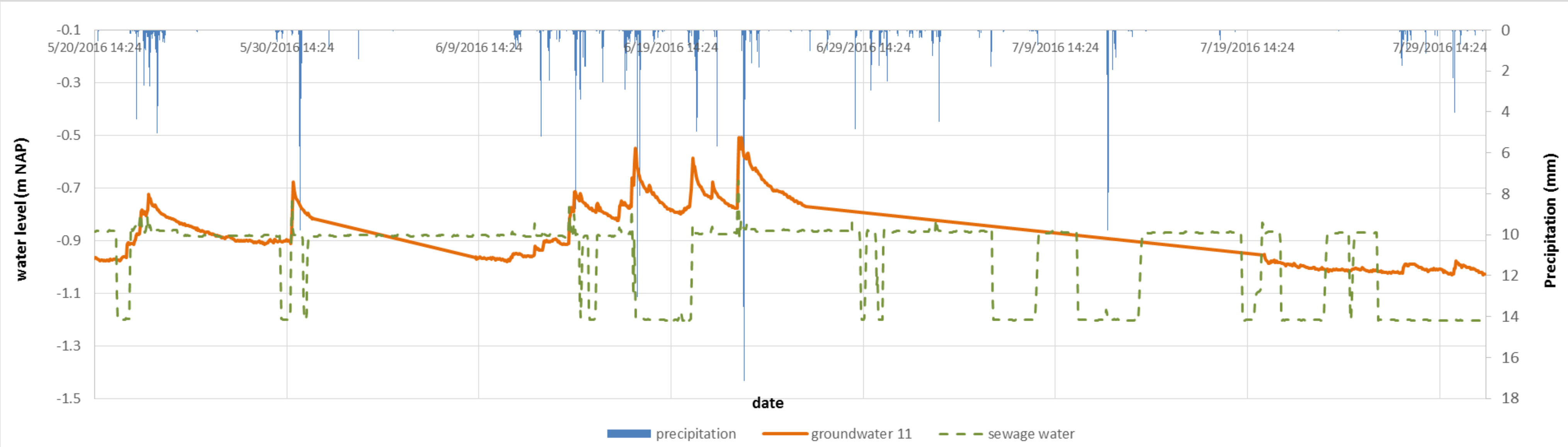
water level (m NAP)
Precipitation (mm)
date
20/5/2016 14:24
30/5/2016 14:24
9/6/2016 14:24
19/6/2016 14:24
29/6/2016 14:24
9/7/2016 14:24
19/7/2016 14:24
29/7/2016 14:24
-0.1
-0.3
-0.5
-0.7
-0.9
-1.1
-1.3
-1.5
0
2
4
6
8
10
12
14
16
18
precipitation
groundwater 08
sewage water



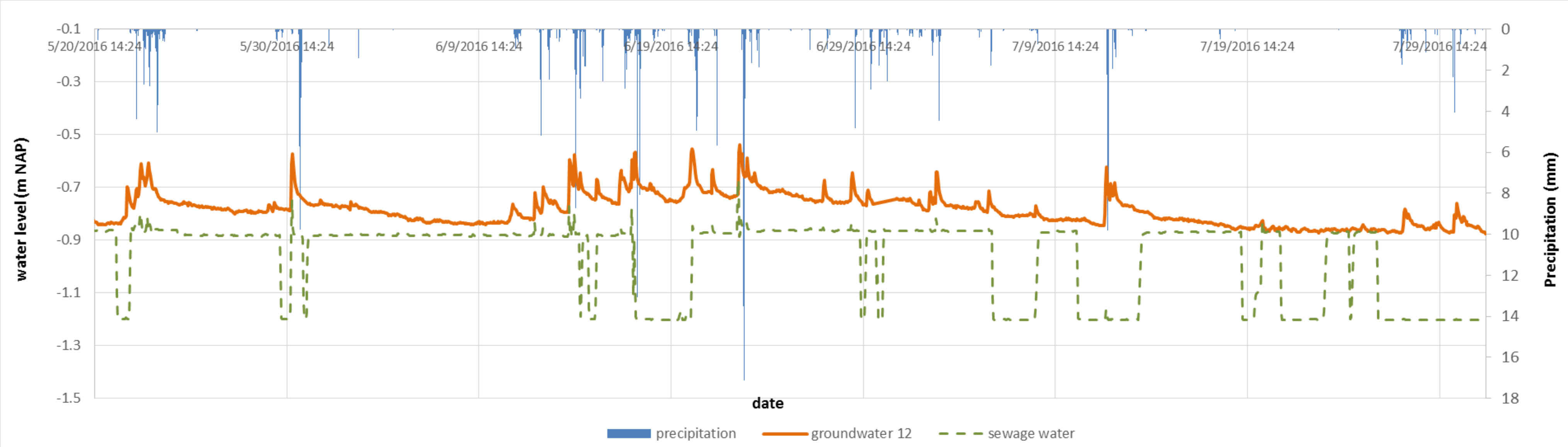
water level (m NAP)
Precipitation (mm)
date
5/20/2016 14:24
5/30/2016 14:24
6/9/2016 14:24
6/19/2016 14:24
6/29/2016 14:24
7/9/2016 14:24
7/19/2016 14:24
7/29/2016 14:24
-0.1
-0.3
-0.5
-0.7
-0.9
-1.1
-1.3
-1.5
0
2
4
6
8
10
12
14
16
18
precipitation
groundwater 09
sewage water

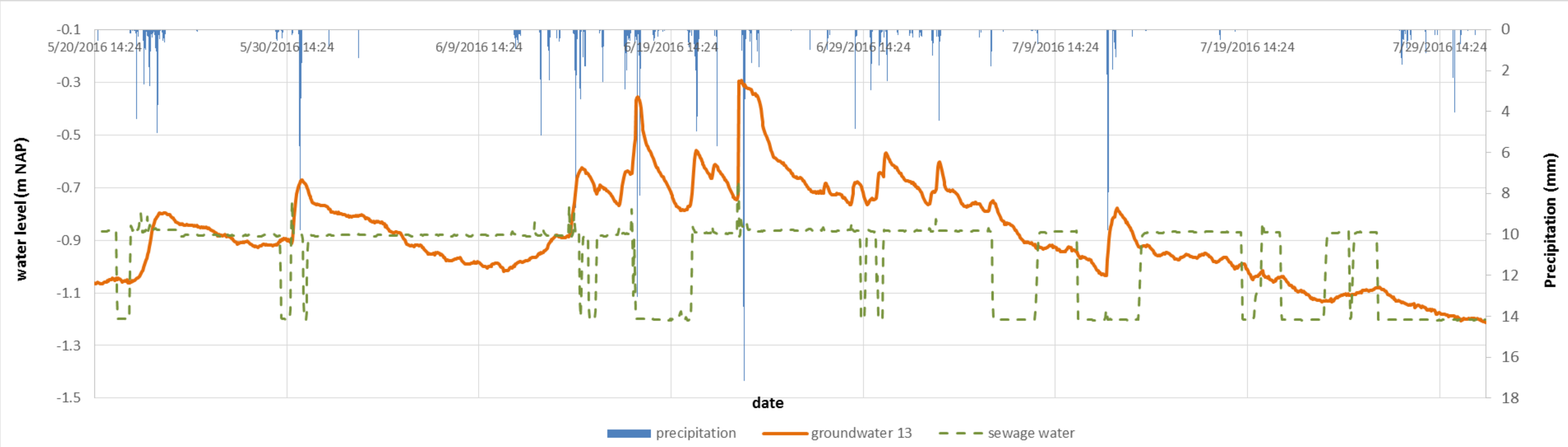


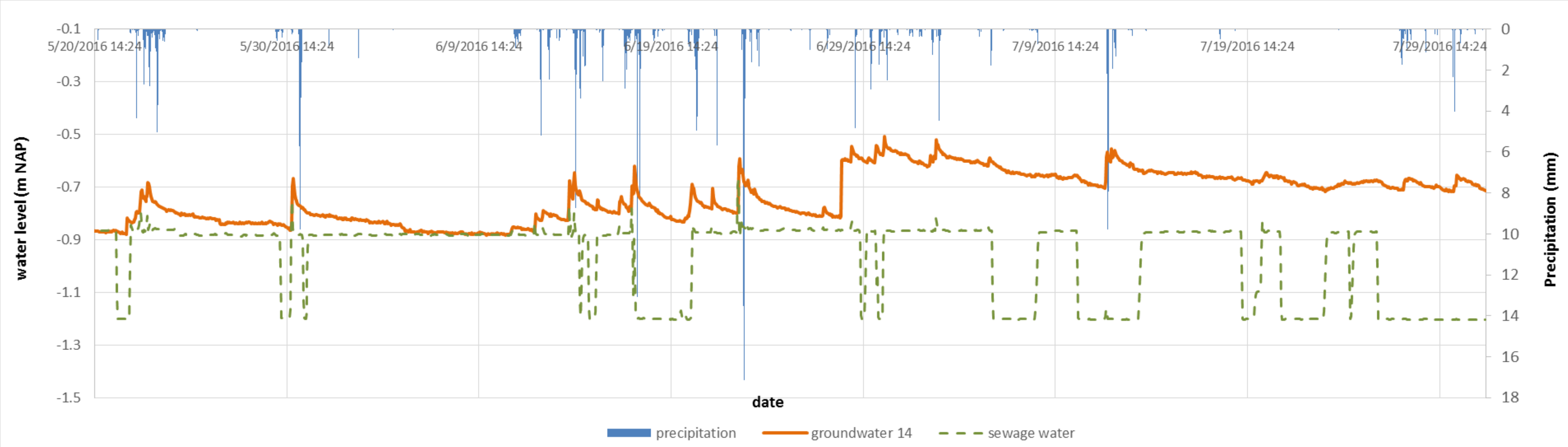
water level (m NAP)
-0.1
-0.3
-0.5
-0.7
-0.9
-1.1
-1.3
-1.5
5/20/2016 14:24
5/30/2016 14:24
6/9/2016 14:24
6/19/2016 14:24
6/29/2016 14:24
7/9/2016 14:24
7/19/2016 14:24
7/29/2016 14:24
Precipitation (mm)
0
2
4
6
8
10
12
14
16
18
date
precipitation
groundwater 10
sewage water

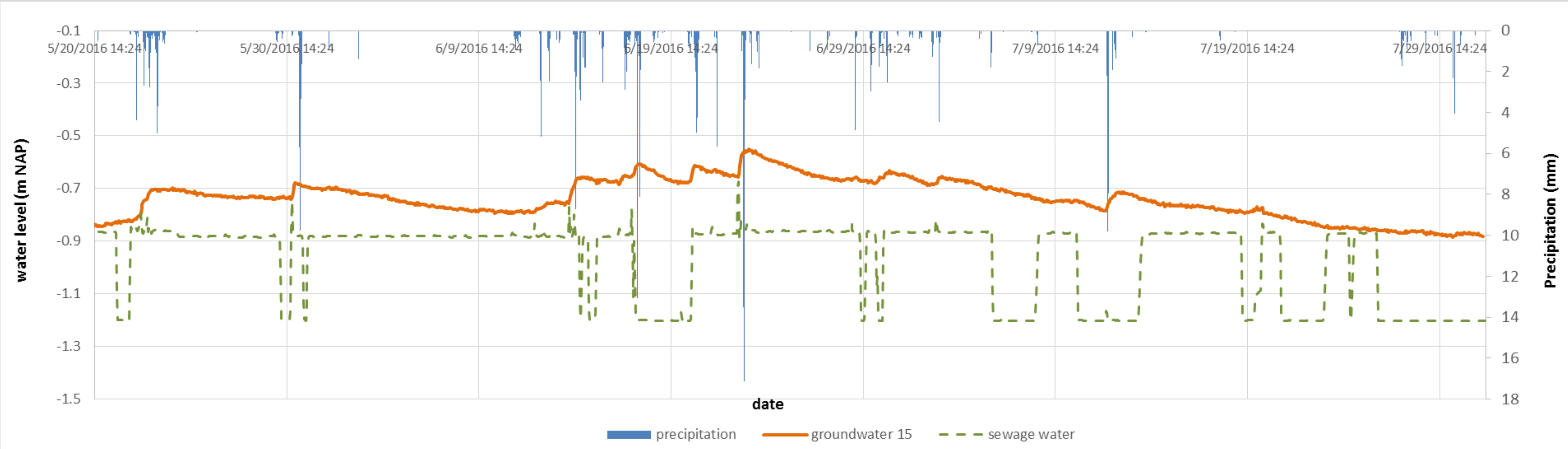


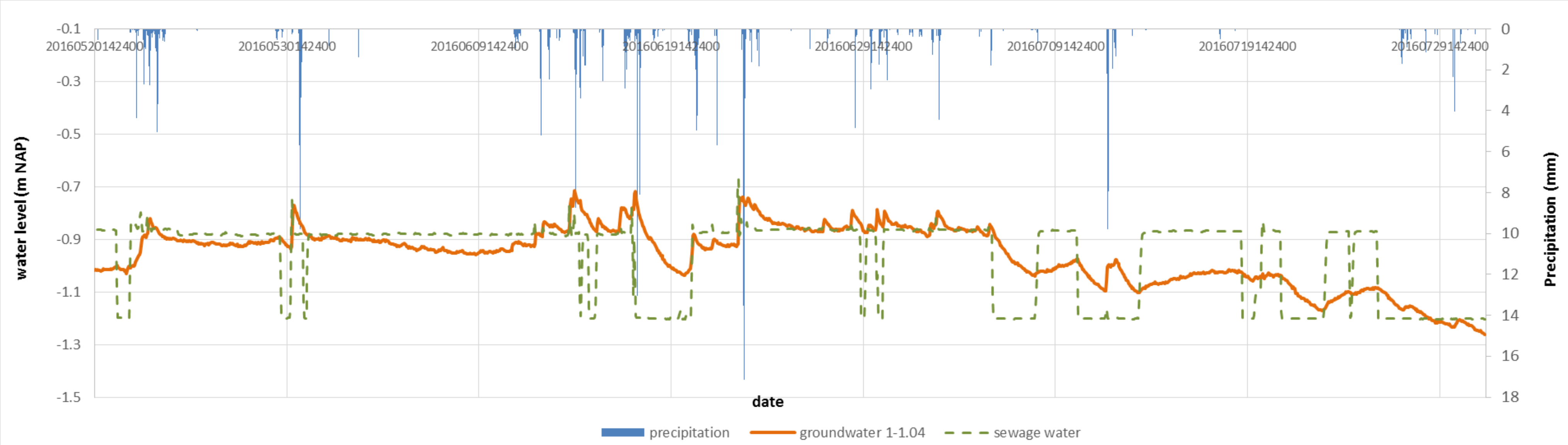
water level (m NAP)
-0.1
-0.3
-0.5
-0.7
-0.9
-1.1
-1.3
-1.5
5/20/2016 14:24
5/30/2016 14:24
6/9/2016 14:24
6/19/2016 14:24
6/29/2016 14:24
7/9/2016 14:24
7/19/2016 14:24
7/29/2016 14:24
Precipitation (mm)
0
2
4
6
8
10
12
14
16
18
date
precipitation
groundwater 11
sewage water


water level (m NAP)
Precipitation (mm)
date
precipitation
groundwater 12
sewage water

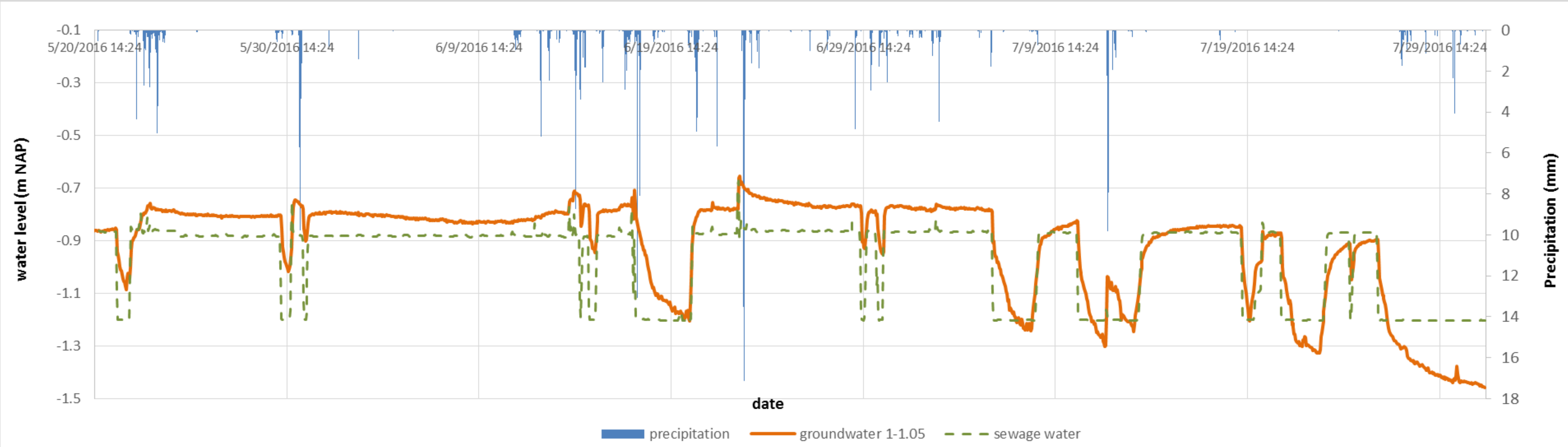

water level (m NAP)
Precipitation (mm)
date
precipitation
groundwater 13
sewage water


water level (m NAP)
Precipitation (mm)
date
precipitation
groundwater 14
sewage water
5/20/2016 14:24
5/30/2016 14:24
6/9/2016 14:24
6/19/2016 14:24
6/29/2016 14:24
7/9/2016 14:24
7/19/2016 14:24
7/29/2016 14:24

