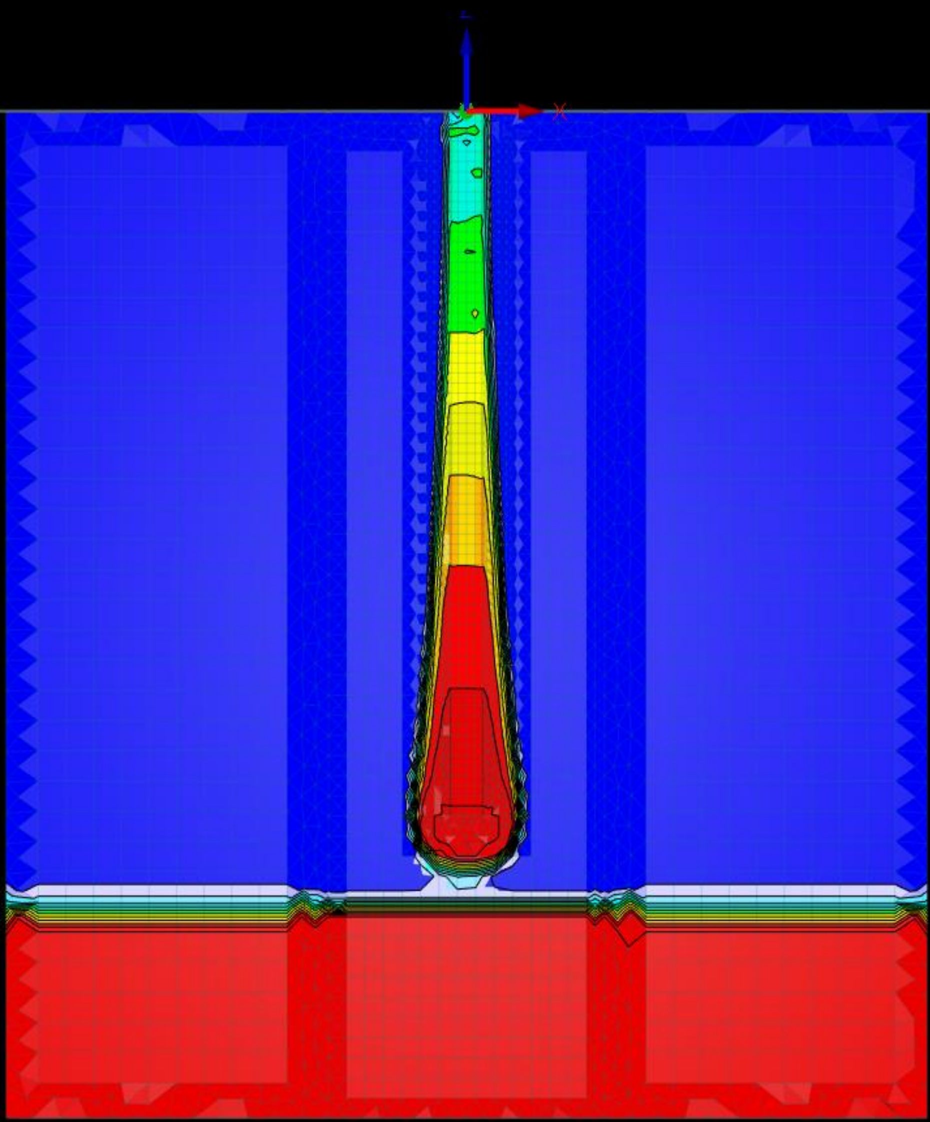
Redefining infiltration drywell design

A study on design and functioning in theory as in practice



Master Thesis

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Redefining infiltration drywell design

A study on design and functioning in theory as in practice

By

M. Schoenmakers

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This thesis is confidential and cannot be made public until 09-09-2022.

Abstract

This research is aimed at creating a generic design method for infiltration drywells on sandy soils by acquiring knowledge on the functioning of these wells. Infiltration drywells are vertical infiltration pipes that are installed above the groundwater table and through which stormwater is drained to the subsurface. Infiltration drywells can have a prominent place in urban water management since they mimic processes that occur under natural conditions. In urban areas the hydrological cycle is altered due to impermeable surfaces. Utilization of infiltration reduces the effects of the alteration on the hydrological cycle. Hereby, reducing the risk on urban flooding and surface water contamination. Furthermore, urban heat stress is reduced by enabling more drought resilient vegetation through groundwater replenishment. Especially considering that climate change will result in more extreme weather conditions, infiltration facilities can aid in creating more resilient urban areas. Until now, there are no design rules for these wells which hinders the implementation in urban areas that are fit for infiltration facilities. In this research a design method for sandy soils in the Netherlands is created and set forth.

The theoretical and practical performance of infiltration drywells is analysed by conducting experiments with Hydrus 3-dimensional geohydrological model simulations, as well as in the field and laboratory. In the field falling head tests were performed with existing infiltration drywells to determine the functioning while soil samples were analysed in the laboratory to determine the hydraulic conductivity. The model simulations also exist of falling head tests and are compared to the experiments in practice. It was found that the most important parameters on functioning of infiltration drywells are the soil hydraulic conductivity and well dimensions. When comparing the simulated falling head tests to field tests and laboratory tests at the same location, discrepancies were discovered. This can be clarified by simplifications that were made like homogeneity and isotropy of the soil in the model. Furthermore, the absence of wall resistance of the well in the model and the method that was used for the calculation of hydraulic conductivity using in practice falling head test data could be the cause.

To this end the generic design method is based on the Hydrus 3D model. This design method consists of empirical contour plots that give the necessary number of wells based on multiple input parameters, including a design storm of 21 mm in 10 min, which has a statistical return period of 25 years in the Netherlands. Due to discrepancies in the research, the design method is used to test the viability of a plan to implement infiltration drywells. Afterwards, a detailed design procedure is still necessary. Overall, the research resulted in a generic design method and shows the advantages of using infiltration drywells, which could be an essential part of urban water management in the Netherlands in the future.

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Contents

1 Introduction

1.1 Infiltrating stormwater

Climate change causes more extreme weather conditions (IPCC, 2021 and 2022). These extreme weather conditions will result in problems in the future. On one side, it will result in more intense rainfall events. On the other side, climate change will result in longer periods of drought. Cities already receive more precipitation than their surroundings according to Manola et al. (2019). This will increase the risk of urban flooding (Skougaard Kaspersen et al., 2017). As stated by Lehner et al. (2021) in some parts of the Netherlands, the return period of a current 100-year drought is decreased to 50 years or less by the 2070s. This will result in enhanced heat stress on cities, which in turn will influence the general liveability of cities. Concluding, after the 2014 climate change scenarios of the Dutch Royal Meteorological Institute (KNMI), and the updated version in accordance with the IPCC report of 2021 (KNMI Klimaatsignaal'21) climate change will result in more extreme precipitation events and longer periods of drought in the Netherlands (Klein Tank et al., 2014, Attema et al., 2014).

According to the European environment agency, extreme weather conditions will especially affect urban areas (Georgi et al., 2012). This is a consequence of the alteration of the natural hydrological cycle in these areas. Extensive changes in land use and -cover have a significant impact on the environment of our cities (Rusu et al., 2012). The presence of impermeable surfaces hinders the infiltration of stormwater into the subsurface (Mount, 1995). This leads to an increase in overland stormwater runoff, an increased total volume of water to be drained, and a larger peak flow (Butler & Davies., 2000). According to Prokop et al. (2011), about 8% of the surface in the Netherlands is impermeable (asphalt, buildings, etc.). This makes the Netherlands the most sealed-off country in the European Union after Malta. This has a profound effect on stormwater management. Under natural conditions, like in rural areas, most of the stormwater infiltrates and only 20% is drained directly through overland runoff via canals, rivers, and the drainage system. In cities, this number is up to 80% (Markovic et al., 2014).

In the Netherlands, the traditional solutions for urban stormwater drainage are based on grey infrastructure. This 'to pipe' principle focusses on hydraulic efficiency or in other words, draining the water from the urban environment as swiftly as possible (Semadeni-Davies & Bengtsson, 2000). This is in contrast to nature-based solutions like blue-green systems. These so-called sustainable urban drainage solutions (SUDS) make use of technologies that mimic the natural processes and so, are more sustainable than conventional urban drainage methods (La Rosa & Pappalardo, 2020, Griffiths, 2017). These blue-green systems have multiple environmental, social, and economic advantages (Hall, 2010). Examples are the increase in flood protection, in health, and in economic development. However, the European environment agency recommends blue-green infrastructure be used as an addition to the existing grey infrastructure systems (Isoard & Winograd, 2013). Therefore, the old systems should not be completely replaced by blue-green infrastructure. Instead, they will be an addition to the existing system. One of the proposed systems is infiltration facilities. Since these facilities enable the infiltration of stormwater into the soil, the natural hydrological cycle is partly restored. This will result in a more robust urban subsurface, through groundwater replenishment, and an urban area that is more resistant to extreme climatic conditions.

In cities in the low-lying areas of the Netherlands extensive canal systems are used for storage and transport of a large part of the expected increased precipitation. In the higher elevated parts of the country, where on average deep groundwater tables and high permeable soils occur, this is not a possibility due to a lack of surface water. As mentioned before, around 80% of the stormwater volume falling on paved surface is drained through the combined or separate sewage system. In the Netherlands 2/3s of the sewer system is combined (Stichting Rioned, n.d.). Combined sewer systems cause an additional risk of faecal contamination of surface water in urban areas (Ten Veldhuis et al., 2010). Tackling the increased number and magnitude of flooding by enlarging the sewers or installing separate sewer systems requires considerable investments. Replacing sewage piping before the lifespan runs out results in large financial deficits, according to Stichting Rioned (2012). In conclusion, cities in regions with permeable soils face considerable challenges in dealing with the extreme weather conditions in the near future.