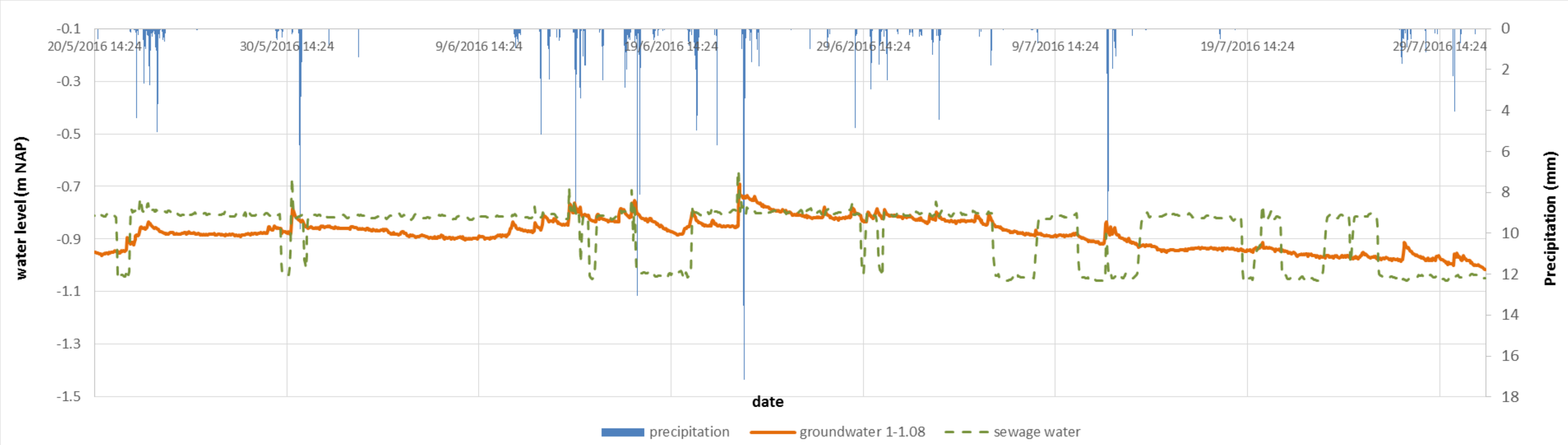

water level (m NAP)
Precipitation (mm)
date
precipitation
groundwater 15
sewage water
5/20/2016 14:24
5/30/2016 14:24
6/9/2016 14:24
6/19/2016 14:24
6/29/2016 14:24
7/9/2016 14:24
7/19/2016 14:24
7/29/2016 14:24



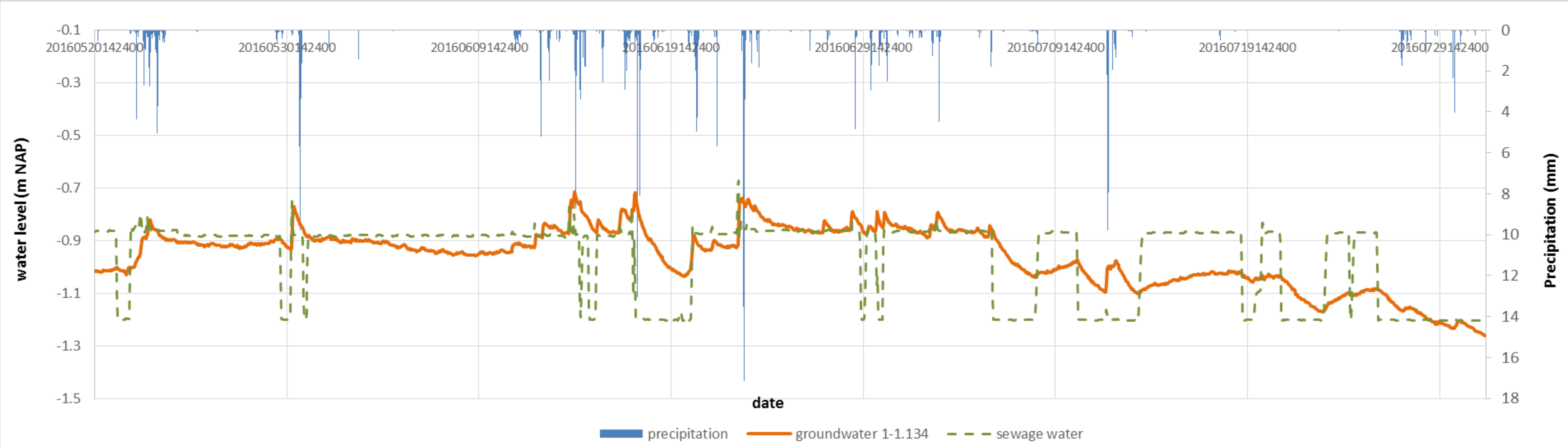
water level (m NAP)
Precipitation (mm)
date
precipitation
groundwater 1-1.04
sewage water



water level (m NAP)
Precipitation (mm)
date
precipitation
groundwater 1-1.05
sewage water



water level (m NAP)
-0.1
-0.3
-0.5
-0.7
-0.9
-1.1
-1.3
-1.5
20/5/2016 14:24
30/5/2016 14:24
9/6/2016 14:24
19/6/2016 14:24
29/6/2016 14:24
9/7/2016 14:24
19/7/2016 14:24
29/7/2016 14:24
Precipitation (mm)
0
2
4
6
8
10
12
14
16
18
date
precipitation
groundwater 1-1.08
sewage water



water level (m NAP)
-0.1
-0.3
-0.5
-0.7
-0.9
-1.1
-1.3
-1.5
20160520142400
20160530142400
20160609142400
20160619142400
20160629142400
20160709142400
20160719142400
20160729142400
Precipitation (mm)
0
2
4
6
8
10
12
14
16
18
date
precipitation
groundwater 1-1.134
sewage water

Appendix 4 Introduction of iMOD interface and iMODFLOW

iMOD (Interactive MODeling) is a user-friendly interface to support groundwater flow models with the combination of a GUI and an accelerated Deltares-version of MODFLOW (IMODFLOW) (*Lambert et al., 2015; Ros, 2008; Vermeulen et al., 2016*). A Modified Incomplete Cholesky Conjugate Gradient method (MICCG) was involved in the modeling code to solve the system equations. In order to minimize the developing cost of individual models overlapping each other and promote the participation of stakeholders, a cooperation between Deltares and other 17 stakeholders was established in 2005 on constructing a numerical groundwater model which comprised the collective regions of interest (*Vermeulen et al., 2013; 2016*). During the model construction procedure, iMOD was developed to provide an internet accessible interface for all the parties to access data, give recommendations, and improve the model based on their experience and understanding of local hydrological system (*Berendrecht et al, 2007; Vermeulen et al., 2013*). By now, iMOD provide a fast and flexible environment for building a large MODFLOW groundwater model with distinctive functionalities such as the user-defined resolution for sub-models, conceptual consistency for the model area, and 2D- and 2D layered-techniques for geo-editing of the subsurface (*Deltares, 2016a; Vermeulen et al., 2016; Warren, 2015*).

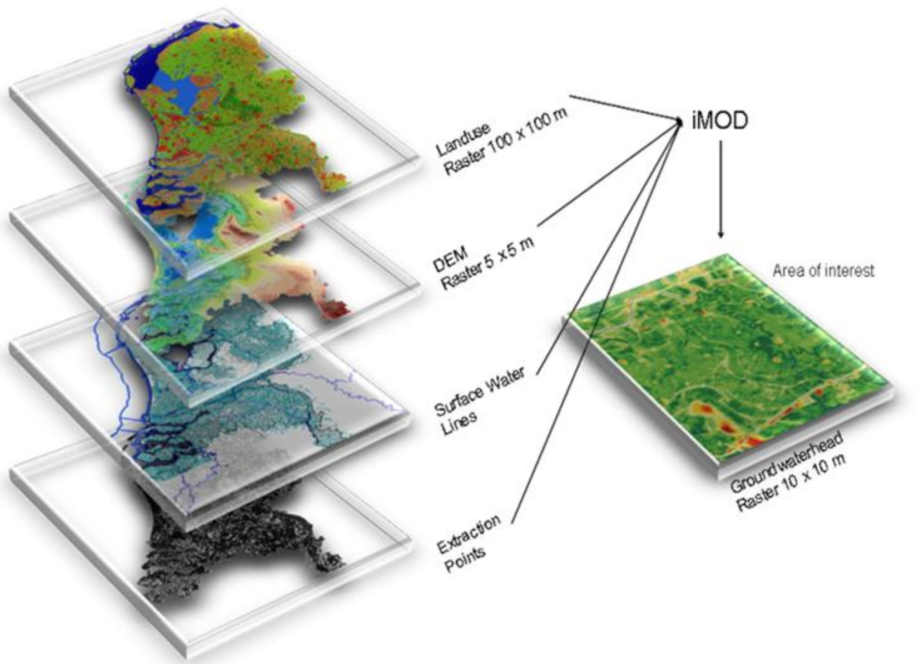


Figure 39 Example of iMOD functionality: one expandable data set covering all possible future areas of interest (Vermeulen et al., 2016)

Traditionally, it is nearly impossible to apply an existing regional MODFLOW model on a local scale with a higher resolution requirement, due to the computational limitation of the

CPU memory. The modelers prefer to have an approach to help with nesting high-resolution local models and switching grid sizes based on need within the whole regional model rather than constructing them everywhere. The developers of iMOD knew this demand and advanced it through generic geo-referenced data structure, which other conventional modeling packages lack of. The generic geo-referenced data structure allows iMOD to gather available spatial data and create sub-domain models with various scales and grid sizes (*Deltares, 2016b; Lambert et al., 2015; Vermeulen et al., 2016*). The input information will be stored at the finest resolution dispensing with any clipping or pre-processing. In terms of the application of up- and down-scaling techniques, it realizes the conceptual consistency between sub-models and administrative boundaries of regional models without additional data sub-sets and incongruous boundary effects (see *Figure 39* and *Figure 40*) (*Deltares, 2016c; Vermeulen et al., 2016*).

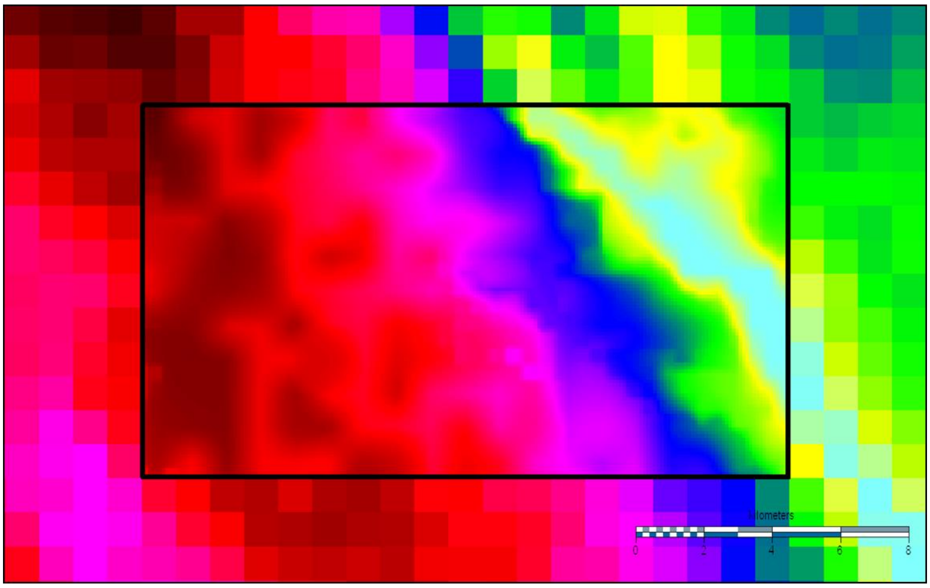


Figure 40 Example of iMOD functionality: consistency between regional and sub-domain models (Deltares, 2016c)

In addition, the up- and down-scaling concepts capacitate modelers to edit and update data within the sub-domain models continuously when new information is obtainable or political agenda changes (*Deltares, 2016c; Warren, 2015*). Because there is a pre-defined mask especially for altering geometry of subsurface data, the data outside of the sub-models will not change along with what happens inside, and it will smoothen the transition edge as well (see *Figure 41*).

Even though there is an error between two cells during the simulation, IMODFLOW embedded in iMOD could solve it by adapting the conductance between them (see *Equation 11* and *Figure 42*). The position of the faults (red line in *Figure 43*) can be altered into the blue line (see *Figure 43*) by using a module called Horizontal Flow Barrier (HFB) (*Ros, 2008; Vermeulen et al., 2016*). HFB can simulate the situation where horizontal obstructions like a badly or impermeable fault zone or a sheet pile wall exist. And it can be separated into Factor fold (without Top of Aquifer (TOP) and Bottom of Aquifer (BOT) packages) and Resistance fold (with TOP and BOT packages).

$$CR_{i,j+1/2,k} = \frac{\frac{TR_{i,j,k}DELC_i}{(1/2)DELR_j} \times \frac{TR_{i,j+1,k}DELC_i}{(1/2)DELR_{j+1}}}{\frac{TR_{i,j,k}DELC_i}{(1/2)DELR_j} + \frac{TR_{i,j+1,k}DELC_i}{(1/2)DELR_{j+1}}}$$

Equation 11

Where:

i, j, k = Index of columns, rows, and layers;

$CR_{i,j+1/2,k}$ = Conductance between cell (i, j, k) and $(i, j+1, k)$ $[L^2T^{-1}]$;

TR = Transmissivity in row direction $[L^2T^{-1}]$;

$DELC$ = Grid width of row $[L]$;

$DELR$ = Grid width of column $[L]$.