Infiltration facilities tackle this problem by not only decreasing the pressure on the drainage system during rainfall events but also by replenishing the groundwater, increasing the stored water volume available during dry periods. Furthermore, since the stormwater first infiltrates into the groundwater, a time lag is created which decreases the peak flow through the combined sewer system and other drainage systems like rivers. Since the late 1990's infiltration facilities gained more popularity in the Netherlands (KIWA, 2001). A promising category of infiltration facilities is the infiltration drywell. An infiltration drywell (IDW) is a vertical perforated pipe that drains incoming precipitation, collected from paved and impermeable surfaces (roads, buildings, etc.), into the soil (Figure 1). It is installed in the vadose zone, or unsaturated zone, which is the part of the soil between the surface and the water table (Sasidharan et al., 2019). Above the water table, the pores are partly filled with water and air, while beneath the water table, the soil is saturated, and all pores are filled with water (Manning, 2016). Since IDWs are easy to install, only require little space, and are relatively inexpensive in comparison to other SUDS, they are expected to be a widespread practice in the Netherlands in the future. However, the problem is that the design of IDWs has not been researched due to complexity of the processes related to the functioning. In the absence of clear design rules for IDWs, these facilities are not commonly implemented up until now for urban stormwater drainage. Besides, the lack of clear design rules results in insufficient design. Consequently, this leads to the malfunctioning of IDWs, followed by a questionable reputation of IDWs. The outcome is a scarcely used facility while it could be a significant solution to create more climate resilient urban areas.
This report encompasses research into the relation between size, characteristics, and functioning of IDWs aimed to provide a generic design method for these facilities. This design method will be used in the development of IDWs in the future.

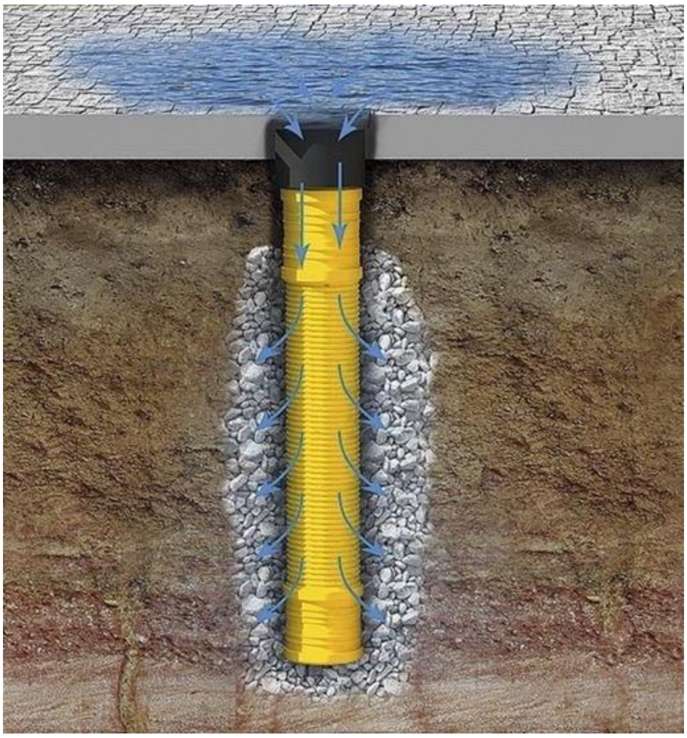


Figure 1: IDW in its simplest form (Leeuwenbergh Tuinen, n.d.)

1.2 Research objective and approach

The objective of this research is to *create a generic design method for IDWs*. For the creation of a design method understanding and quantifying of key processes and characteristics of IDWs and their effect on performance is necessary. To achieve this, a 3-dimensional geohydrological model is created in Hydrus 3D and laboratory and field experiments are conducted. For the laboratory and field experiments, a case study was applied. The case study of this research is the urban area of Hilversum in the centre of the Netherlands (Section 4.1 Research area).

To structure the report, every chapter has a main objective based on an important part of the research. This is clarified in Figure 2 where a schematization of the research approach is shown.

The various activities elaborated in each chapter are as follows:

II. A literary study is performed to determine which processes and characteristics are of importance for the functioning of an IDW. Furthermore, the concept of infiltration stormwater is elucidated. The knowledge is used to set up a 3D model.

III. A 3-dimensional, geohydrological model is set up to study the theoretical functioning of an IDW. This indicates the sensitivity of theoretical functioning to different parameters in order to get an understanding of the processes under relevant conditions in the Netherlands. It is done by creating a 3D model of an IDW and performing falling head experiments for different soil characteristics and well dimensions. A falling head experiment is a test where the well is filled with water and the drop in the water level is measured.

IV. In-situ and laboratory experiments are conducted to determine the practical functioning of IDWs in an urban area. Parameters of importance are determined in practice. The results are compared to the theoretical functioning of IDWs. This comparison links the in-situ functioning to the functioning of the theoretical model.

Furthermore, the important parameters in theory and in practice are compared and the main parameters are included in the design method.

V. The data obtained from simulations of the model form the basis for the generic design method. This design method is introduced and explained as well as the proper use of the design method.

VI. The results and limitations of the research are discussed in this chapter. Additionally, recommendations for future research are presented which can strengthen the overall knowledge of IDW design and use.

VII. In the last and final chapter, the most important conclusions of this research are summarized.

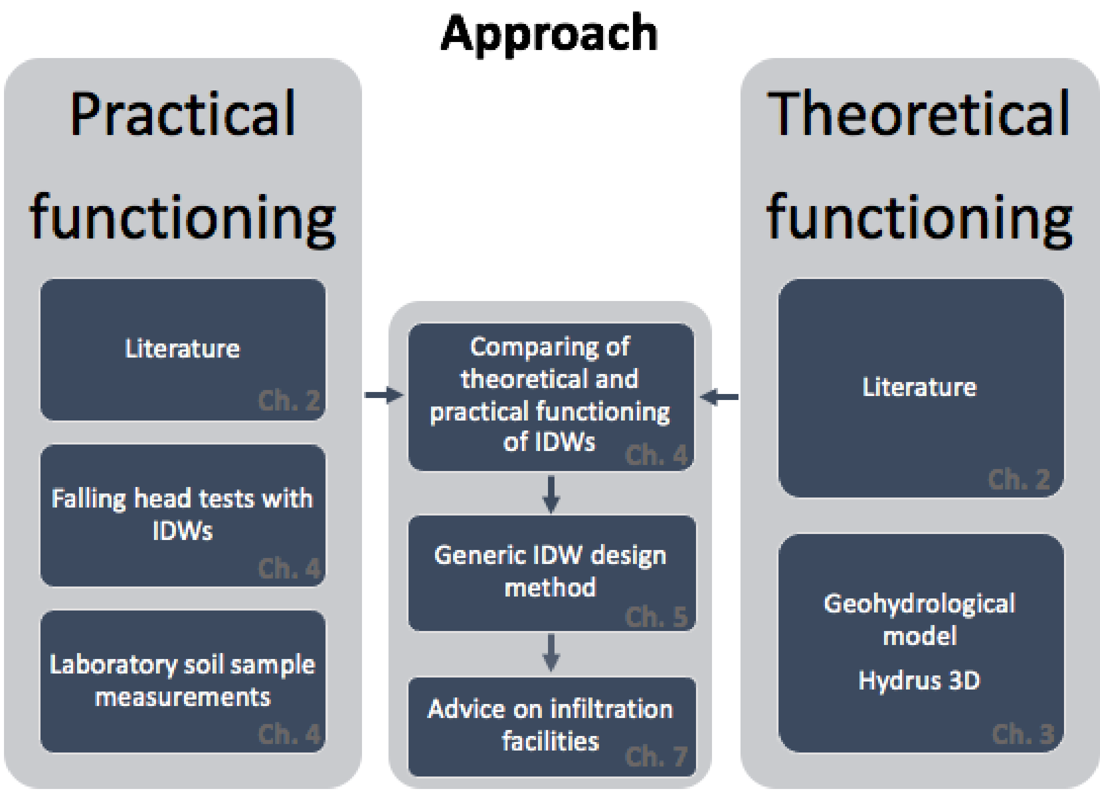


Figure 2: Schematic approach of this research

2 Infiltrating stormwater

2.1 Introduction

Large-scale implementation of infiltration facilities is not yet common use in the Netherlands. Across the field of urban water management, the call for blue-green solutions, to tackle problems of the urban water cycle, is increasing. As mentioned in the introduction, these problems include the alteration of the hydrological cycle due to impermeable surface and the accompanying risk of flooding and heat stress which are enhanced by climate change. Infiltration facilities can have a prominent place in urban drainage management as it offers substantial benefits in addition to grey infrastructure. These benefits include the partial restoration of the hydrological cycle resulting in a decrease in stormwater draining into the sewer system. The reduction of stormwater inflow in the sewer system reduces the risk of flooding. Furthermore, the reduction of stormwater inflow in the system also reduces the risk of the sewer system overflowing and the risk of contamination of surface water.

The risk of drought in urban areas is a development of increasing concern. Longer periods of drought will increase the urban heat island (UHI) effect. The UHI effect is caused by significant absorbed radiation due to low albedo surfaces, and relatively low wind speeds in cities (Claessens & Dirven, 2010). Due to groundwater replenishment, water availability during dry periods is enhanced, which in turn can strengthen urban vegetation. It is this vegetation that creates substantial social, economic and environmental benefits (Claessens et al., 2014). Vegetation can reduce the UHI effect through shading and evapotranspiration, both reducing the average temperature of their surroundings (Bowler et al., 2010).

Infiltration facilities make use of the natural storage capacity and infiltration capability of the subsurface. In this way, the natural process is artificially enhanced. The potential storage volume of the soil for water is considerably large, especially for regions with sandy soils and deep groundwater tables. Sand typically has a high porosity. This asset can be used for different goals e.g., when stormwater first flows through the soil, before reaching drainage canals and rivers, retardation of the peak flow is achieved (Claessens & Van der Wal, 2008).

Infiltration facilities can be categorized into three distinct categories: surface infiltration, vadose zone infiltration and direct injection wells. Surface infiltration encompasses facilities where the water infiltrates from the surface either to the groundwater or a subsurface drain. Examples are (bio)swales and permeable pavement. With vadose zone infiltration, the stormwater is directed to facilities in the subsurface from where the water naturally drains into the vadose zone. This is the part of the soil between the surface and the groundwater table and is usually unsaturated (Butler & Davies, 2000). Besides IDWs this also includes soakaway crates and infiltration trenches and sewers. With direct injection, the stormwater is pumped directly into the groundwater from direct injection wells. the difference with IDWs is that the water does not naturally drain, and it is usually injected deeper into the subsurface and so, directly into the aquifer.

The focus of this research is IDWs, which is a vadose zone infiltration facility. There are different types of infiltration facilities but IDWs have the advantage that they require little space, the installation is easy, and the costs are relatively low (Sasidharan et al., 2018). Chapter 2 focuses on prior knowledge of IDWs and important parameters on the functioning of IDWs. This knowledge aids in the creation of a 3-dimensional IDW model and the deduction of relevant parameters for the design method using the functioning of IDWs in practice and in theory.

2.2 Infiltration drywells

As described in the introduction, an IDW is a vertical perforated pipe wrapped in a geotextile that is installed directly in the vadose zone of the soil, above the groundwater table (Figure 3 or Figure 1). The geotextile is installed around the pipe to prevent flushing in of sediment from the soil. The stormwater is drained into the vertical well after which it infiltrates into deeper horizons (Pitt et al., 1999). The pipes are installed above the groundwater table to prevent the stagnation of water in the well during high groundwater levels. The installation above the groundwater table has an additional practical reason since drilling below the water table is difficult and cannot be achieved by using a simple drill. Also, stagnant water would enable the growth of biological organisms like algae, resulting in clogging of the well or the geotextile. In addition, this decreases the storage capability of the well.

Before the stormwater is discharged into the well, various stages of pre-treatment are advised (Bouwer, 2002). This pre-treatment can consist of simple filtration through grating and sedimentation, but it can also be extended with other filtration methods like membrane- or sand filtration. Downsides to extensive filtration are the additional costs of installation and maintenance of these filtration stages and the time lag that occurs of inflow into the well. If the rainfall intensity exceeds the velocity of the water flow through the filtration steps, the well will overflow, and excess water will drain into the sewage system. This is a substantial downside. A solution to this problem is the installation of a bypass that can be used to redirect water directly into the IDW in case of excess runoff. Ultimately, even with extensive pre-treatment wells clog eventually, if left undisturbed. For this purpose, the maintenance and cleaning of facilities should be given priority.

The wells are installed using an excavator drill, see right side of Figure 3, to mechanically drill a vertical hole into the soil. The pipes are made of geolon, which is a rigid PVC plastic intended for implementation in the subsurface. After drilling, the pipe is positioned into the hole after which the remaining gap, between the pipe and the soil, is filled with the excavated sand. For holes shorter than the length of the pipe (6 m) the remainder of the pipe is sawed off, after which the well is connected to gutters or road gullies and it will be sealed off by a manhole. From here on the well can be used for stormwater infiltration.

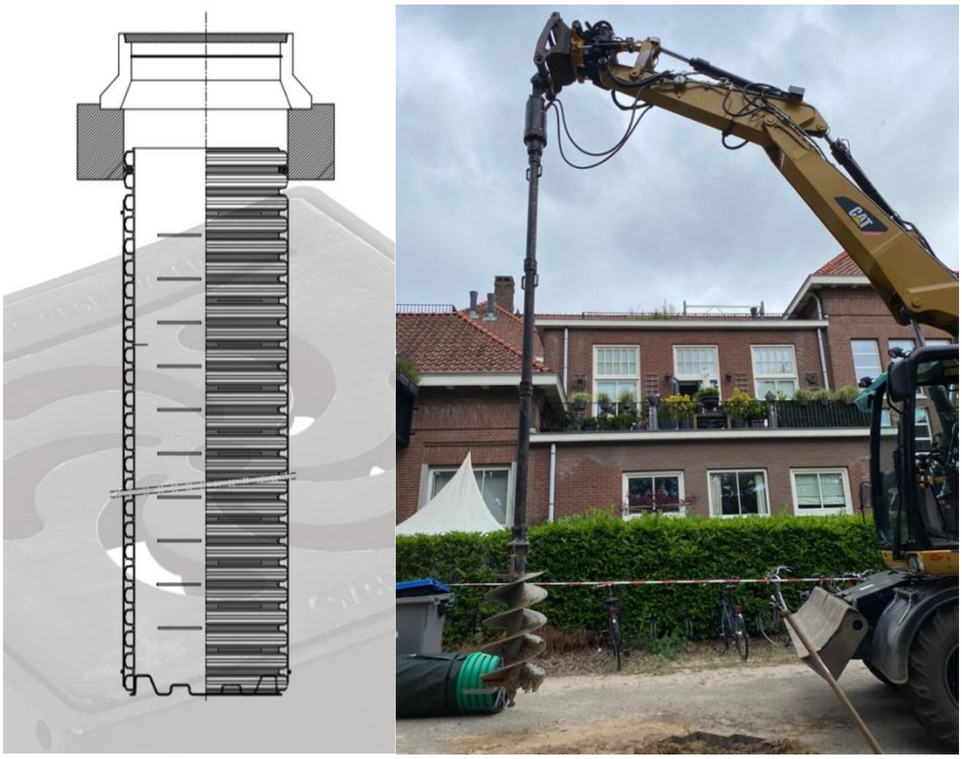


Figure 3: Left: schematic illustration of vertical infiltration pipes by Wavin. Right: drill used for installation of IDWs

2.3 Functioning

The quality of functioning of IDWs is dependent on different factors. These can be divided into:
- soil characteristics,
- input parameters for design,

The soil characteristics determine the circumstances under which the well functions. The design parameters are established during the design process based on the desired functioning. The design is ultimately made based on the surface area that has to be connected and limited by the soil characteristics and design storm.

2.3.1 Soil characteristics

The ability of water to infiltrate into and flow through the soil is one of the factors that determines the functioning of an IDW. The easier the flow of water through the soil, the better the performance of a well. If a well can quickly get rid of the incoming stormwater, it can serve a larger surface area. But what are the factors that determine the potential flow velocity of water through the soil? In this section, the most prominent soil characteristics and their influence on infiltration are discussed.

Infiltration rate and infiltration capacity

When considering infiltration one of the first things that come to mind is the infiltration rate or infiltration capacity. The infiltration rate is the amount of water that can infiltrate into soil for a given timestep. It is dependent on soil characteristics like soil moisture content and hydraulic conductivity but also land use, vegetation, slope etc. It is typically a vertical movement of water into the subsurface and it's measured in mm/hr or a similar unit (Robinson & Ward, 1990).

The infiltration capacity however is the maximum infiltration rate occurring for a specific situation. To give an example, when soil is completely dry all the pore space is available to be filled with water. Due to this, the water will infiltrate into the soil at a high rate. The infiltration rate will approach the maximum rate or in other words, the infiltration rate approaches the infiltration capacity.
If the same soil is partly saturated a large part of the available pore space is already occupied by water. Because of this, the water will infiltrate at a lower rate. Even though both soil samples have an equal infiltration capacity, the initially dry soil will have a higher infiltration rate. The infiltration rate could also be limited by other factors. To give another example, two identical soil samples are considered. Since the samples are the same the infiltration capacity is also equal. Now consider the precipitation on one of the samples to be lower than the infiltration capacity. Due to the limited availability of water, the infiltration rate is lower than the infiltration capacity. If for the other sample the precipitation rate is equal to or higher than the infiltration capacity, the infiltration rate will be equal to the infiltration capacity.
As shown with these examples the infiltration rate and capacity are dependent on other soil characteristics. The important characteristics will be discussed here in more detail.

Porosity
The porosity of soil represents the percentage of the volume that consists of void space. In other words, this is the percentage of volume in the soil that can be filled by a fluid, like water or air. Logically the porosity influences the infiltration capacity of the soil. The larger the porosity, the larger the available volume that can be filled with infiltrating water. In sandy soils the porosity usually varies between 30-50% according to Robinson & Ward (1990) and for the Netherlands specifically, the total porosity for sandy soils is on average 38% (Olsthoorn, 1977).

Hydraulic conductivity
The hydraulic conductivity (k) is a soil characteristic and an indicator of the permeability of the soil. It has the dimensions of a velocity [L/T] but it is in fact not a velocity but the proportionality factor between the hydraulic head and the flux through a permeable medium with dimensions $L^3/L^2/T$. In easier terms, the hydraulic conductivity is a measure of the ability of a soil to transmit water. The hydraulic conductivity is again dependent on other factors. These factors can be attributed to the fluid characteristics as to the solid characteristics.

According to Robinson & Ward (1990), the fluid characteristics seem to affect the hydraulic conductivity to a smaller degree than the solid characteristics. Important fluid factors are viscosity and density, which are in turn dependent on the temperature and salinity. For a temperature increase of the fluid the viscosity will decrease and so will the velocity at which the water flows through the soil. It can be assumed however that temperature, and so viscosity, variations are slim and therefore will not be accounted for in this research (Robinson & Ward, 1990).
Salinity can influence fluid density, but it can also affect the aquifer material. This is mainly the case for clayey soils. For this research into sandy soils, it can be assumed this is not an important factor and thus will not be accounted for in this research.

The solid characteristics influencing the hydraulic conductivity are pore space geometry, the geometry of the solid particles and the presence of macropores and such (Robinson & Ward, 1990). These factors are mainly of importance for groundwater flow for aquifers in bedrock and similar sediments. Since the sandy soils are usually made up of fluvial and alluvial sand deposits, they are usually well rounded. For this reason, the solid characteristics will not have a particular outspoken effect on hydraulic conductivity variations and so, will not be accounted for in this research.

Soil water content

As demonstrated with examples in the sections on infiltration rate and hydraulic conductivity, the level of saturation of the soil determines the rate at which the water infiltrates. The wetter the soil, the more the pores are occupied by water. When soil is completely saturated only flow of water through the pores will occur, so the infiltration rate is solely determined by the hydraulic conductivity and therefore constant (Figure 4). For the design process, it is important to use saturated conditions to determine the minimum infiltration rate of a well. Under unsaturated conditions, an overestimation of the functioning of an IDW is risked. This has to be avoided in IDW design.