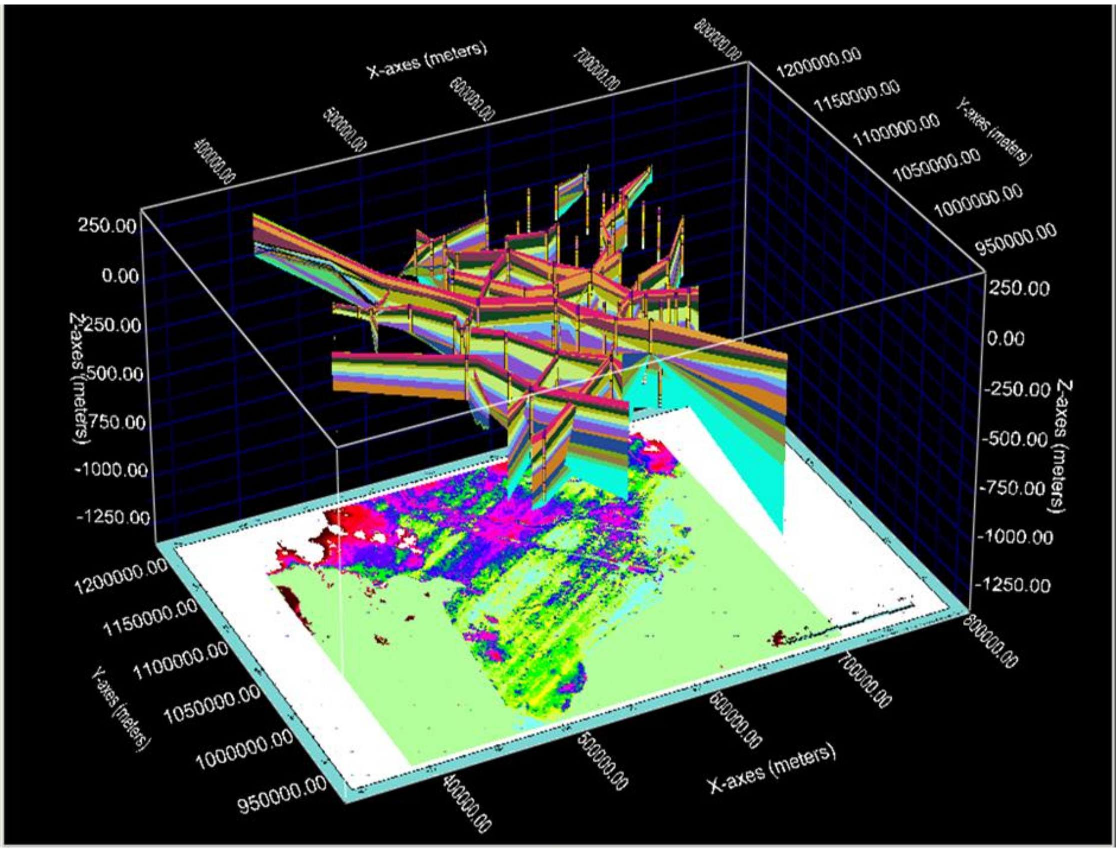


Figure 41 Example of iMOD functionality: interactively editing the geometry of the subsurface

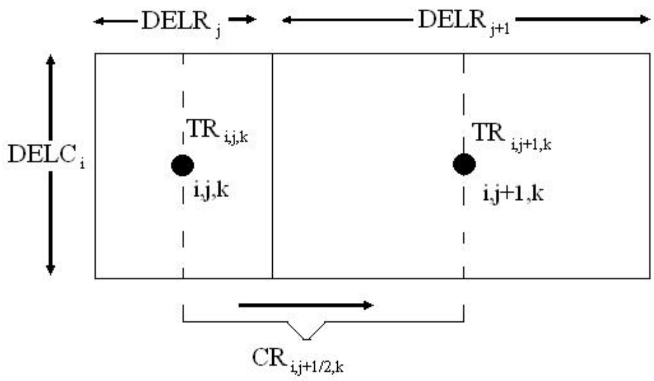


Figure 42 Calculation of conductance between two cells using transmissivities (Ros, 2008)

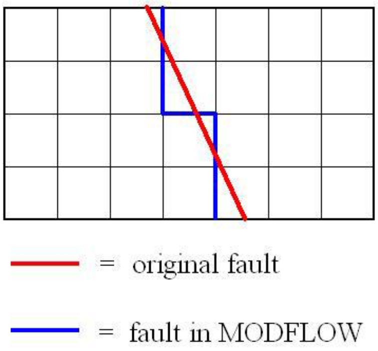


Figure 43 Fault as mdeled by IMODFLOW (Ros, 2008)

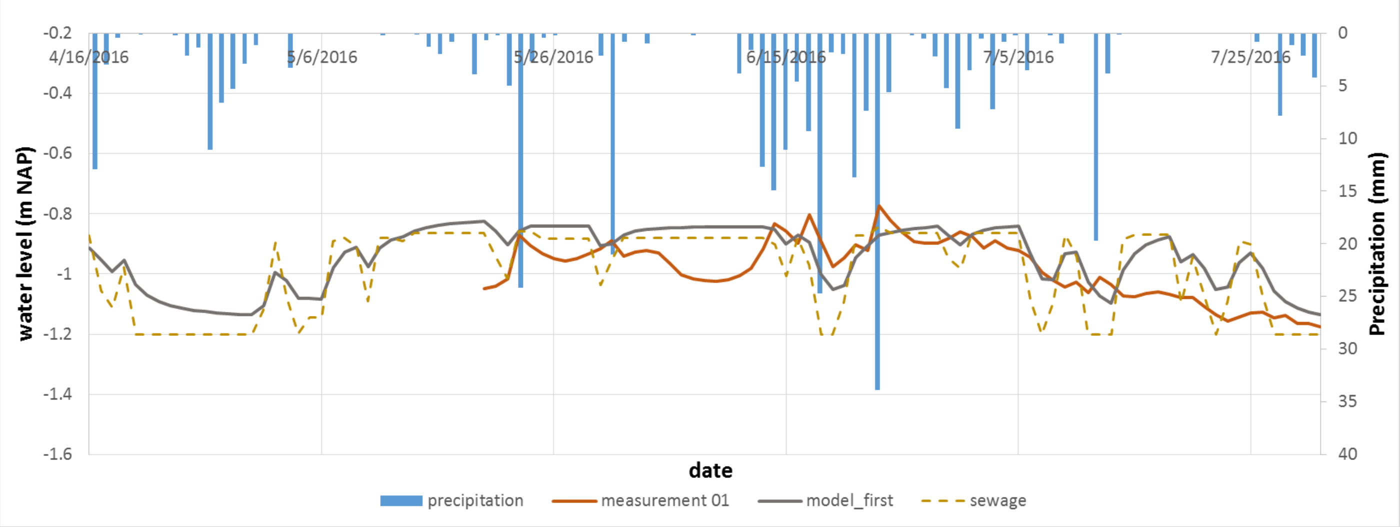
Owing to the limitation of computer hardware, iMOD could run a number of partly overlapping but abutting sub-models instead of a large regional model to reduce computation memory and time. The computation time (T) has an exponentially relationship with number of model cells (n): $T = f(n^{1.5\sim2.0})$. This approach offers a flexible and fast modeling workflow for modelers to construct groundwater flow model according to the actual requirement. At the same time, iMOD would bring forth MODFLOW inputs directly in the computer memory rather than standard production of input files in ASCII format, which may take hours to complete. In this way, the efficiency has been improved considerably, especially in visualization of input and output data during the model construction process. Furthermore, the zoom-extent-dependent visualization techniques embedded in iMOD can facilitate a rapid and comprehensive view of the borehole information, stationary geologic or hydro-stratigraphic results, as well as dynamic model outputs in both 2D and 2D layered (*Deltares, 2016c*).

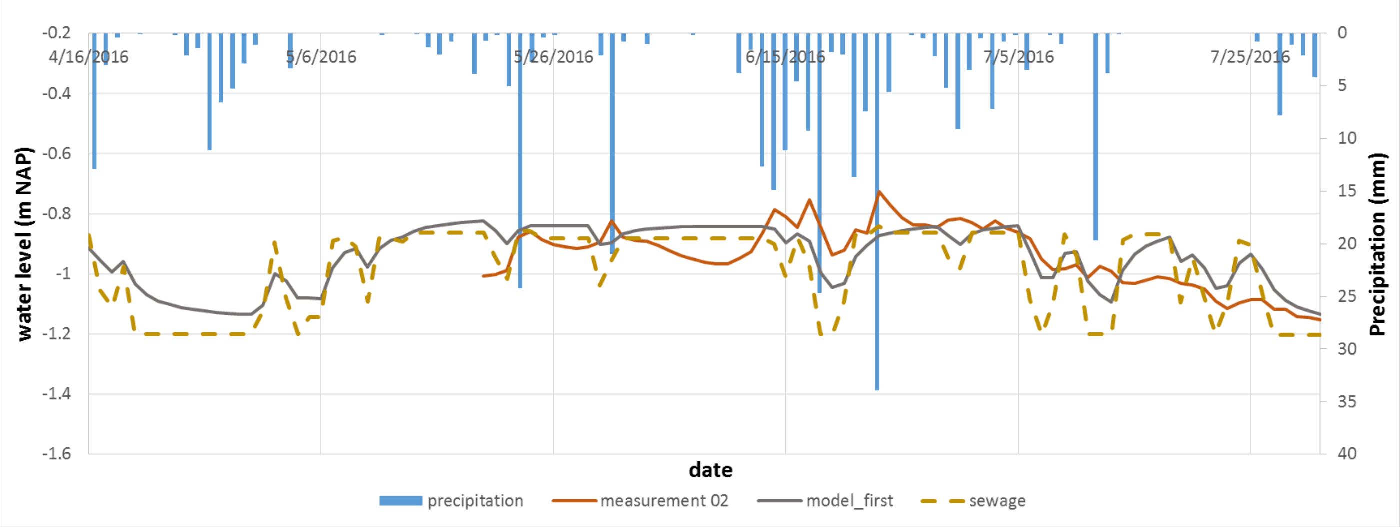
Compared to other conventional groundwater flow models, iMOD establishes a bridge between numerical modeling and decision-making through high performance, reasonable runtimes, flexibility, and transparency (*Vermeulen et al., 2016*). For some groundwater flow model, for example, GMS (Groundwater Modeling System) and Visual MODFLOW, which do not support GUIs, it is difficult for stakeholders and the public to fully understand and trust the model results (*Deltares, 2016b*). Yet, iMOD can display different scenarios and effects of interventions clearly via a quick-scan from the impulse-response database, functioned as a valid decision support tool (*Berendrecht et al., 2007*).

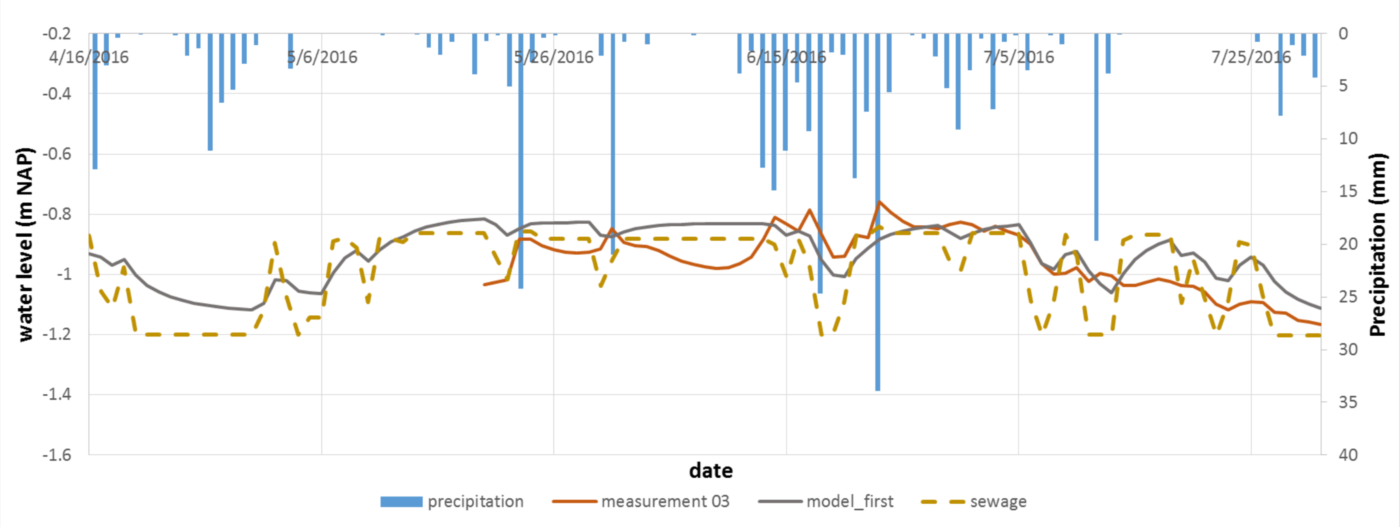
As mentioned above, iMOD was developed through comments and recommendations from stakeholders. The majority of them were concentrated on the improvement of local area issues, including fluxes (seepage, infiltration, and drainage), surface water levels, and depths of drainage systems (*Berendrecht et al., 2007*). To a certain degree, iMOD was considered to be suitable to apply on a local scale, even though it was designed as a regional groundwater flow model originally. *Vermeulen et al. (2013)* constructed a groundwater flow model for Mekong Delta, Vietnam by the application of iMOD. In order to prove the ability of iMOD on local refinements, they zoomed in for Ho Chi Minh City and Can Tho City from a scale of 1,000 m to 50 m and added more detailed local data in these two sub-domain models. Pursuant to sub-models, they analyzed the drawdown in Can Tho City and relationship between groundwater and surface water in Ho Chi Minh City. Since iMOD can be easily in contact with a land subsidence model, *De Roover (2015)* used it to figure out the most influential parameter on model result, in the interest of alleviating land subsidence caused by groundwater overexploitation. The results indicated that horizontal hydraulic conductivity had the greatest impact on the model while vertical hydraulic conductivity had the least. Subsequently, *Lambert et al.* (2015) applied an integrated iMOD-SCR (Settlement Creep) - DAM (Dike Analysis Module) model to investigate the impacts of cessation of groundwater extraction on dike stability in Delft, the Netherlands. The model was able to separate the influence from sea level rise and land subsidence, which provided a more detailed insight into local circumstances and a chance to test the effects of different management decisions.

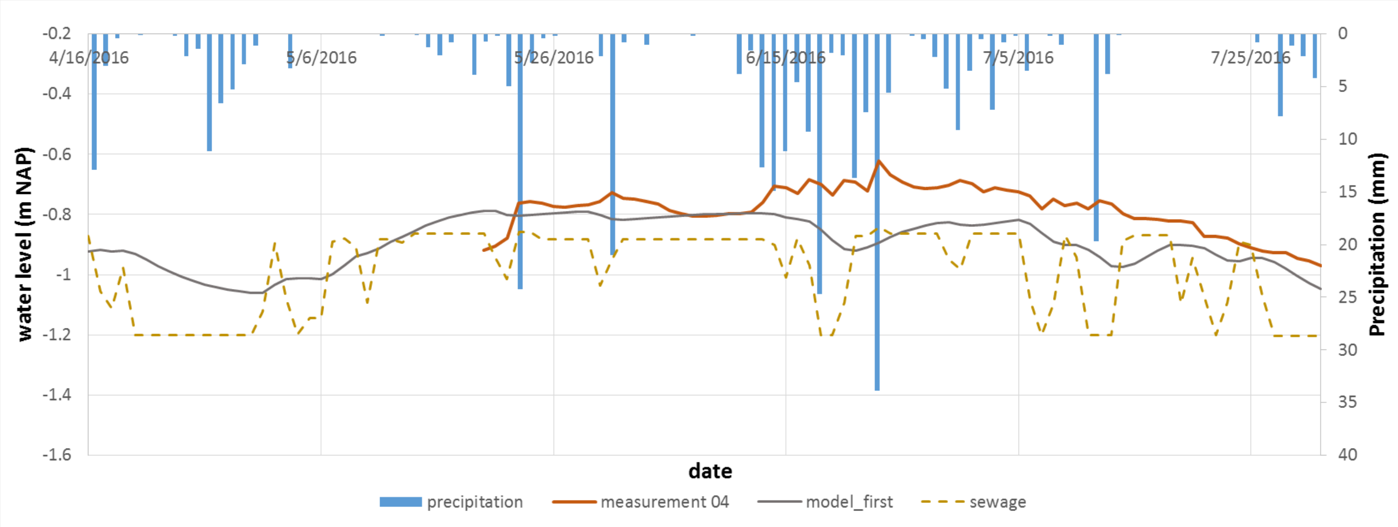
Appendix 5 Model simulation results

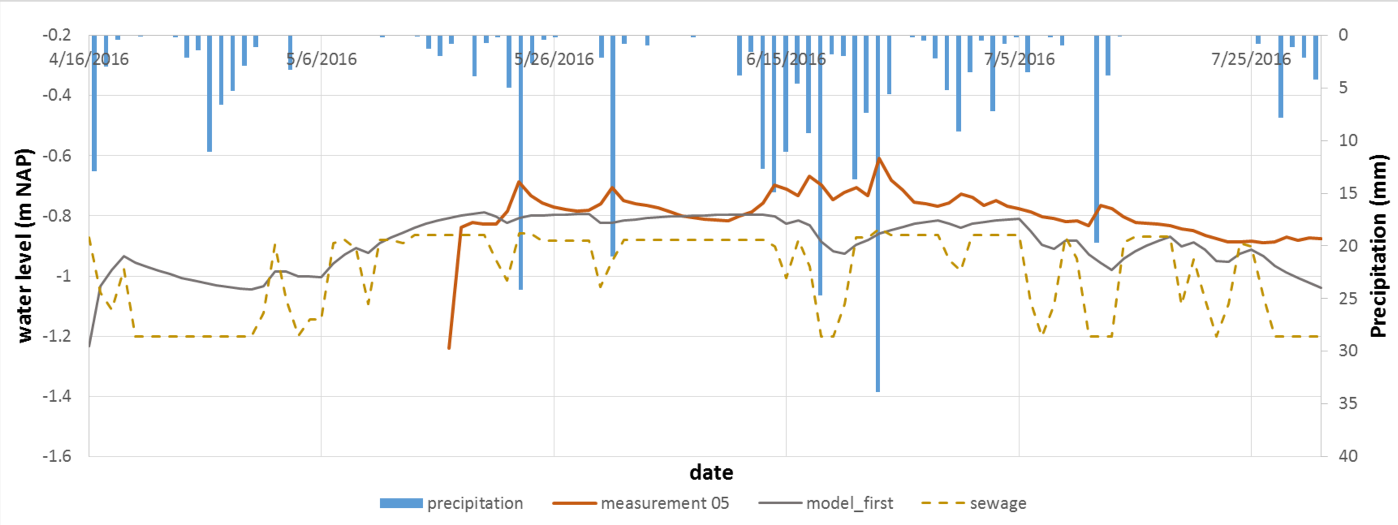
Simulation results of model with MetaSWAP:








water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 04
model_first
sewage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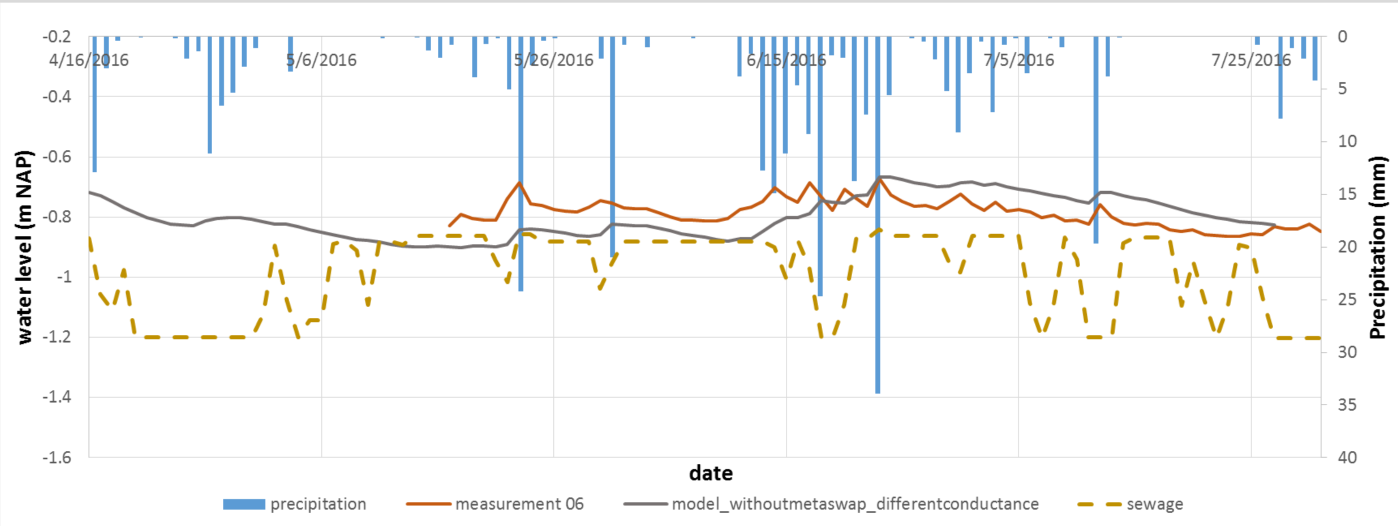
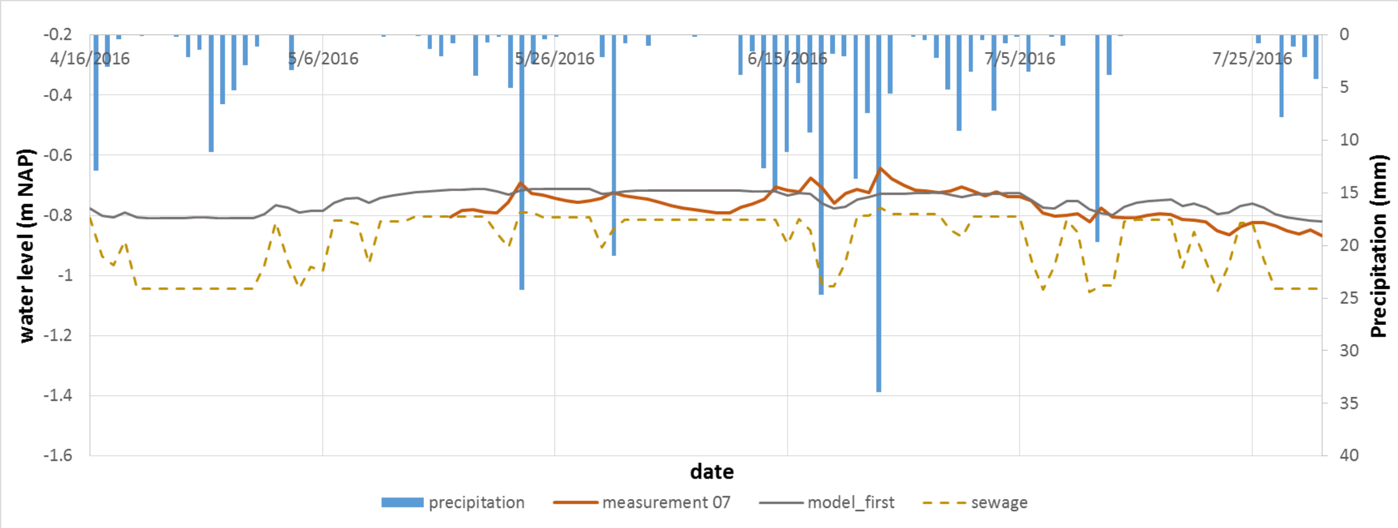
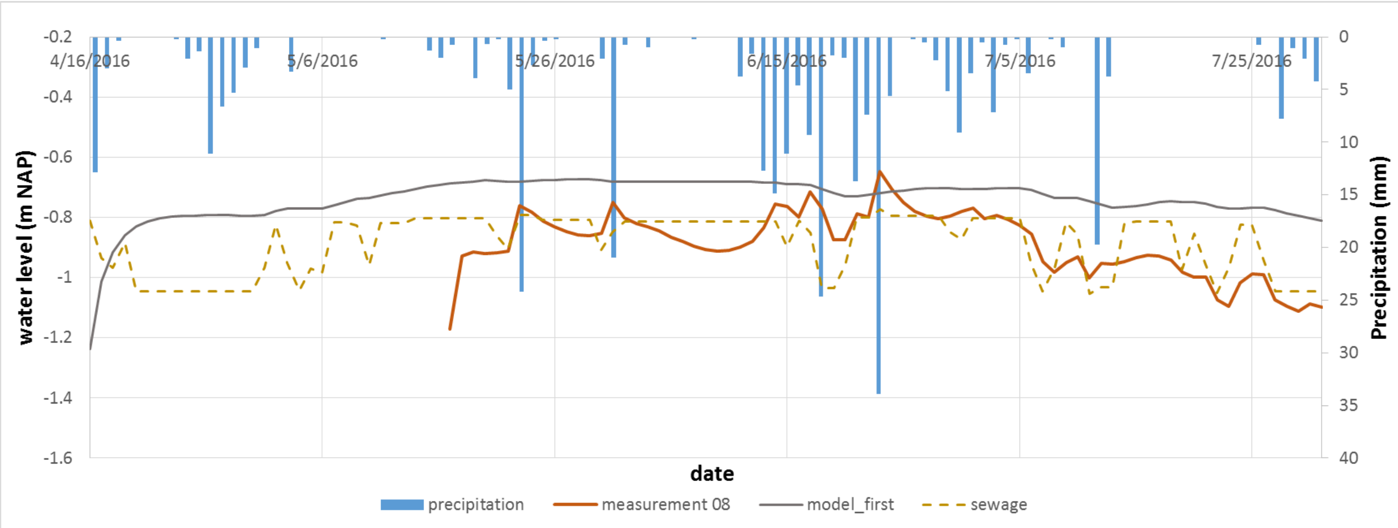
Precipitation (mm)
date
precipitation
measurement 05
model_first
sewage

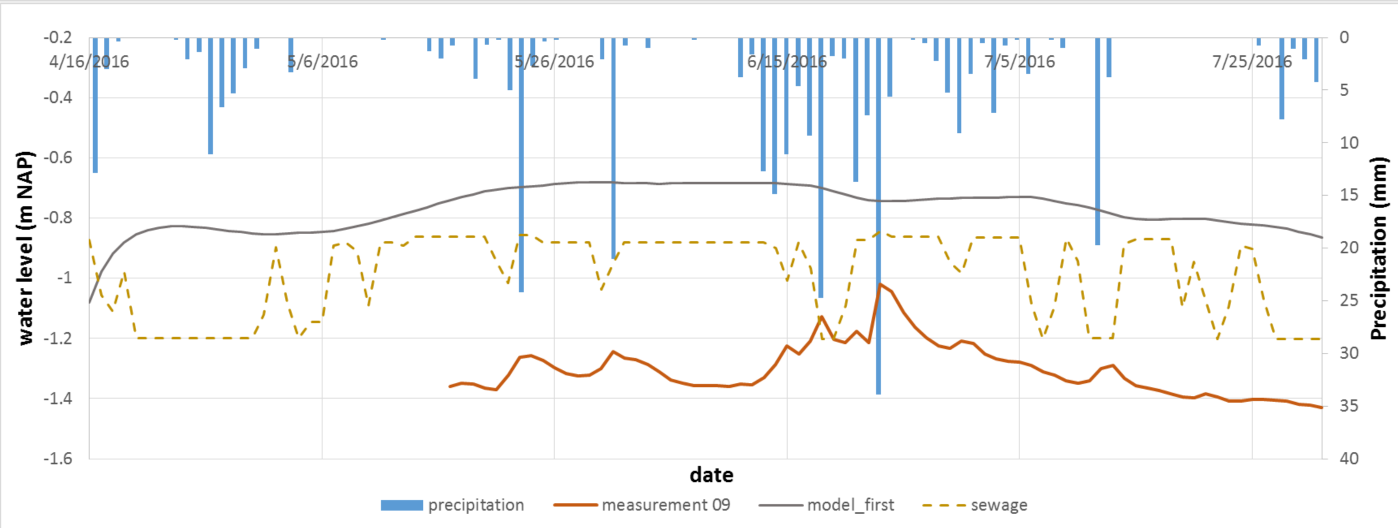

water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 06
model_withoutmetaswap_differentconductance
sewage



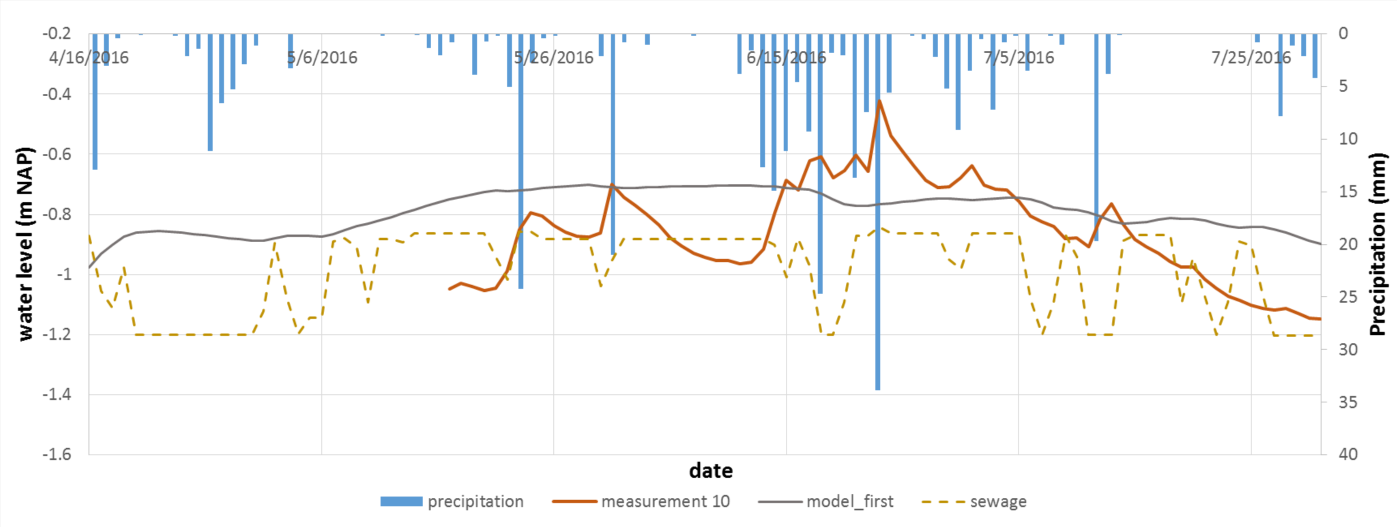
-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
water level (m NAP)
0
5
10
15
20
25
30
35
40
Precipitation (mm)
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
date
precipitation
measurement 07
model_first
sewage



-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
water level (m NAP)
0
5
10
15
20
25
30
35
40
Precipitation (mm)
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
date
precipitation
measurement 08
model_first
sewage