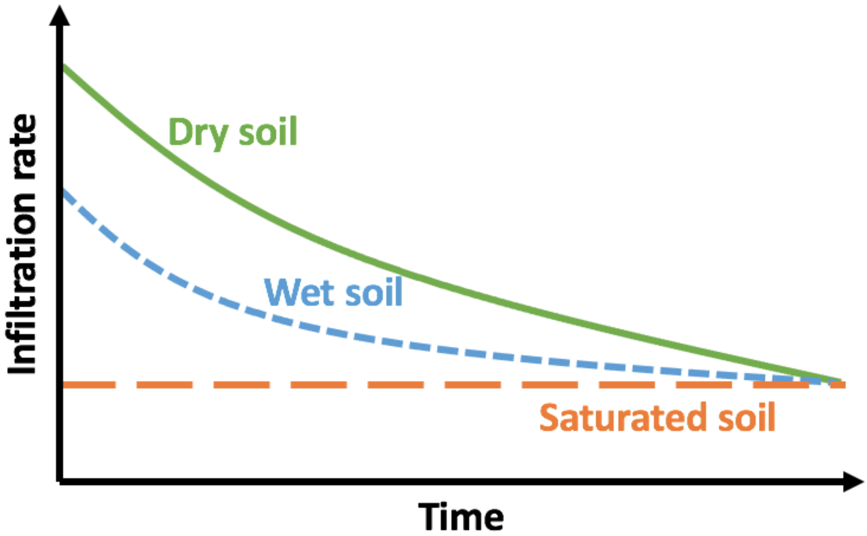


Figure 4: Relation between soil moisture content and infiltration rate

Soil type

The type of soil greatly determines the soil characteristics. In this research, the focus will be on sandy soils, as most IDWs are installed in high permeable soils in the Netherlands. Silt and clay soils have a low hydraulic conductivity, which makes them less fit for implementation of infiltration facilities.

Land use

Land use might not be the first characteristic that comes to mind when considering IDW functioning. Even so, it can greatly influence the infiltration capacity of soil. Soils that are subjected to heavy traffic like agricultural, transport or maintenance vehicles are compacted and can see a considerable decrease in infiltration capacity. According to Deb & Shukla (2012), annual tillage negatively affected the saturated hydraulic conductivity. Even though land use does affect infiltration capacity of soil, it is difficult to account for it in the model and design method. It is however of influence on the soil measurements explained in Chapter 4 Field and lab experiments.

2.3.2 In- and output parameters for the design

The importance of the soil characteristics on the functioning of an IDW seems clear but the functioning is also dependent on the design parameters that are used. In this section, the main parameters for the created design method are discussed.

Input

Connected surface area

When initiating an IDW design, the first input parameter is the surface area that should be connected to the well(s). This determines the volume of water the well(s) should be able to drain in the design process. The larger the surface area, the more stormwater has to be drained. In the design method, the connected surface area will be one of the main parameters to determine the number of IDWs. The amount of incoming stormwater connected to this surface area is determined by the design storm.

Distance to the water table

Another limiting factor in IDW design is the distance to the water table. As mentioned in the previous chapter, the wells are installed in the vadose zone, above the water table. Therefore, the groundwater level restricts the total length of the vertical infiltration pipes. In the design method, the water table depth is one of the main parameters used. It should be mentioned that this indicates the maximum depth of the well, so it could be decided in the design process to install a well with a smaller depth. This is therefore a design choice that is limited by water table depth.

Precipitation amount

To come to a sensible design method a design storm is established. This is a hypothetical rainfall event that is used to determine the performance of a design. This hypothetical event is based on rainfall statistics. Commonly an intensity-duration-frequency (IDF) curve is used to determine the precipitation amount of a design storm, resulting in block rainfall, a rainfall event with constant rainfall intensity (Butler & Davies, 2000). IDF curves are based on historical statistics. They depict the rainfall intensity over the duration of the events for different return periods (Figure 5). The return period is the statistical frequency of an occurring rainfall event and is based on extreme value statistics. As stated by Butler & Davies "*An annual maximum rainfall event has a return period of T years if it is equalled or exceeded in magnitude once, on average, every T years*" (Butler & Davies, 2000, p. 77).

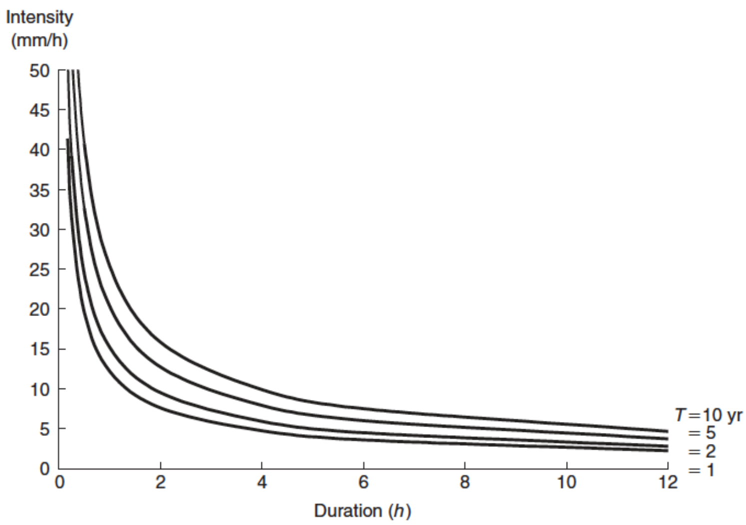


Figure 5: Typical IDF curve after Butler & Davies (2000)

In the Netherlands (and internationally) many organizations, like municipalities, use precipitation statistics as input for the design of water management structures. These form a reliable basis for design. The proposed design method is also based on precipitation statistics.

The Dutch royal meteorological institute presents general precipitation data for the Netherlands every approximately seven years. At this moment the most recent climate scenarios are from 2014 (KNMI, 2014) with new climate scenarios being published in 2023. In association with the KNMI, the STOWA foundation published precipitation

statistics in 2019 (Table 1). The return period is the average time between two precipitation events for which a certain threshold is surpassed, based on historical data. Generalized extreme values (GEV) statistics are used to determine return periods for long events (> 12 hours). For the short events (<12 hours) generalized logistic distribution (GLO) statistics are used. In general, GLO results in larger extreme values (STOWA report 2019-19, 2019). Different methods are used for different even durations after it was concluded that the GEV method resulted in the underestimation of precipitation statistics for short precipitation events. This was especially apparent for events with long return periods.
Table 1 illustrated the statistics for the Netherlands in rounded numbers (this is recommended when used for design). It shows the precipitation amount for different return periods and different precipitation durations. In Chapter 3 Modelling drywell infiltration, the effect of the design storms on the functioning of the IDW is further discussed.

Table 1: Precipitation amounts (rounded to whole millimetres) for different return periods from 10 minutes till 8 days, based on STOWA 2019-19 (2019).

T [yrs]	10 min	30 min	60 min	2 hrs	4 hrs	8 hrs	12 hrs	24 hrs	2 days	4 days	8 days
0.5	8	10	13	15	19	22	25	30	39	50	68
1	10	14	16	20	23	28	31	37	46	59	79
2	12	17	20	24	28	33	37	44	54	69	91
5	15	21	26	31	36	42	45	54	66	81	105
10	18	25	31	37	43	49	53	63	75	92	116
20	20	30	37	44	51	58	62	73	85	102	127
25	21	32	40	47	54	61	65	76	89	106	131
50	25	38	48	57	65	73	77	87	100	117	142
100	29	46	58	68	78	86	90	99	111	128	152
200	33	55	70	81	89	95	98	112	124	140	163
250	35	58	75	87	94	100	103	117	129	144	167
500	41	70	91	105	112	118	120	132	143	156	178
1000	48	85	111	128	134	138	139	148	158	169	188

Well dimensions

As can be expected, the dimensions of an IDW greatly influence the functioning of that well. Generally speaking, the larger the diameter and length, the larger the area of contact between water and soil, and the larger the volume of the well. Even when the soil characteristics are not favourable for fast infiltration, the well can still be used for storage during the precipitation event. While the diameter of a well depends on the available sizes offered by the manufacturer, the well depth is limited by the groundwater level.

Output

Number of wells

The output of the design method is the required number of wells to be installed based on the different input parameters. These are based on a general connected surface area of 100 m^2 and can be multiplied to achieve the wanted total connected surface area. Unrounded numbers are used to leave the rounding of the required wells up to the user. For a complete explanation of the design method and its use see Chapter 5 IDW design method.

3 Modelling drywell infiltration

3.1 Objective

The created Hydrus 3D model is used to mimic the theoretical performance of IDWs. In the model the conditions of the functioning of the well can be altered. This forms the basis for the design method. So, the objective of the modelling is to create a design method based on the empirical functioning of IDWs in a 3-dimensional geohydrological model and the in practice functioning of wells.
In order to compare the theoretical functioning to the practical functioning of IDWs, falling head tests will be simulated in Hydrus 3D. These simulations are compared to field experiments (Chapter 4 Field and lab experiments).

3.2 Model set-up

In the first stage of the research, a general IDW model was created in Hydrus 3D. Hydrus 3D is a geohydrological software program developed by the company pc-progress. It is used for simulating water, heat and solute movement in two- and three-dimensional variably saturated media (Simunek et al., 2016). The build-up of the Hydrus 3D model is defined in this part.

3.2.1 Geometry

The general model consists of a cylindrical geometry with a radius of 5 m and a depth of 10 m. The geometry consists of 3 interlinked solids and an opening in the centre which represents the drywell. These separate solids have increasing radii to enable mesh refinements for different parts of the model. The reasoning behind this is the fact that closer to the well, the flow of water through the individual mesh parts requires additional refinement to run calculations. Also, the flow is calculated in a more accurate and precise way for a finer mesh. The geometry of the model is illustrated in Figure 6.

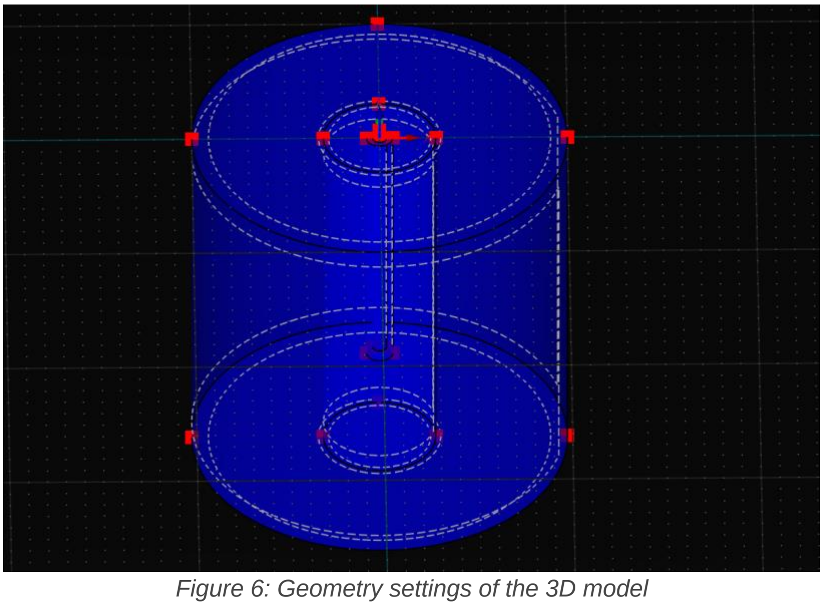


Figure 6: Geometry settings of the 3D model

In practice the interface between the soil and the water from the well is not at the boundary of the well but slightly outside the wells. This is caused by the geotextile that is wrapped around the pipes to block sediment inflow in the well. The outer wall of the well consists of a wavy surface with openings to enable water flow. The protrusions on this outer surface determine the position of the geotextile in relation to the well. This results in the interface between water and soil being 3.5 cm outside the inner wall. For this reason, the radius of the drywell in the model is increased by 3.5 cm, to properly indicate the interface between soil and well. In Table 2 the radii and FE-refinements for the model and of each of the three solids in the model are given for a well with a diameter of 500 mm. For the other diameters only the inner radius of the first solid is different. For D = 300 mm this is 0.185 m and for D = 800 mm this is 0.435 m. For more explanation on FE-refinements see the next section.

Table 2: Geometry settings of the 3D model for a well with diameter = 500 mm

Solid #	1	2	3
Inner radius	0.285 m	0.5 m	1.5 m
Outer radius	0.5 m	1.5 m	5 m
FE-refinement size (S)	0.1 m	0.2 m	0.5 m

3.2.2 Spatial and temporal discretisation

From the created geometry the mesh can be generated. This is an automated process based on the refinements of the different elements of the model. To obtain solutions at specific time intervals, numerical methods usually subdivide the time and spatial coordinates into smaller sections. In this way, the continuous process, described by partial differential equations, can be replaced with a set of algebraic equations to determine flow at every timestep. Most numerical methods, such as those used in the Hydrus program, use time-stepping between the initial condition and the end simulation. For the spatial coordinates, Finite Elements (FE) are used, while for time Finite Differences are used to solve the equations. The concept of FE-mesh is a type of design that splits a flow region of three-dimensional problems into tetrahedral, hexahedral and/or triangular prismatic elements formed by the nodes of the geometry.

The size of such an element (S) is determined by the refinement settings. In Figure 7 the settings and layout of the model FE-mesh are illustrated. In Table 2 the FE-refinement settings of the geometries are included.

The time discretization is done using an implicit (backward) finite difference scheme, for (un)saturated conditions. Since this scheme is highly non-linear an iterative process is necessary to come to a solution for the global matrix equation at every time step (Simunek, van Genuchten & Sejna, 2011). For each iteration, a system of linearized algebraic equations is derived that is solved using either Gaussian elimination or the conjugated gradient method.

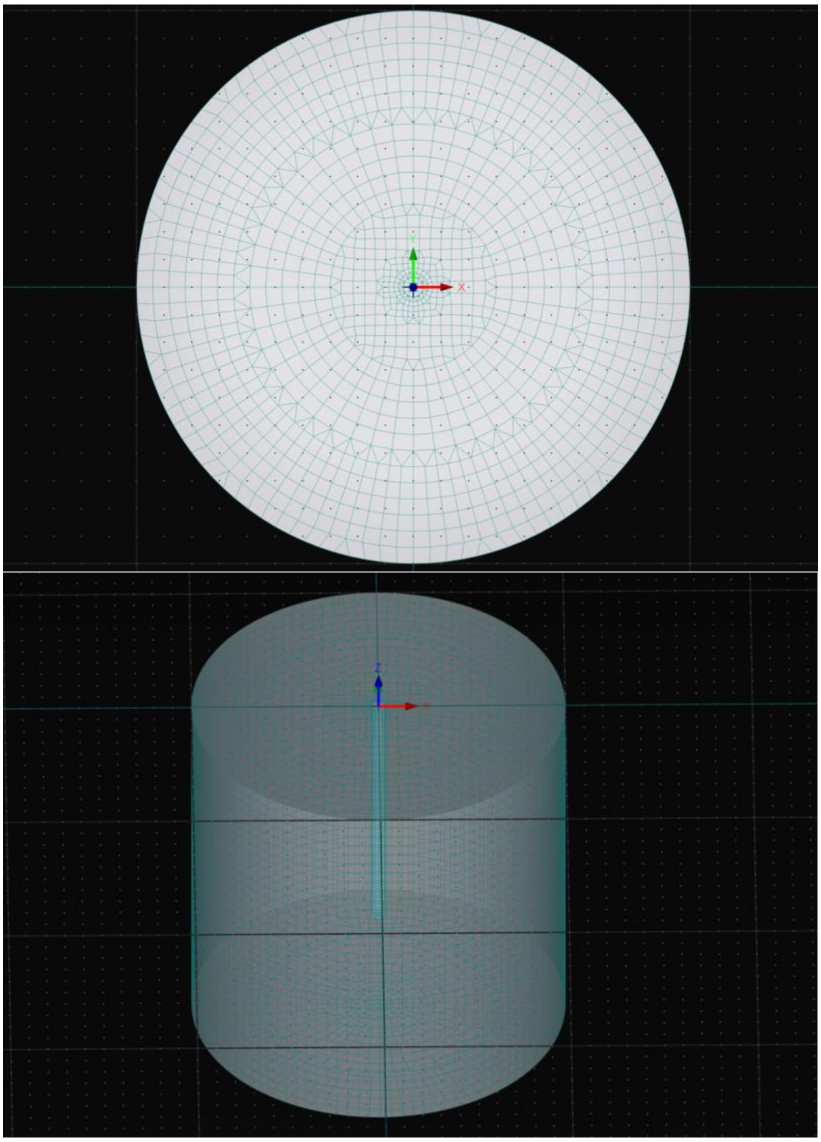


Figure 7: Top: close up top view of the generated FE-mesh. Bottom: overview of the mesh of the entire model

To converge to a solution when numerically solving differential equations the spatial and temporal conditions have to be set accurately. To this end, the Courant-Friedrichs-Lewy (CFL) convergence condition is used in mathematics. The CFL stability criterion number, or Courant number (C), is the ratio of two lengths, namely the fluid distance (velocity x time step) and cell distance. The determination of the Courant number in three dimensions is shown in Equation 1.

$$C = \frac{u_x \, \Delta t}{\Delta x} + \frac{u_y \, \Delta t}{\Delta y} + \frac{u_z \, \Delta t}{\Delta z} \tag{1}$$

If the fluid distances are larger than the cell distance, or C larger than 1, the equations will not converge to a solution. The conditions are set in such a way that C < 1.
To set the right conditions, the initial time step should be set to a low value, so the software can make small enough steps to keep C below 1. The downside is a long calculation time due to small time steps. To this end, the Hydrus software uses adaptive time steps which enable it to enlarge the time step to decrease calculation time. When the time step is too large, and C > 1, the software simply decreases the time step (with a minimum of the initial time step which is set beforehand), to enable the convergence of the solution. To ensure

proper C values the initial time step was set low at 1E-6 seconds. This enabled the software to calculate the infiltration for the well for different parameter settings

3.2.3 Domain properties

In the domain property settings, the water flow parameters are set. These parameters are subdivided into material properties for water flow for direct and inverse problems. For this research, a direct approach is taken since the runs should predict the water flow. In Hydrus (2D/3D) Richard's equation *(2)* is used to describe water flow through soil:

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial x_i}\left[K\left(K_{ij}^A \frac{\partial h}{\partial x_j} + K_{iz}^A\right)\right] \tag{2}$$

With θ being the volumetric water content [-], t being unit of time [T], x_i being coordinates [L], K being the unsaturated hydraulic conductivity [$L^3L^{-2}T^{-1}$], and K_{ij}^A being a component of dimensionless hydraulic conductivity anisotropy tensor. To solve Equation (*2*), the Van Genuchten (1980) closed-form equations are used, which in turn use the statistical pore-size distribution model of Mualem (1976). Table 3 includes a short description of these parameters. The general settings of the Van Genuchten parameters of Hydrus 3D used for the model are listed in Table 3. The parameters used for sandy soils in Hydrus 3D are based on Carsel and Parrish (1988). The parameters are the input for the soil hydraulic model of Van Genuchten (1980). The formulas used in this model to describe water flow are the following:

$$\theta(h)\begin{cases}\theta_r + \dfrac{\theta_s - \theta_r}{[1 + |\alpha h|^n]^m} & h < 0\\ \theta_s & h > 0\end{cases} \tag{3}$$

$$K(h) = K_s K_e^l [1 - (1 - S_e^{l/m})^m]^2 \tag{4}$$

$$m = 1 - \frac{1}{n}, n > 1 \tag{5}$$

$$S_e = \frac{\theta - \theta_r}{\theta_s - \theta_r} \tag{6}$$

Table 3: Description of the van Genuchten parameters

Parameter	Description	Set value
θ_r	Residual soil water content [-]	0.045
θ_s	Saturated soil water content [-]	0.43
α	Empirical parameter α in the soil water retention function [L^{-1}]	14.5
n	Empirical parameter n in the soil water retention function [-]	2.68
m	Empirical parameter m in the soil water retention function [-]	Varying
K_s	Saturated hydraulic conductivity [LT^{-1}]	Varying
$K(h)$	Unsaturated hydraulic conductivity at pressure head h [LT^{-1}]	Dependent on K_s
l	Tortuosity parameter [-]	0.5
h_s	Air-entry value [L]	Dependent on water content
S_e	Effective water content [-]	Varying

3.2.4 Initial conditions

In the initial conditions section, the pressure head or water content distribution of the model can be set. These are both units for the soil moisture. When the soil is increasingly saturated both the water content and pressure head will increase (the suction head will decrease). The effects of parameter pressure heads are examined. In Figure 8 the effect of varying pressure heads on the infiltration of water into the soil is illustrated. A couple of scenarios were run with varying initial conditions to visualize the effect of these adjustments on the graph for a falling head experiment.
Because the depth of the groundwater table is unknown for the field experiments, this depth is varied over different runs of the model. The groundwater table interface is determined in the model by the depth at which a head of 0 m (atmospheric pressure) is present. This is the location where the soil is completely saturated. If the soil is not saturated it will have a 'negative' pressure (or suction head) which means the pressure and so, pressure head, is below the atmospheric pressure.