-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
water level (m NAP)
0
5
10
15
20
25
30
35
40
Precipitation (mm)
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
date
precipitation
measurement 09
model_first
sewage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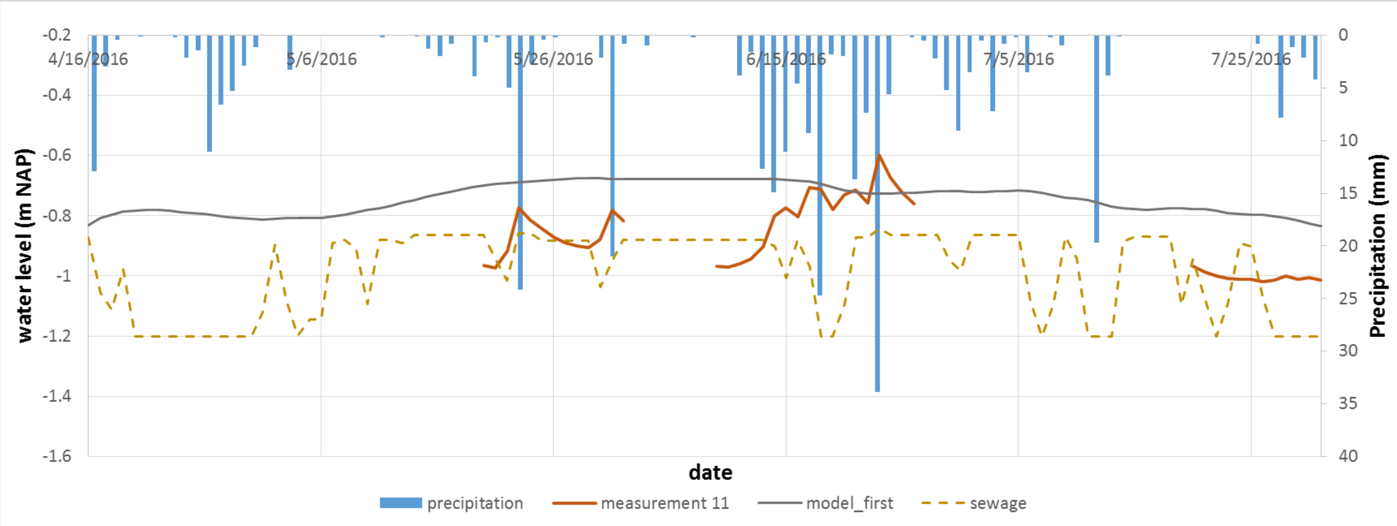
Precipitation (mm)
date
precipitation
measurement 10
model_first
sewage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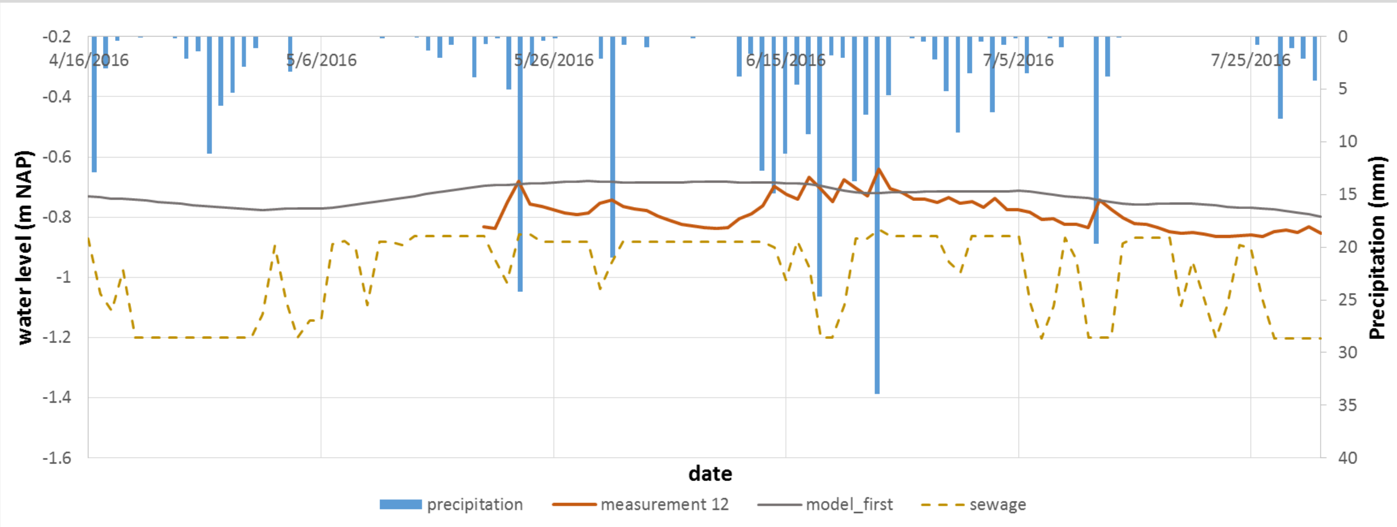
Precipitation (mm)
date
precipitation
measurement 11
model_first
sewage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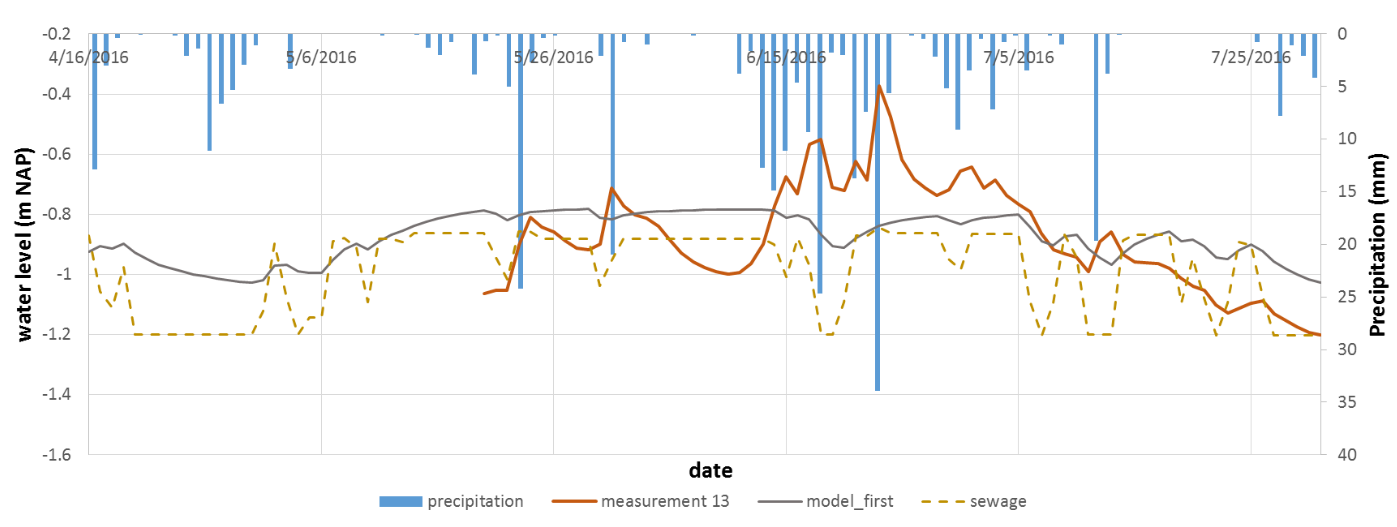
Precipitation (mm)
date
precipitation
measurement 12
model_first
sewage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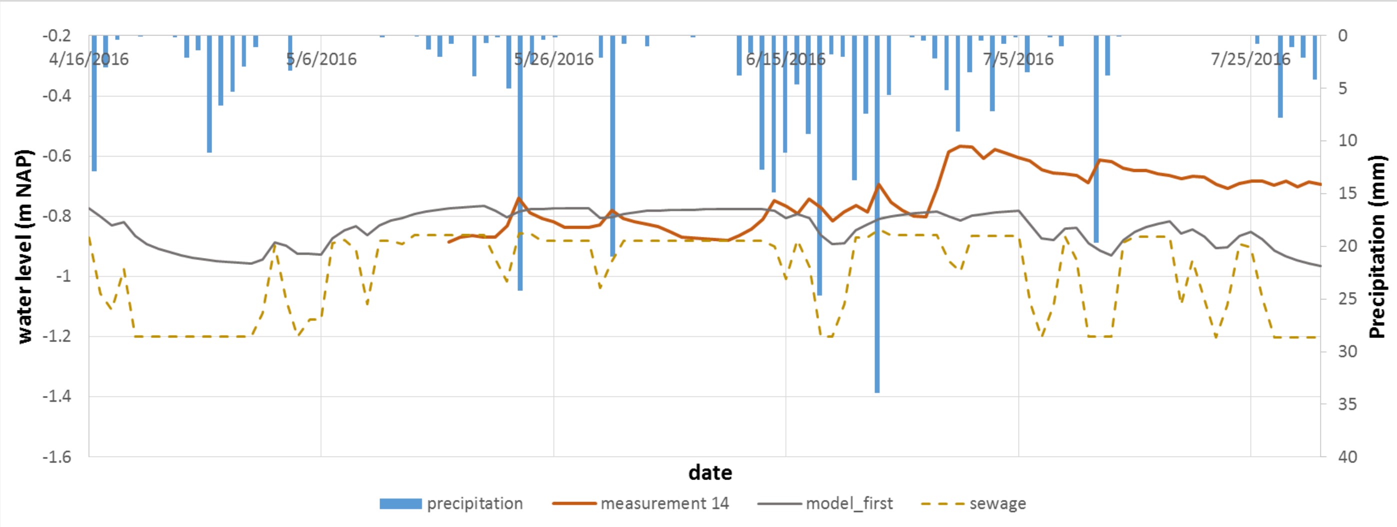
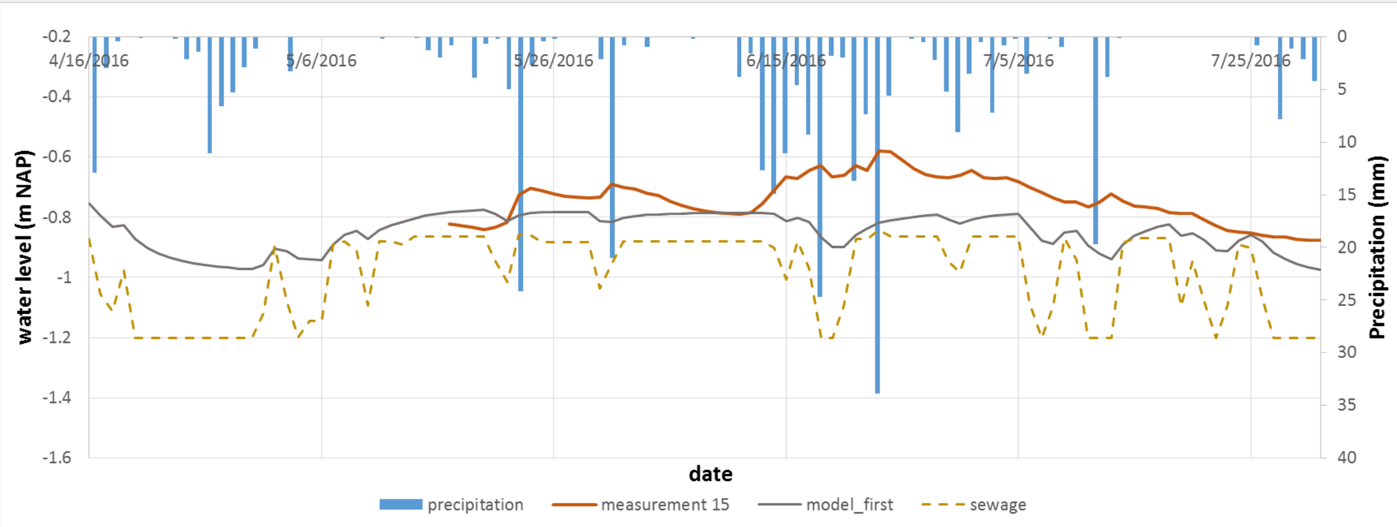
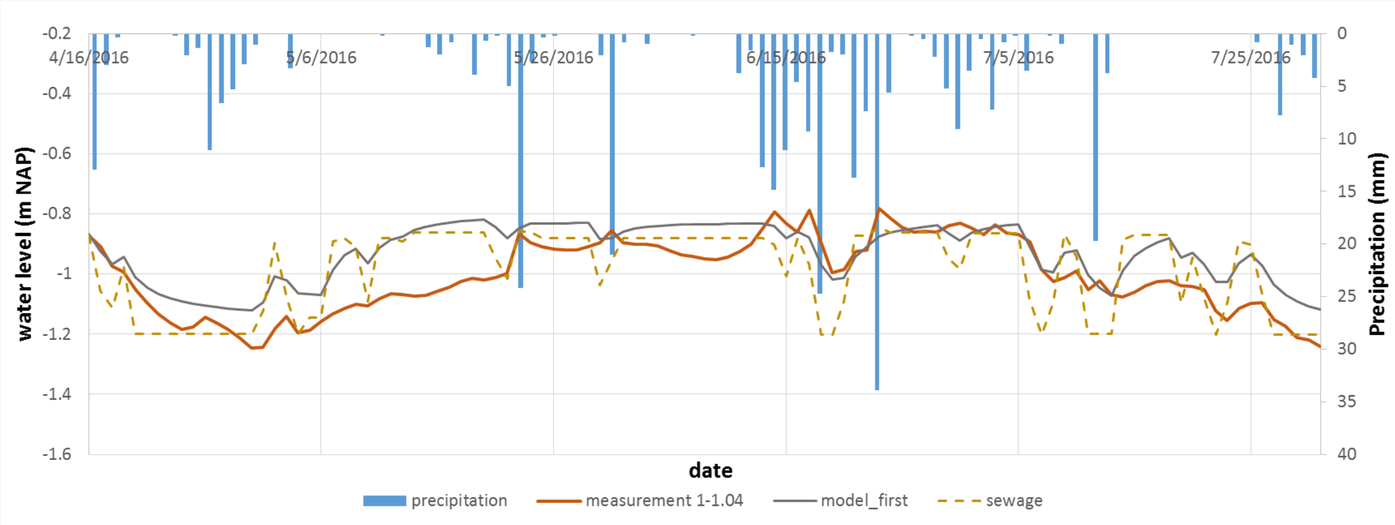
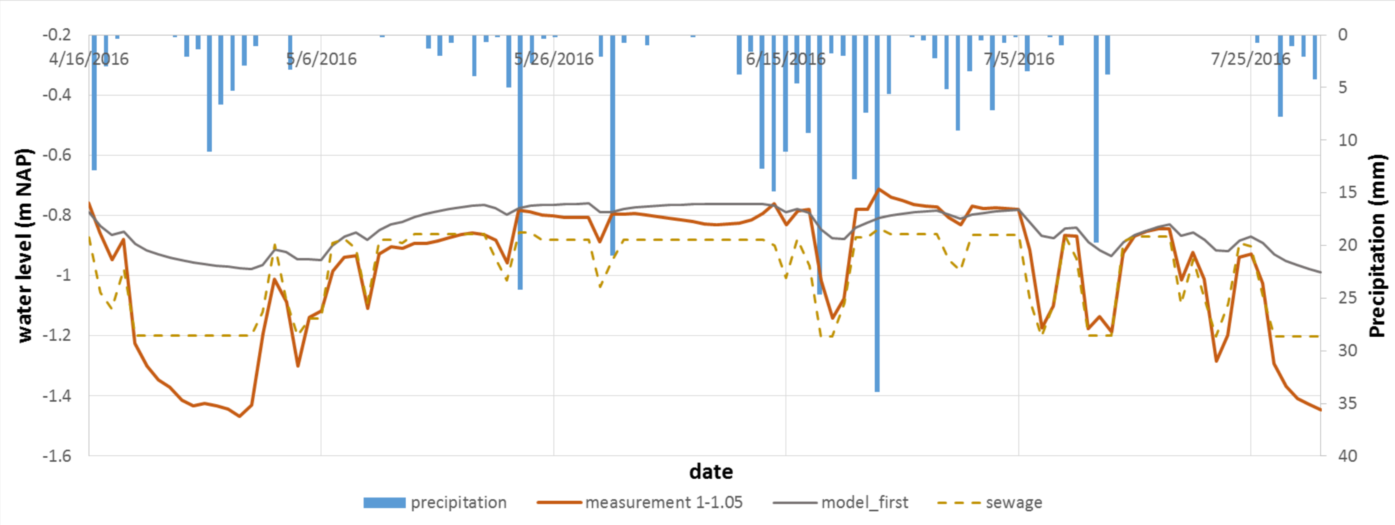
Precipitation (mm)
date
precipitation
measurement 13
model_first
sewage


water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 14
model_first
sewage

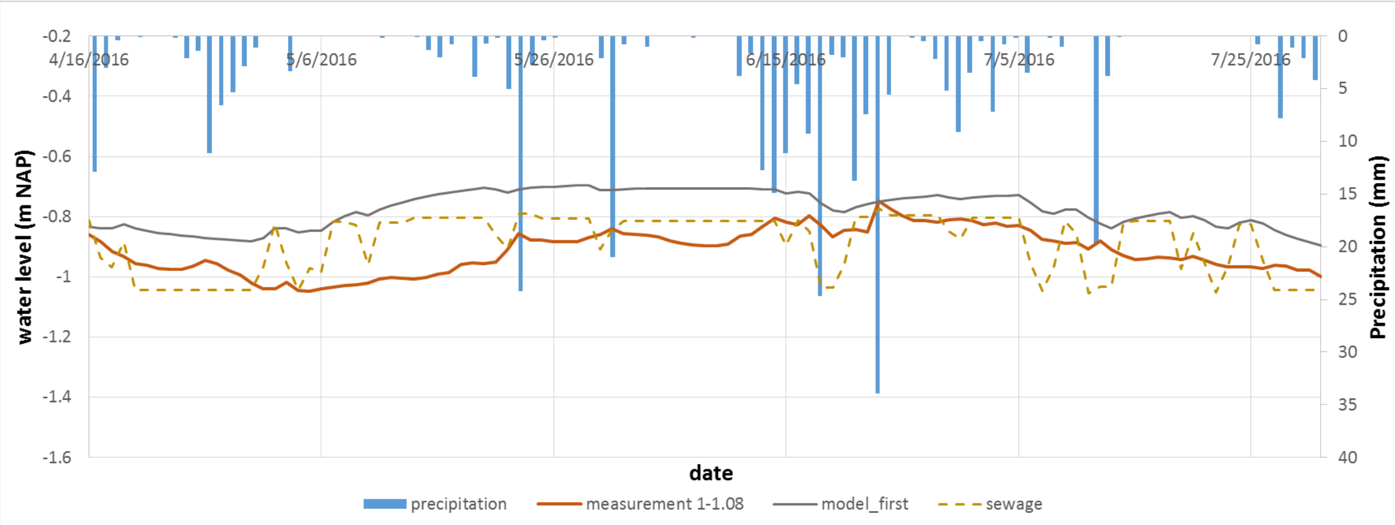

water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 15
model_first
sewage



water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 1-1.04
model_first
sewage



water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 1-1.05
model_first
sewage