In the initial condition section, the hydraulic head or soil moisture content can be set for the whole domain. The options are:
1) constant: homogenous distribution over the soil profile.
2) equilibrium from the lowest local nodal point: only the bottom condition is set, and equilibrium conditions are applied to the soil above.
3) and linear distribution with depth: the top and bottom settings are set, with a linear distribution in between.

The settings were varied to illustrate the result of a simulated falling head test for different initial conditions. The graph below clearly shows that the variations in initial conditions do not cause a single variation in the falling head test data. All the simulations give the same falling head results. The water table depth does not have a significant impact on the falling head test simulations as long as it is below the bottom of the well. This can be explained by the fact that sand has low sorptivity. This means water cannot attach easily, or sorb, to the sand particles causing the water to drain under gravity rather easily.
For the remainder of this research the water table is set to be 1 m below the bottom of the well with a linear distribution over the depth.

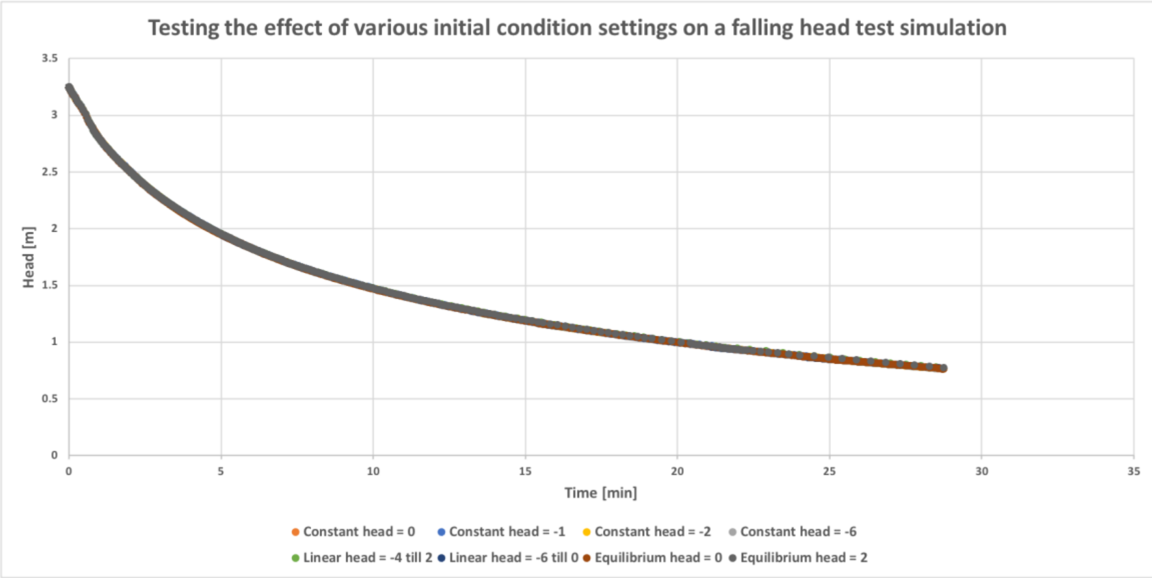


Figure 8: Effect of changing the initial conditions on model falling head performance

3.2.5 Boundary Conditions

The boundary conditions determine the flow of water through the boundaries of the model. In Figure 9 the selected boundary conditions of the model are visible. The top plane is set to no flux. This is to solely review the functioning of the well in the subsurface and not the additional infiltration of precipitation from the surface. Besides, in urban areas, the surface is also largely impermeable, so it is expected that surface infiltration does not have a considerable effect on the infiltration from the IDW. The sides of the cylindrical domain are set to a constant head, meaning the water table is constant. Hence, the inflow of water into the water table will result in an outflow through the sides. As such, to prevent a rapid rise in the water table. This would happen when the domain is regarded as a storage volume, with no outflow through the sides of the domain.
Lastly, the sides of the well are set to seepage face. This enables the infiltration of water from the well into the surrounding soil.

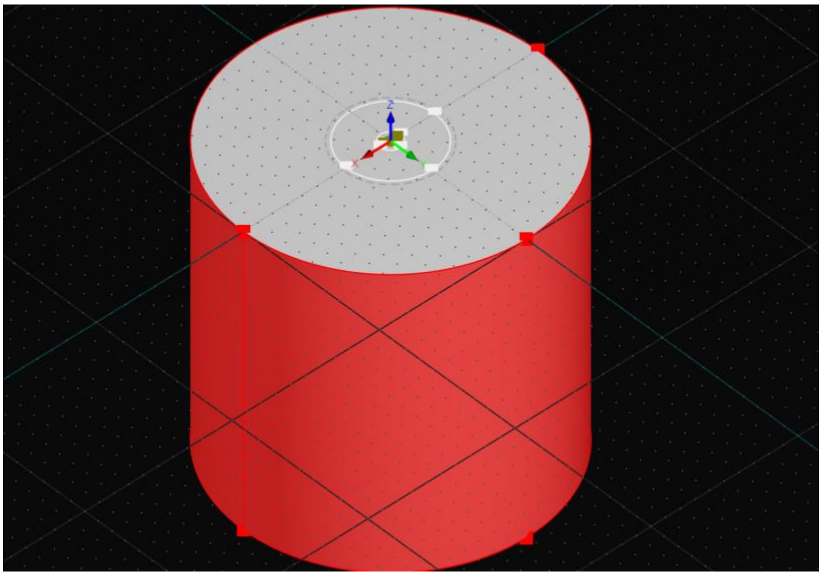


Figure 9: Boundary condition settings of the 3D model

3.2.6 Scenarios

In compliance with the literature study on the important parameters of IDW performance, the effect of variation in diameter, depth and hydraulic conductivity will be simulated. The falling head experiments that are conducted include variations of the different parameters over different simulation runs to assess the performance of hypothetical IDWs under different circumstances. Since the initial water content of the soil does not have a large impact on the functioning of the wells, it was set constant over the different runs. The saturation of the soil around the well however did result in variation in the functioning of the wells. Since saturated conditions are favoured, the area around the well will be wetted first. This is done by keeping the water level in the well constant for 10 minutes, after which the falling head test is performed. The soil type and porosity are also constant over the different runs since the goal is to get general data on the functioning for sandy soils. The other main parameters are explained further.

Hydraulic conductivity

Starting with the variation of the hydraulic conductivity, since it is expected to be one of the major soil characteristics for the design method. For this reason, the saturated hydraulic conductivity value was set to 0.1, 0.5, 1, 2, 5, 10, 15, 30 or 50 m/d for different simulations. These values were used to get a wide variety of hydraulic conductivity values for sandy soils. This is because the hydraulic conductivity of sand shows larger variations in general, see Table 10.

Depth

Since the calculation of the simulations of different wells can take up to 120 hours for a single run, the finalized model will consist of a large well of 7 m deep. The infiltration pipes used in practise have a maximum length of 6 m. The additional metre in the model is used to wet the soil around the well as a spinoff period to 'warm up' the model, prior to the falling head test. In this way, the behaviour and performance of wells of different depths can be determined. It is expected the behaviour and performance will not differ from simulating wells of different depths in different models since the area around the well is already saturated. To give an example, the performance of a well with a depth of 2 m is obtained by observing the falling head test from the moment it reaches a water level depth of 2 m. In this way, the performance of hypothetical wells with different depths (observed for every 0.5 m) can be obtained with a single simulation. This approach greatly lowers to run time of all the simulations.

Diameter

The diameters of infiltration pipes used for IDWs come in three different sizes, namely: 300 mm, 500 mm and 800 mm. These are also the diameters that are used in the simulations. This is because the purpose of this research is not only to investigate the effect of diameter variation on the functioning of a well, but also to use the diameter as an input parameter in the design method.

The hydraulic conductivity is varied for 9 values while the diameter varies over 3 different diameters. In total 27 different sets of parameters are simulated. With these 27 simulations, the falling head tests for 12 different depths are observed, resulting in the theoretical performance of 324 IDWs.

3.2.7 Assessment framework

The performance of the wells is assessed by the drain time. This is the amount of time necessary to completely drain a well. In practise an IDW should be drained relatively quickly in order for it to be a valuable asset. If the IDWs drain too slowly it cannot be used to cover consecutive rainfall events and stagnation of water in the well can occur. Especially keeping in mind that clogging will occur over time, expected slow draining wells are unfavourable. In order for IDWs to be useful in urban areas this research persists on a threshold of 75% drained water after 45 minutes after the start of the falling head test. This because it is a supposed midway number between 50% and 100%. With 50% draining being too slow and 100% being too strict. Besides, assessing after 30 minutes might be too short of a time frame while 60 minutes might be too long between consecutive rainfall events. Concluding, the water level should be decreased at least 75% in 45 minutes for a well to be redeemed useful.

3.3 Results

3.3.1 Main parameters

Hydraulic conductivity

In Figure 10 the falling head tests for different saturated hydraulic conductivity values are illustrated, for a well diameter of 500 mm. The falling head graphs for 300 mm and 800 mm are included in Appendix A. The effect of hydraulic conductivity (k) is visible. In the case of k = 50 m/d the well would be emptied in about 10 minutes, while a well with k = 0.1 m/d would only see a total drop of the water level 1.25 m after 1 hour.

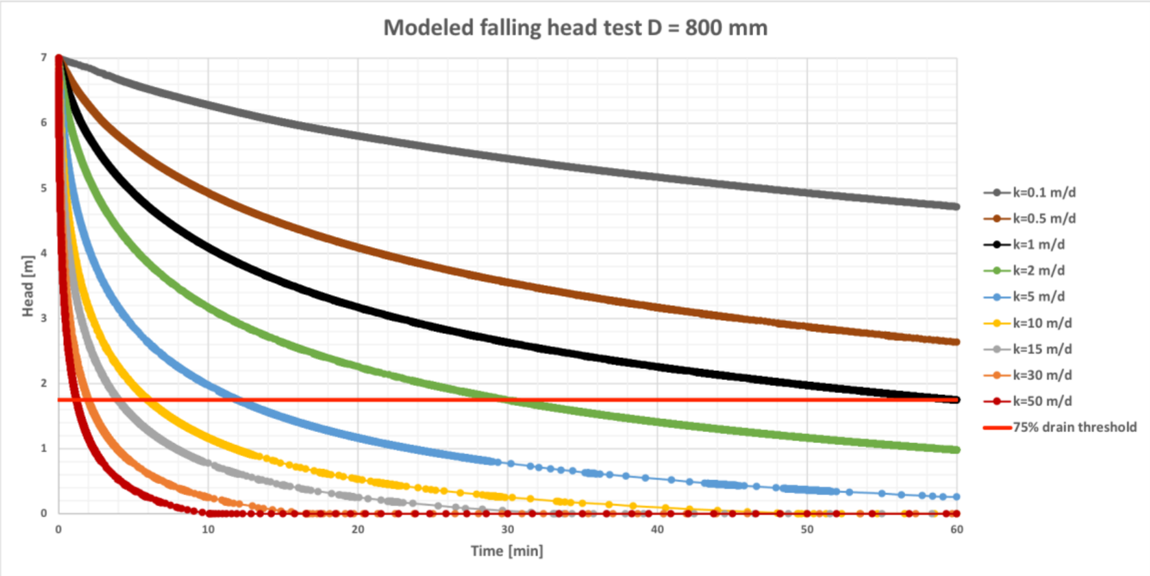


Figure 10: Falling head performance for different hydraulic conductivity values (k) for a well diameter of 800 mm and a run time of 1 hour. The red line represents the 75% drain threshold. The falling head graphs (of hydraulic conductivity values) that are above this line at 45 minutes do not meet the assessment criteria.

Depth

In Figure 10 also the effect of the depth of a well is visible. Since the head in the well decreases with the water depth and the infiltration is dependent on the head, the IDWs will drain more slowly as the water level drops. For shallow wells, the falling head rate is lower than for deep wells. Furthermore, in deep wells, there is a larger contact area between water and soil. So, even though drain time is longer for deep wells, the total drained volume is also larger. This also illustrates the profound effect of the depth on the amount of stormwater that can be infiltrated by a deep well. While a deep well has a longer drain time, a large amount of water can be stored and infiltrated.

Diameter

Another important parameter is the diameter of the well. The effects of the well diameter on the falling head tests are illustrated in Figure 11 which shows the falling head graph for

3 different diameters and 3 different hydraulic conductivities. A larger diameter increases the area over which the water can infiltrate but at the same time, it also greatly increases the volume of the well. This causes the water level in wells with smaller diameters to decrease more rapidly because they contain a smaller volume to drain. This can also easily be demonstrated with a simple calculation. Looking at the falling head tests for hydraulic conductivity of 1 m/d, the water level for a diameter of 300 mm drops from 7 to 0.5 m. For a well with a diameter of 800 mm, the water level drops to a depth of 1.8 m within that same hour. In the well of 300 mm, this means 0.46 m^3 of water is infiltrated while in the well of 800 mm about 2.61 m^3 is infiltrated. This can also be explained by the difference in the hydraulic radius. This is the ratio between the cross-sectional area and the wetted perimeter, see Equation (7). For a lower hydraulic radius, the well drains faster.

$$R_H = \frac{r}{2} \tag{7}$$

Concluding, the water level in smaller wells decreases rapidly in comparison to larger diameter wells but a smaller volume of water is eventually infiltrated.

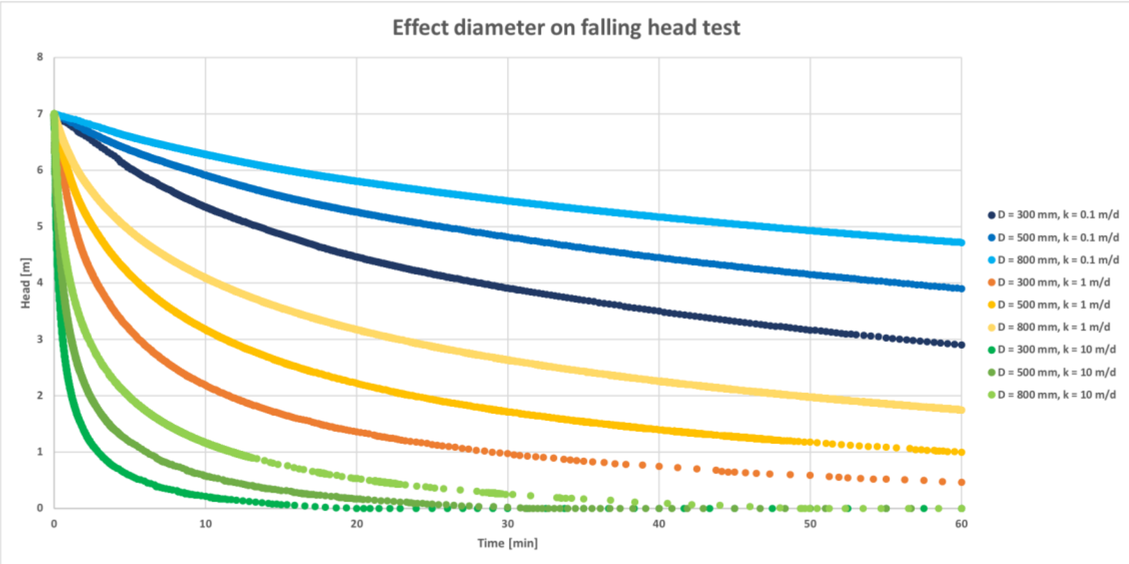


Figure 11: Effect of varying the diameter on falling head test, illustrated for 3 different hydraulic conductivity values

Soil water content

Lastly, the effect of wetting the soil around the well showed that the velocity of the falling head decreases when the soil is saturated more. This proves the predictions made in Chapter 2 Infiltrating stormwater, that the infiltration rate decreases when the soil is increasingly saturated. To use saturated conditions in the final model the soil is saturated prior to the falling head tests by keeping the water level in the well constant for 10 minutes. This approach is similar to the one applied during the falling head experiments in the field, explained in Chapter 4 Field and lab experiments.

3.3.2 Assessment

The results of the assessment of the performance of the wells are shown in Table 4. The wells are assessed on the capability of decreasing the water level by 75% within 45 minutes.

Based on this assessment it is clear that the wells in soils with very low hydraulic conductivities (< 1 m/d) frequently do not meet this threshold. Furthermore, soils with low hydraulic conductivities (1 – 2 m/d) are sufficiently suitable but only if the wells are deep.

This can be seen in Figure 10, and similar graphs for the other diameters (Appendix A), where the graphs level out for lower hydraulic heads. For hydraulic conductivities of 5 m/d and up the levelling out of the graphs is to a lower degree, indicating that even wells with smaller depths still see a significant drop in the water level when the hydraulic conductivity is high.

Table 4: Emptying of a well over the course of 45 minutes

k_{sat}	D = 300 mm [%]	D = 500 mm [%]	D = 800 [%]
0.1	47.3	33.2	21.3
0.5	79.5	66.3	51.3
1	89.3	79.2	65.7
2	95.8	89.2	79.0
5	100	98.0	92.7
10	100	100	100
15	100	100	100
30	100	100	100
50	100	100	100

3.3.3 Possible connected surface area

Based on the performance of the theoretical IDWs, and a design storm, the possible connected surface area of a single well can be determined. In turn, this can be used to determine the number of IDWs necessary to provide proper drainage for a specific surface area. This data is desired for the creation of the generic design method. In this research, the performance of IDWs is analysed for three different design storms. This is because the rainfall intensity greatly determines the performance. To this end three different design storms are used: an intense rainfall event of 21 mm/10 min, a medium intense rainfall event of 40 mm/1 hour, and a spread-out rainfall event of 66 mm/12 hours. These rainfall events all have a return period of once every 25 years, based on the precipitation statistics. This return period was selected to create a design method for IDWs that is based on rainfall intensities that do not have a frequent reoccurrence. It is expected that this enables a robust design of the IDWs in practice.