water level (m NAP)
Precipitation (mm)
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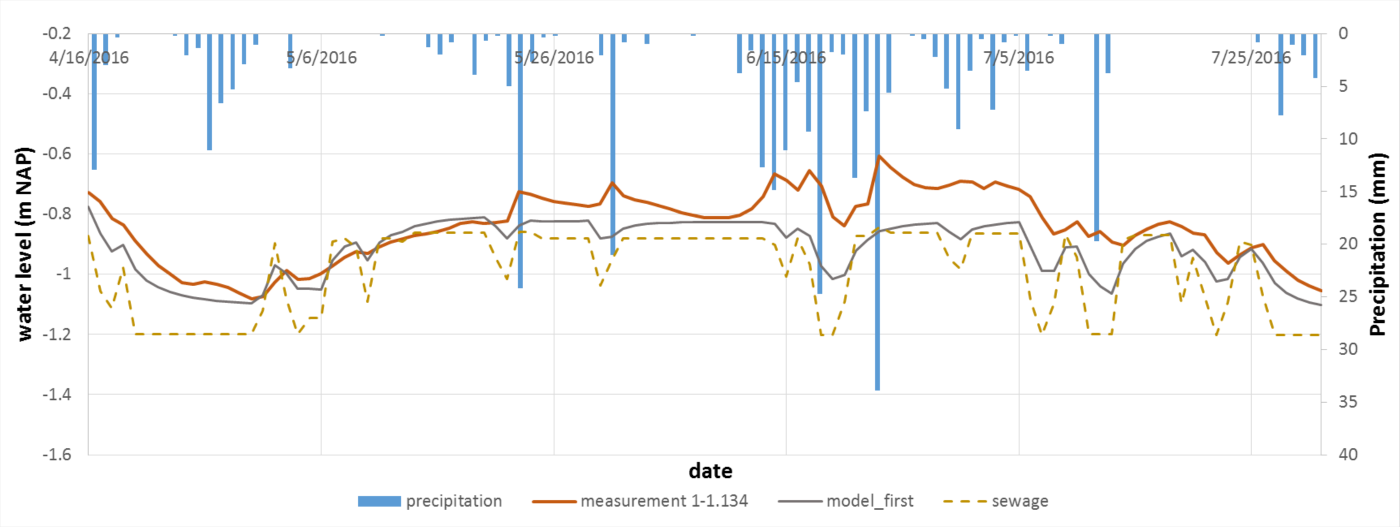
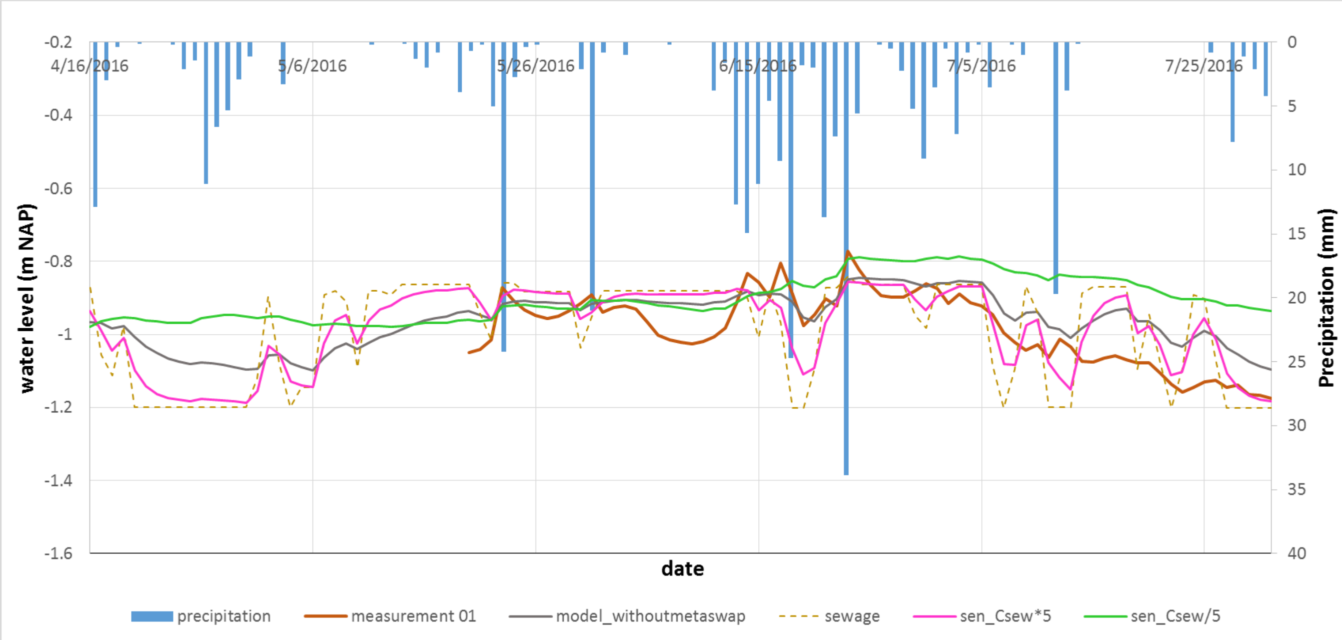
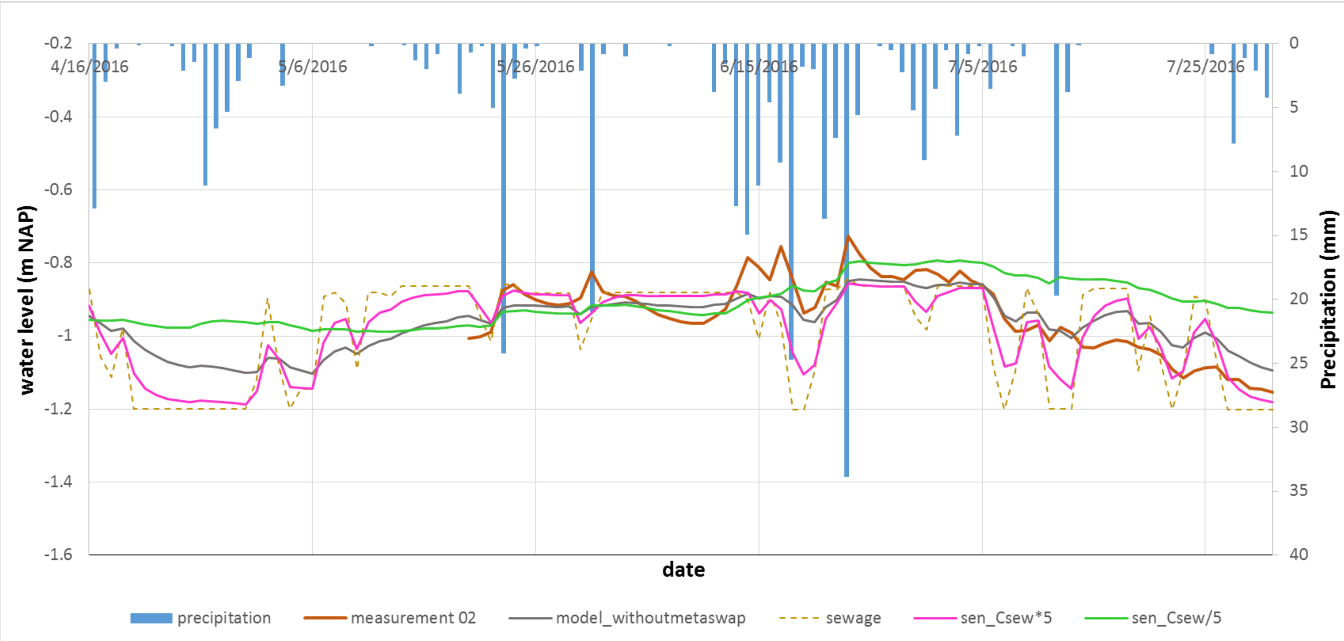
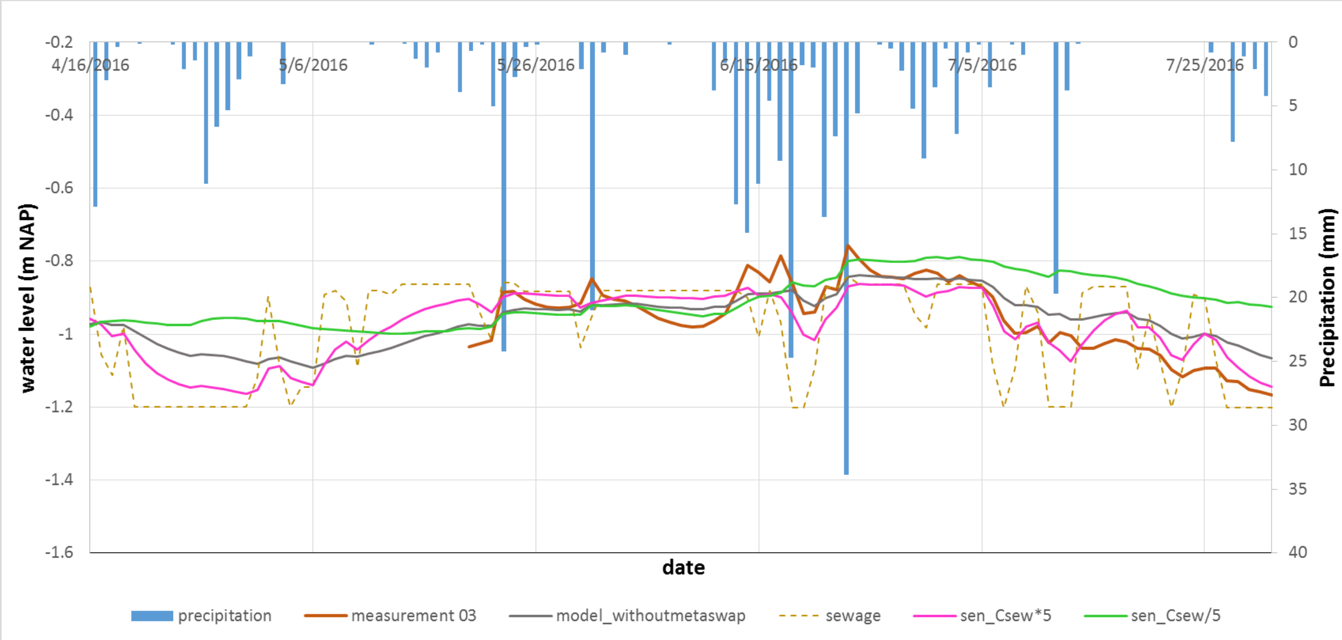
measurement 1-1.08
model_first
sewage

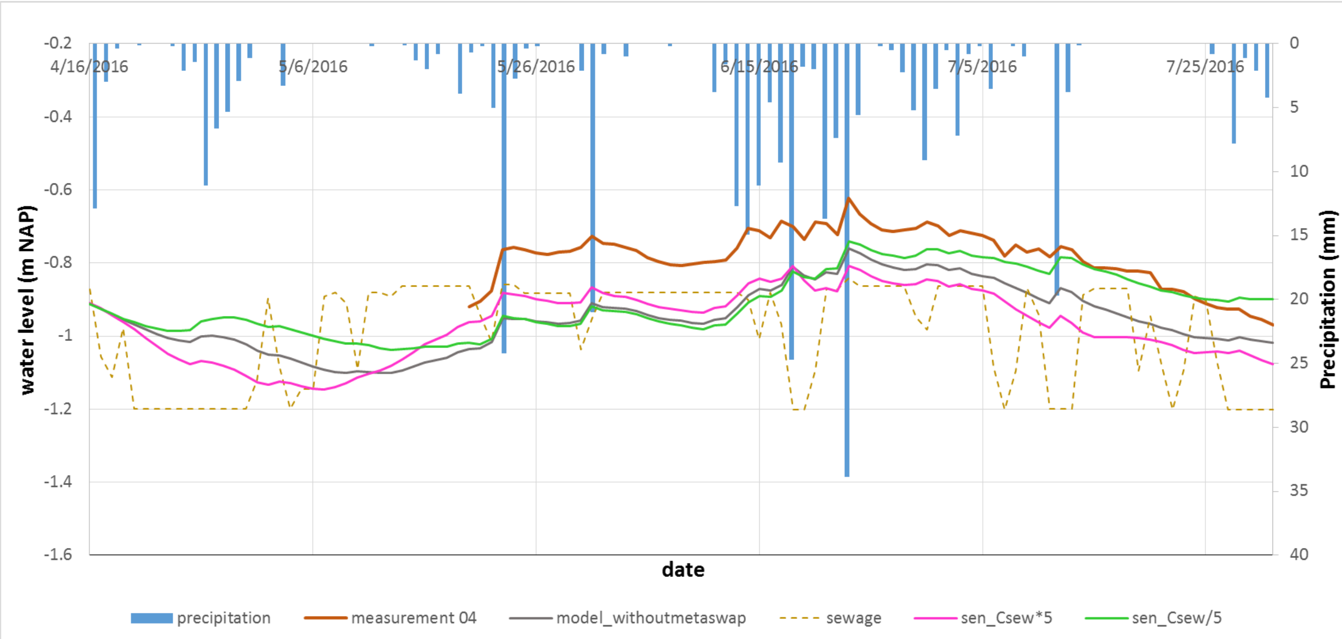


Simulation results of model without MetaSWAP and parameter sensitivity analysis:






water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 03
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5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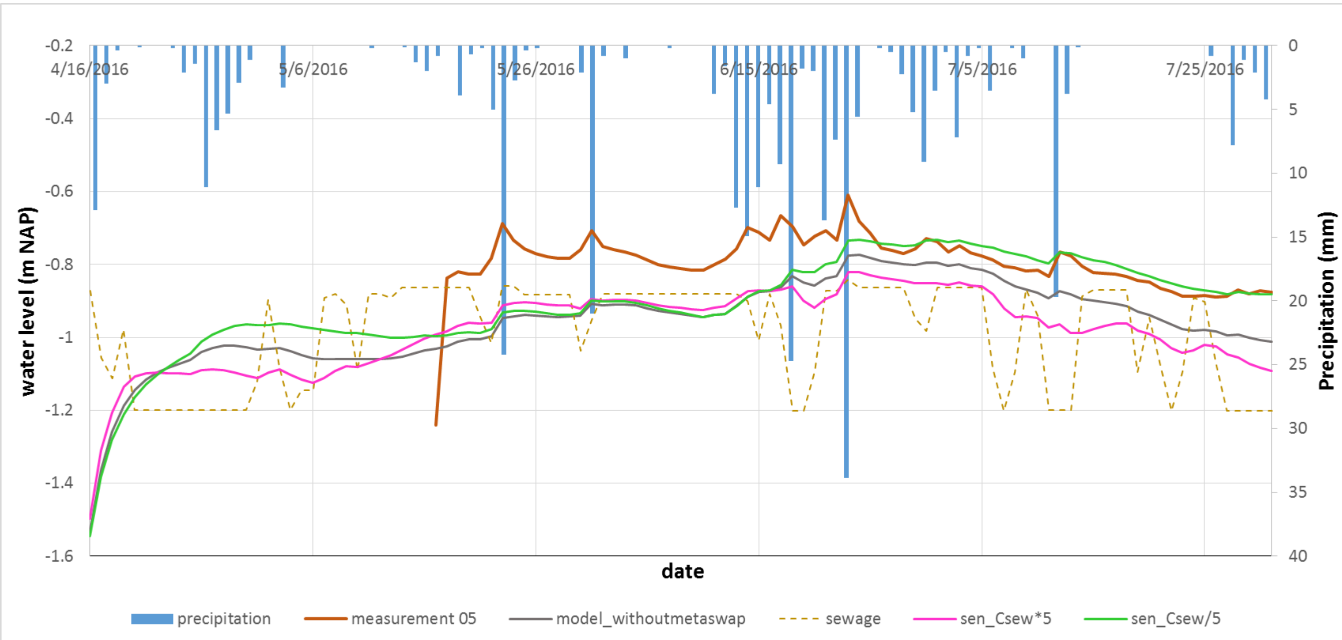
Precipitation (mm)
date
precipitation
measurement 04
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5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Precipitation (mm)
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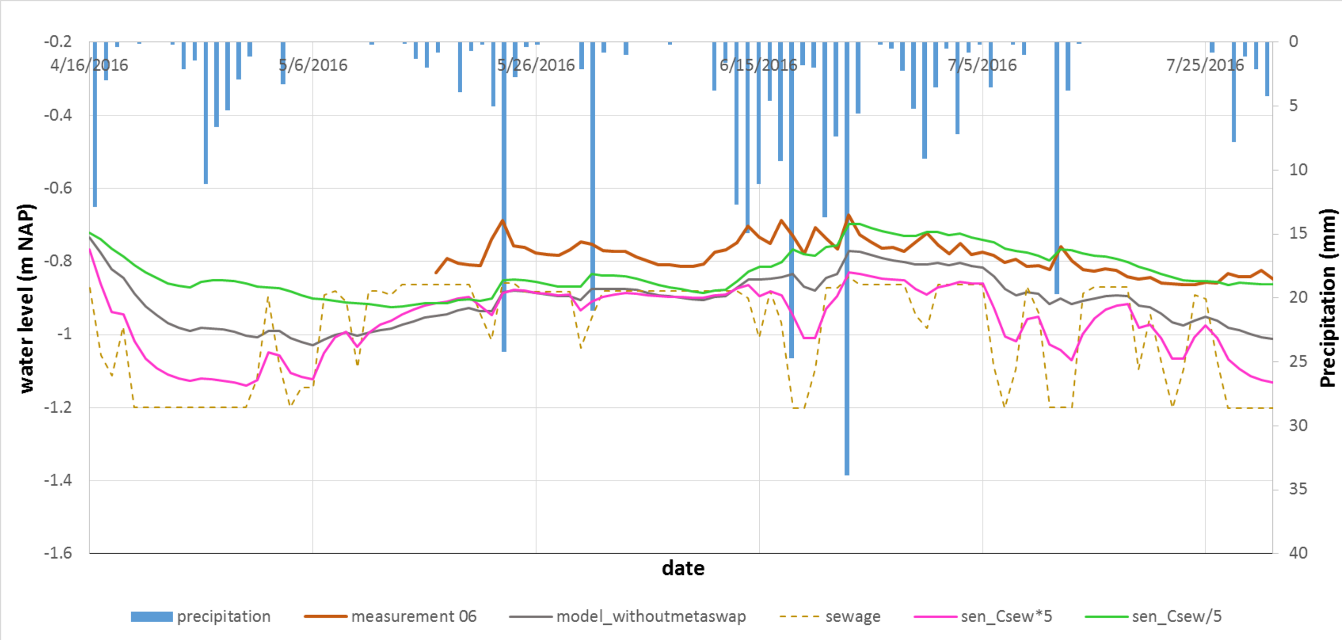
precipitation
measurement 05
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5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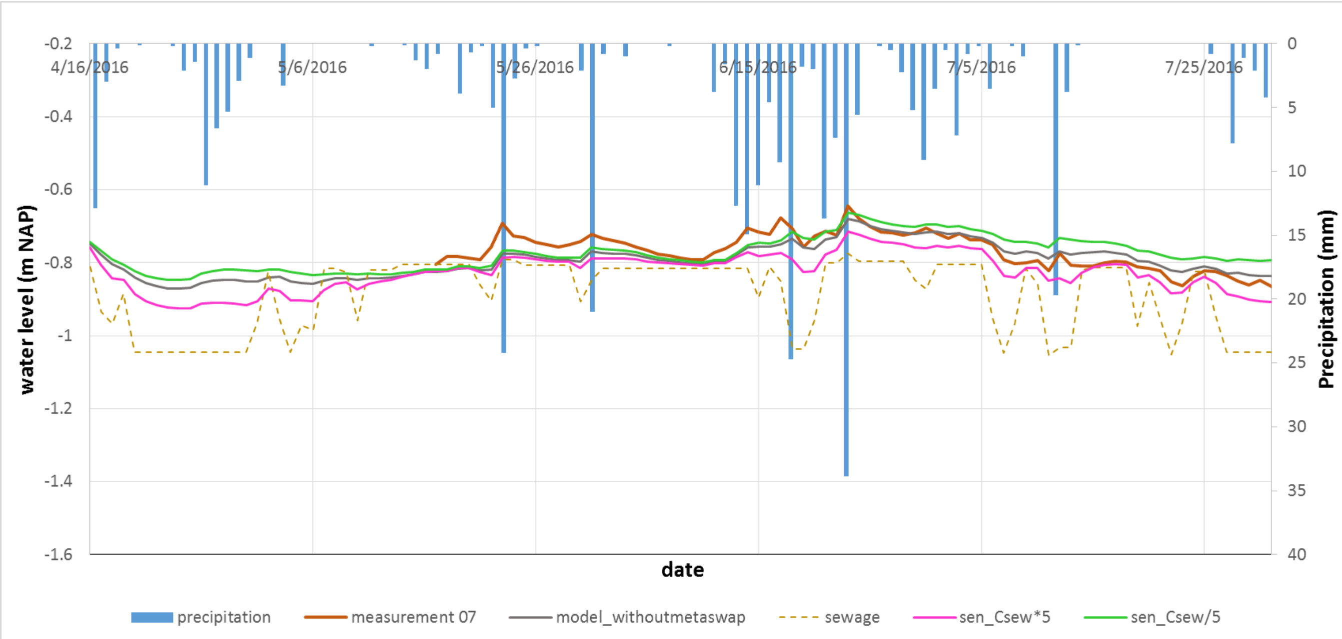
Precipitation (mm)
date
precipitation
measurement 06
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5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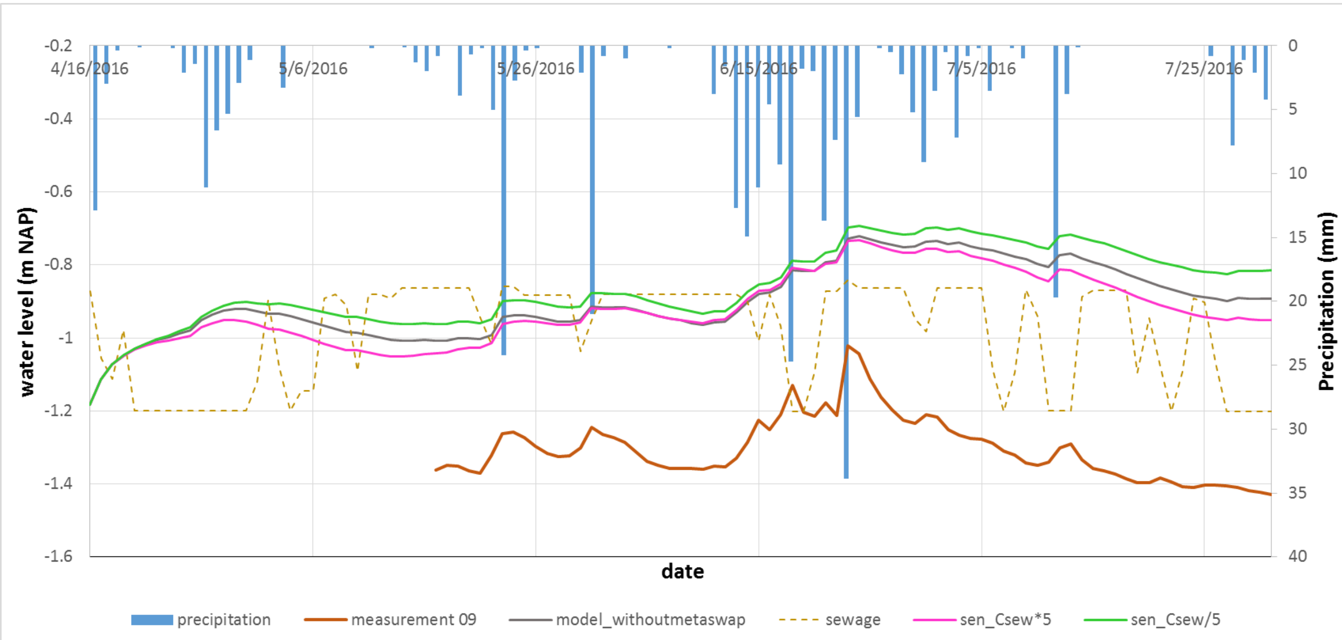
Precipitation (mm)
date
precipitation
measurement 07
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5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Precipitation (mm)
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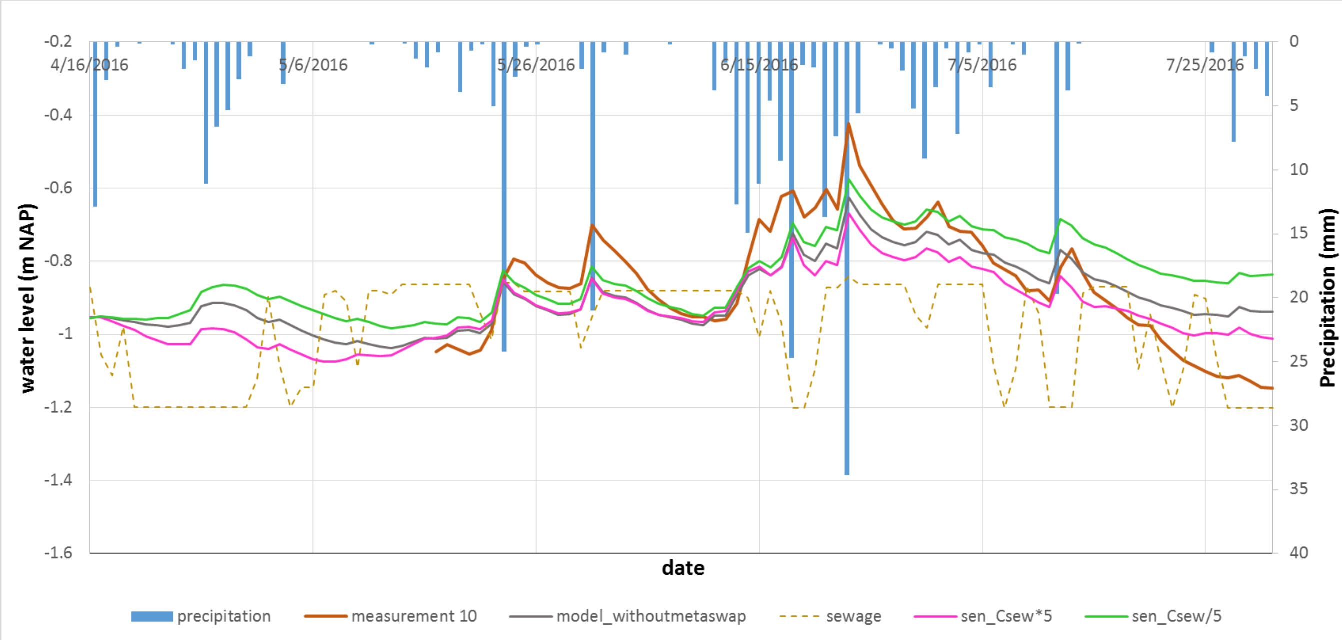
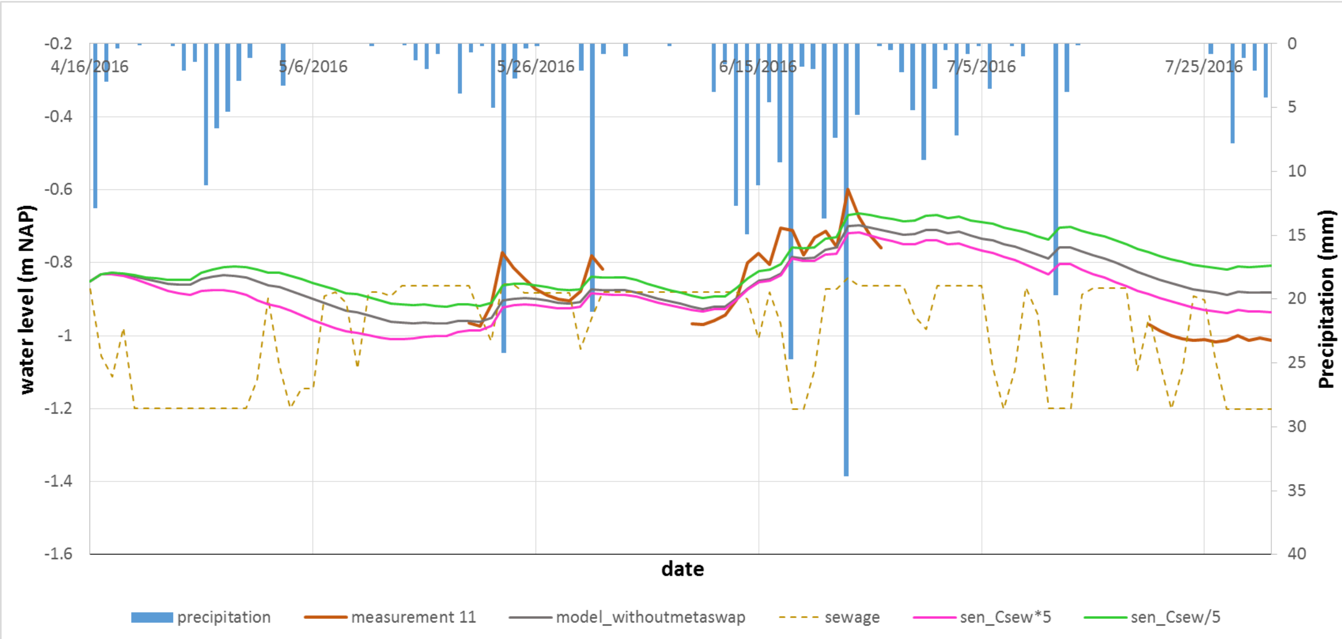
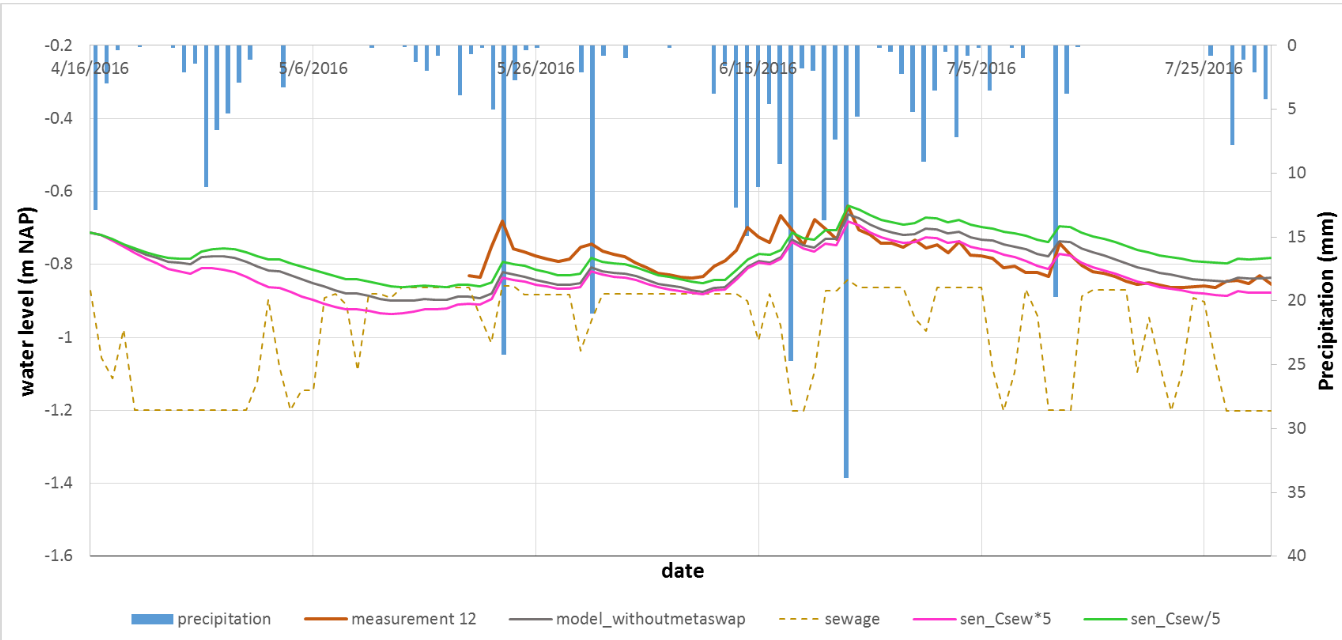
precipitation
measurement 09
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5



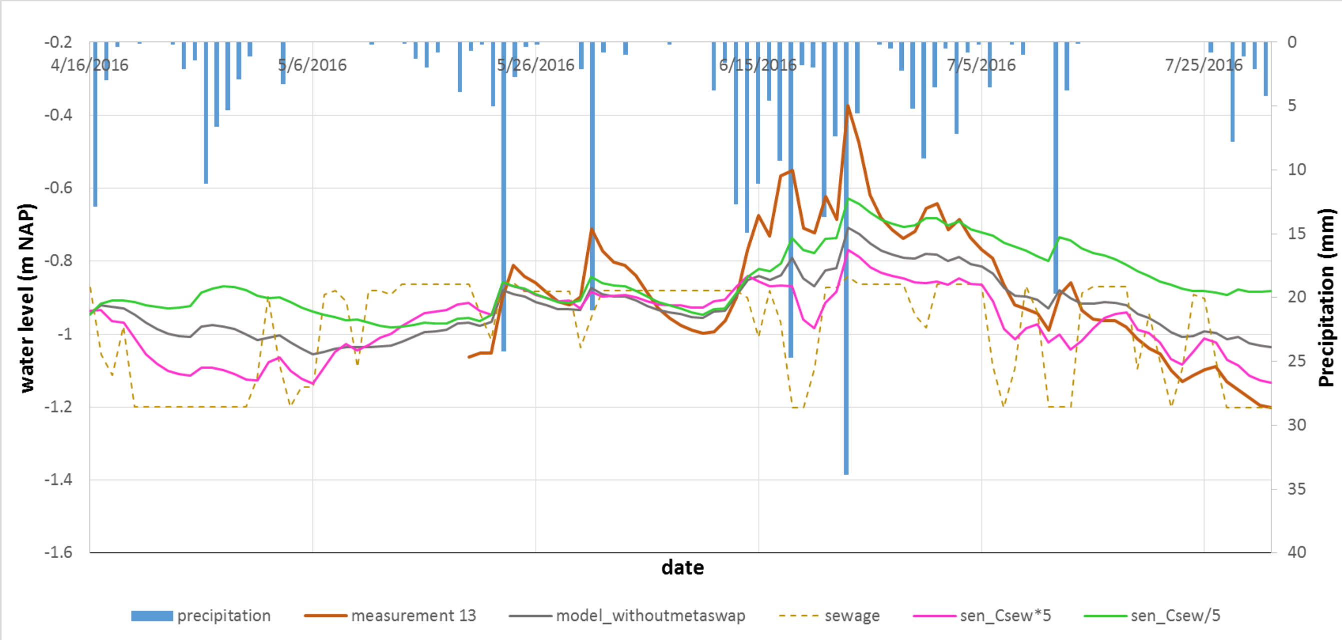
water level (m NAP)
Precipitation (mm)
date
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
precipitation
measurement 10
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5

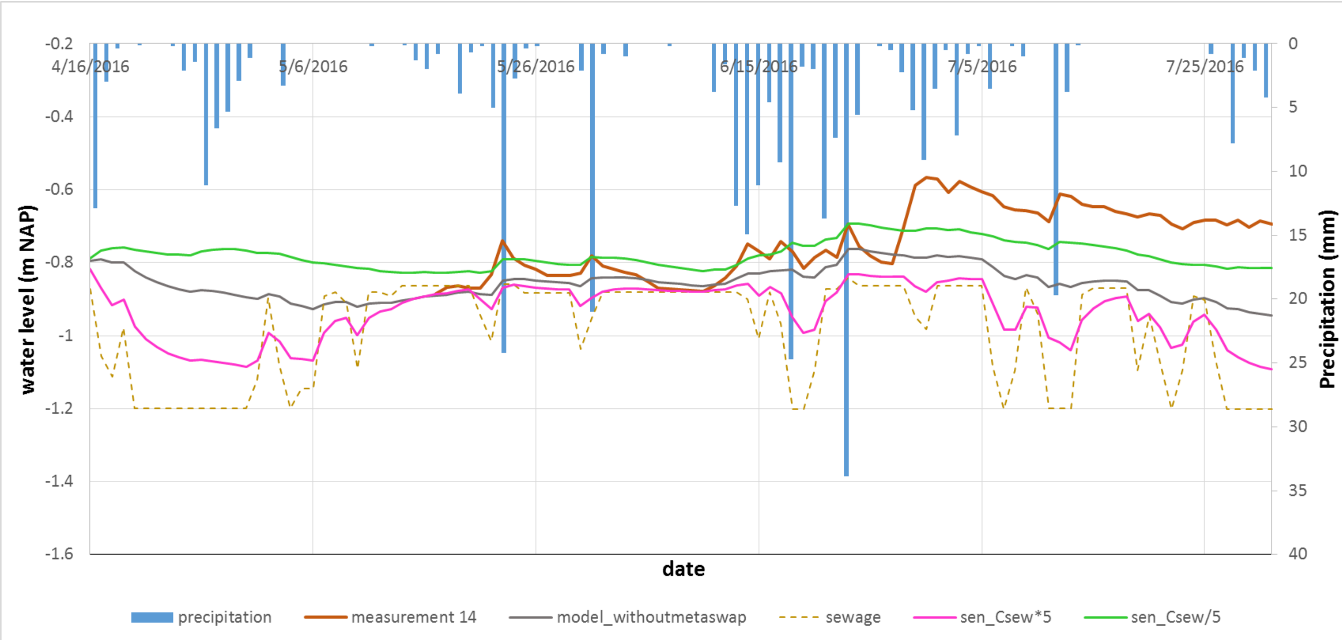


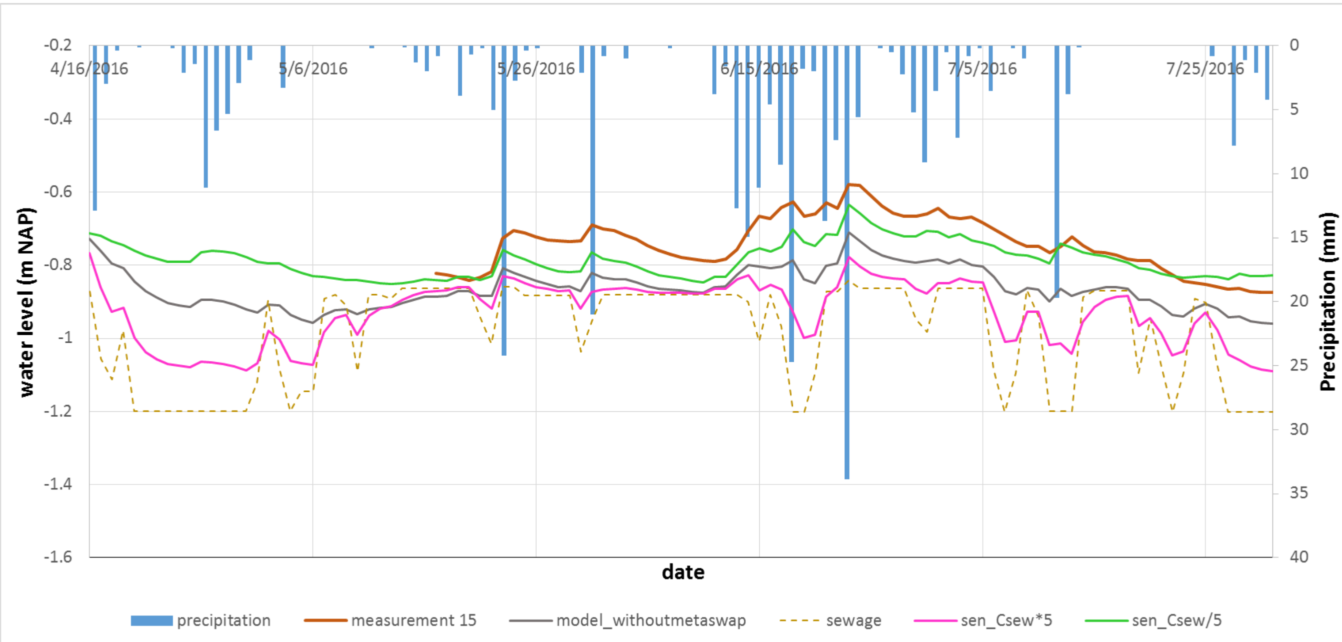
water level (m NAP)
Precipitation (mm)
date
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
precipitation
measurement 11
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5

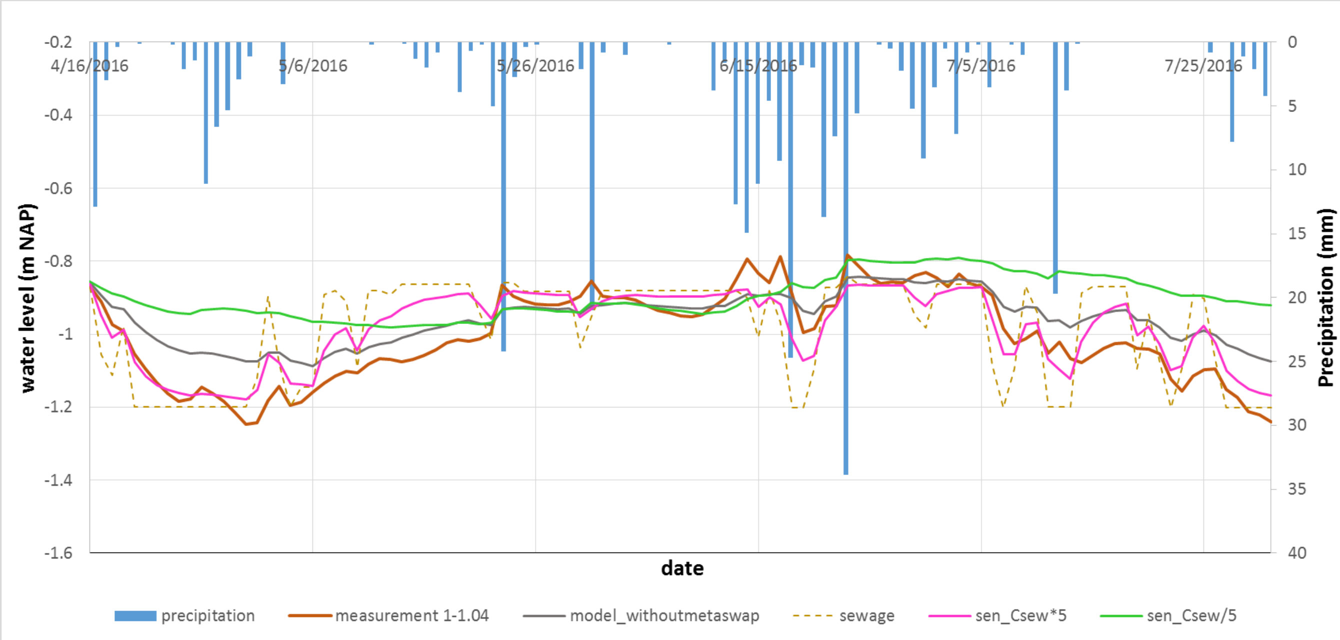


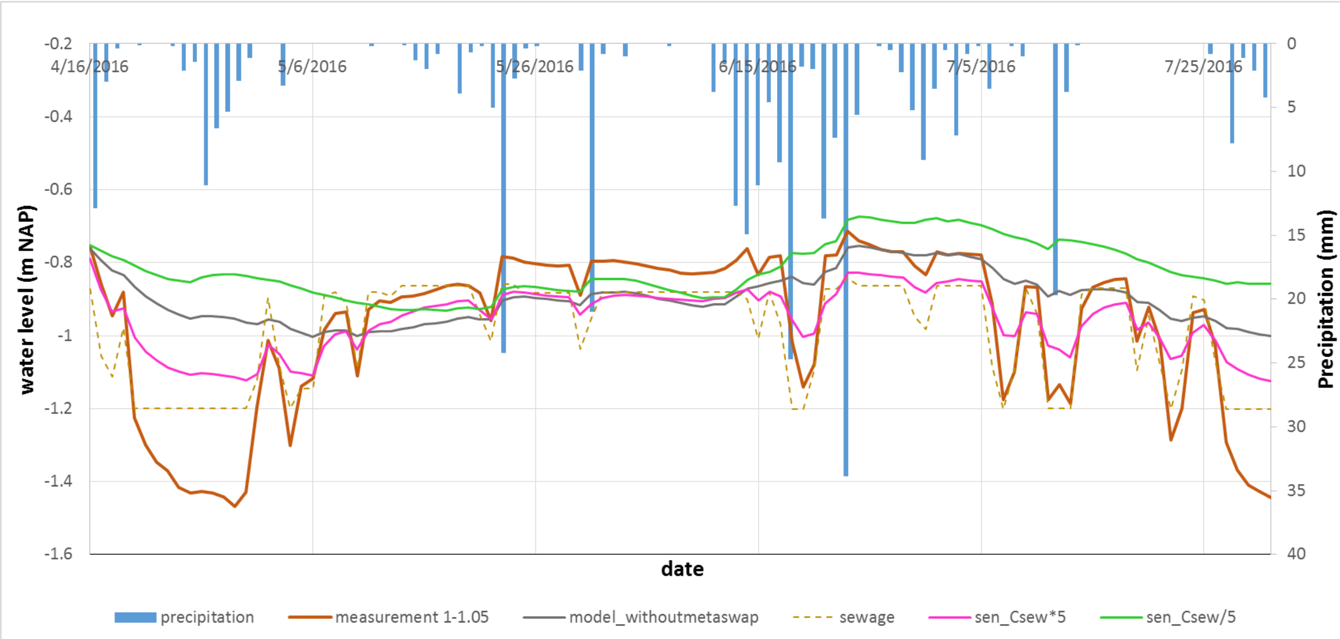
water level (m NAP)
Precipitation (mm)
date
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
precipitation
measurement 12
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5

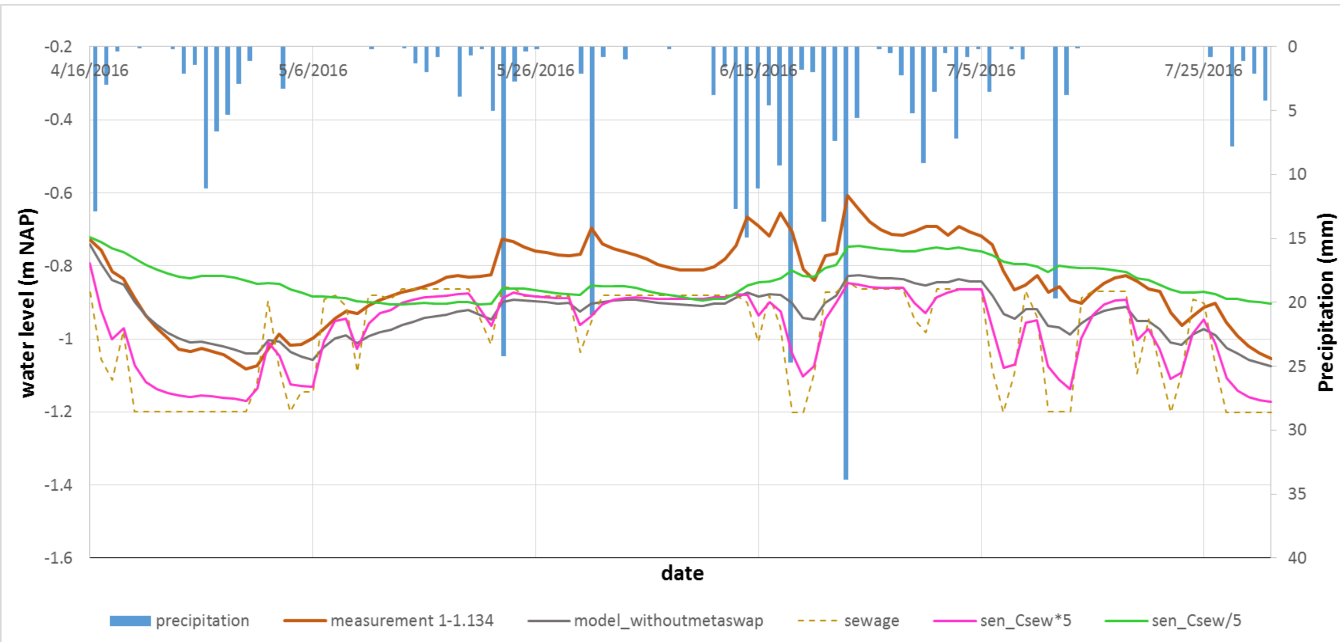

water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 13
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5

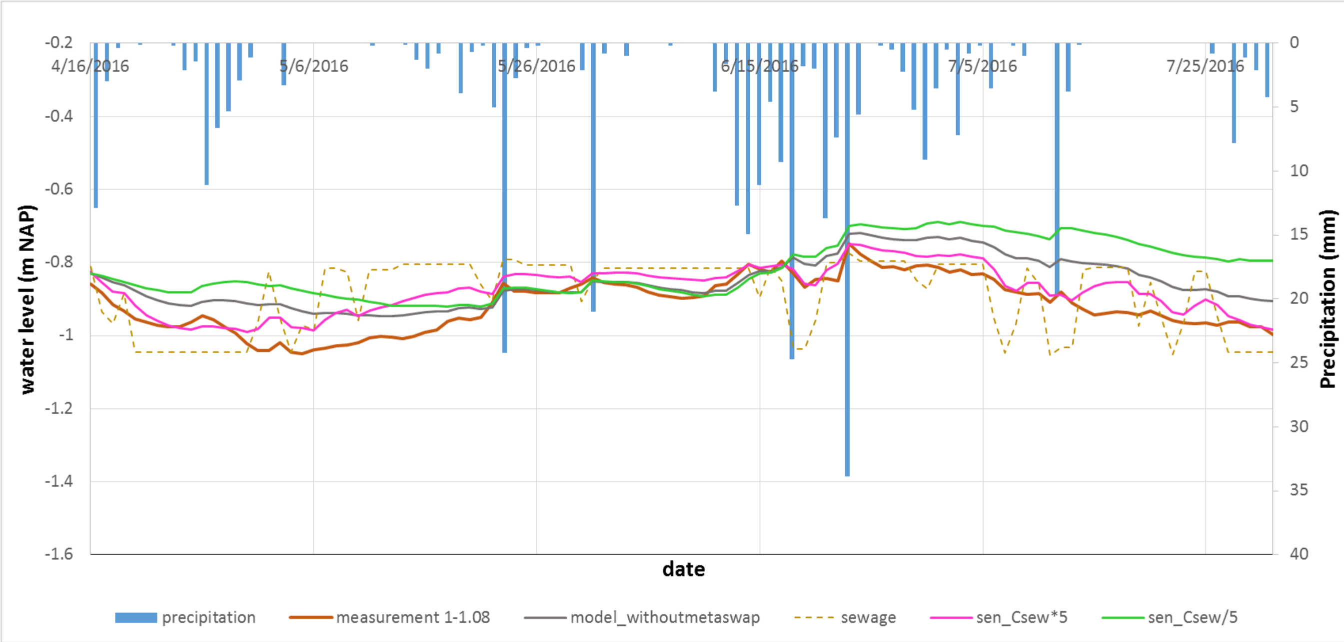

water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 14
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5


water level (m NAP)
Precipitation (mm)
date
precipitation
measurement 15
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5


water level (m NAP)
Precipitation (mm)
date
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
precipitation
measurement 1-1.04
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5


water level (m NAP)
Precipitation (mm)
date
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
precipitation
measurement 1-1.05
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5


water level (m NAP)
Precipitation (mm)
date
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
precipitation
measurement 1-1.134
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5



-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
water level (m NAP)
4/16/2016
5/6/2016
5/26/2016
6/15/2016
7/5/2016
7/25/2016
date
0
5
10
15
20
25
30
35
40
Precipitation (mm)
precipitation
measurement 1-1.08
model_withoutmetaswap
sewage
sen_Csew*5
sen_Csew/5