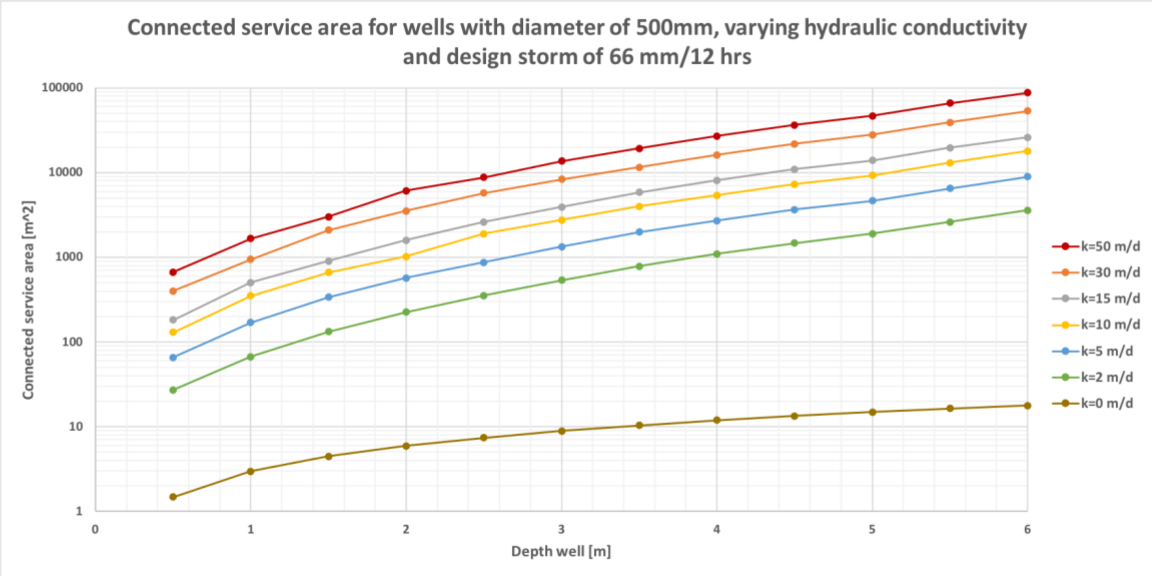
For the calculation, some assumptions are made. First of all, it is assumed the infiltration rate does not change. This means there is no decrease in infiltration rate due to part of the soil being unsaturated. Luckily this is already accounted for in the model simulations by saturating the soil around the wells. Second of all, during the precipitation event, the water level in the well is considered to be constant in such a way that the entire well is filled with water. Since the infiltration rate is not decreasing, the inflow into the well should be equal to the outflow of water infiltrating from the well into the soil. This also means the rainfall rate on the connected surface area is constant and not greatly varying during the event. In reality, it is unlikely that the rainfall rate during a storm is constant. The intensity during a rainfall event usually varies but unfortunately, it would be very time-consuming to simulate an entire rainfall event for every possible IDW depth, diameter and hydraulic conductivity. This is beyond the scope of this research. Lastly, this means the precipitation amount is averaged out over the duration of the design storm in order to make the calculation possible. This is called block rainfall and enables simple calculations with design storms (Butler & Davies, 2000), see Section on Precipitation amount.

The way this is calculated is as follows. With the data of the model, the rate at which the water level drops for every combination of diameter and depth of the well and the hydraulic conductivity of the soil is known. This corresponds to the amount of water that infiltrates for a given timestep of a filled IDW. This amount of infiltrating water is extrapolated to the

total duration of the design storm and so, the total amount of infiltrated water is determined. This is added to the amount of storage in the IDW that is possible which results in the total amount of stormwater that can be processed by the specific IDW, for a given time. This number (L^3) in turn is divided by the precipitation amount ($L^3\,L^{-2}$) of the design storm which results in the total possible connected surface area (L^2).
This calculation was done for 3 different design storms, based on precipitation statistics of the Netherlands.
Figure 12 depicts 2 graphs for possible connected surface area for IDWs with a diameter of 500 mm, based on 2 design storms. The vertical axis represents the connected surface area for a single well. On the horizontal axis, the well depth is given, and the different lines represent the different average hydraulic conductivities of the soil. Complying with the results of the well performance assessment the low hydraulic conductivity soils (<2 m/d) are not taken into account.

Figure 12 shows an additional line. This line represents the possible connected surface area if the hydraulic conductivity would be equal to 0 m/d. This means it represents the surface area for a well that is solely used for storage. All the lines above it show the added value of using infiltration. Since the y-axis has a logarithmic scale, this added value of using infiltration for stormwater management is large. Especially for soils with high hydraulic conductivity an immense amount of extra area could be connected in comparison to a well solely used for storage or a well installed in low hydraulic conductivity soil.

Since the graphs in Figure 12 depict the calculated surface area per well for different design storms, the possible connected surface area differs greatly. Because of the assumption that the precipitation rate is constant, the short intense rainfall event results in smaller surface areas than the longer events. The possible connected surface area graphs for all the diameters and design storms are included in Appendix B.



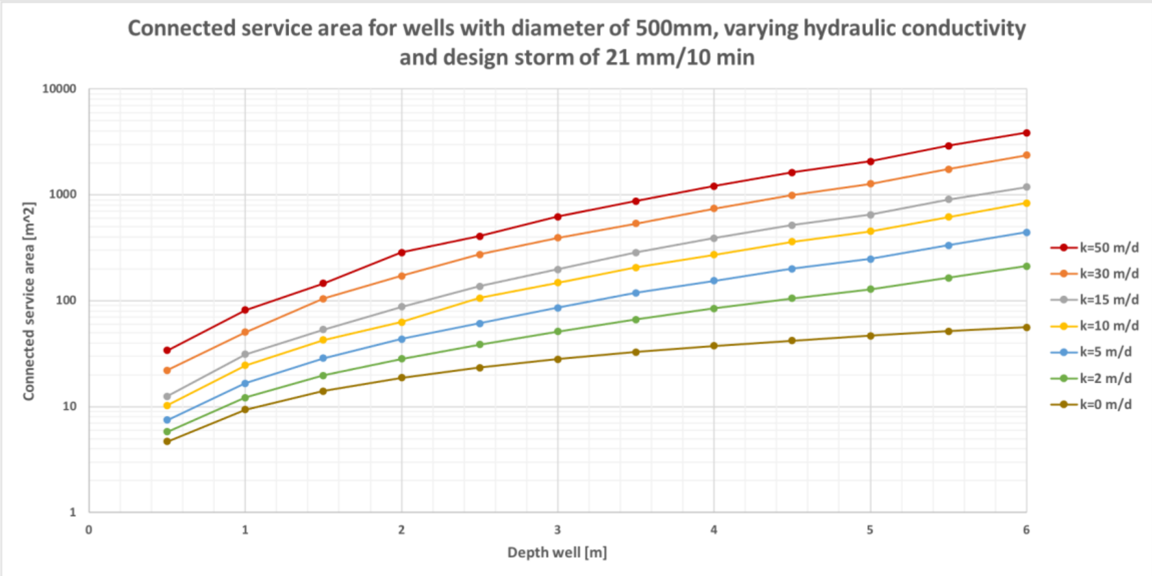


Figure 12: Possible connected surface area for wells with a diameter of 500mm. Top: for a design storm of 66 mm in 12 hours. Bottom: for a design storm of 21 mm in 10 minutes.

The resulting data on connected surface area per single well, is used to determine the number of IDWs necessary to connect a certain area. So, for different surface areas, for every combination of well parameters and soil characteristics, the required number of wells is determined. Table 5 shows an example for IDWs with a depth of 3 m and a diameter of 500 mm. Depending on the soil hydraulic conductivity, the total necessary number of wells for varying connected surface areas can be read from the table. This was also done for all the other wells. This information will be incorporated in the design method.

Table 5: Required number of IDWs per connected surface area, with d = 3 m and D = 500 mm

K [m/d]	50 m^2	100 m^2	250 m^2	500 m^2	1000 m^2
50	1	1	1	1	2
30	1	1	1	2	3
15	1	1	2	3	6
10	1	1	2	4	7
5	1	2	3	6	12
2	1	2	5	10	20

3.3.4 Minimum well distance

To determine the minimum distance between wells, without a well influencing the water flow out of another well, the spread of the wetting front around the wells is analysed. To do this the radius at which the soil is 50% saturated in the simulations is located. At this location, the wetting fronts of two adjacent wells meet theoretically, without obstructing the outflow from both wells into the subsurface. The results of the anisotropy simulations that are mentioned in Chapter 6 Discussion, indicate the spread of the wetting front around the well. The results are gathered in Table 6. This table includes the outermost radius of the wetting front at the depth of the bottom of the well and the absolute minimum distance required between adjacent wells of this diameter. Since the minimum distance can vary depending on deviating soil characteristics, the recommended minimum distance between wells (right column) is slightly larger to ensure a larger safety margin.

Table 6: Information on minimum distance between IDWs for design

Diameter well [mm]	Radius of wetting front [mm]	Minimum distance between wells [m]	Recommended distance between wells [m]
300	675	1.35	2
500	1005	2.01	3
800	1415	2.83	5

3.4 Conclusion on modelling drywell infiltration

The model gives a sound theoretical representation of the functioning of IDWs. The geohydrological model can be used as a basis for the generic design method. This will be discussed in more detail in Chapter 5 IDW design method. Concluding from the simulations with the Hydrus model the most important parameters are the dimensions, soil water content and hydraulic conductivity. This information on the theoretical functioning of the wells is used to compare to the practical functioning, as explained in Chapter 4 Field and lab experiments.

The effect of the diameter on the functioning of the well is visible in Figure 11. Wells with a smaller diameter drain faster than wells with a larger diameter. Due to the shorter drain time the smaller diameter wells are more practical to process incoming precipitation. In this sense, it would be wiser to install a large number of small wells than a few wells with a large diameter. However, this is dependent on the desired functioning of the IDWs. For soils with high hydraulic conductivity, the wells would drain relatively fast independent of the diameter of the wells.

For the well depth, this is slightly different. While deeper wells do have a longer drain time than shallow wells the use of deep wells is still encouraged, provided that the groundwater table depth allows this. This is because the deeper the well, the larger the contact area between water in the well and the soil. This of course is also true for the diameter, but the storage volume increases more than the contact area between water and soil. For this reason, the drain time greatly increases for larger-diameter wells. This is not the case for deeper wells. The increase of contact area with increasing depth is larger than the increase in contact area for increasing diameter. Besides, due to a deeper well the water level in the well is also greatly increased, and so is the hydraulic head. This leads to a faster infiltration rate of the water from the well, simply because of a larger head, creating a larger gradient between the water in the well and water in the soil. The shallow wells did also not meet the threshold values for the assessment framework. For these reasons, shallow wells (< 2 m of depth) will not be included in the design method.

The hydraulic conductivity has a profound effect on the functioning of an IDW. Since the hydraulic conductivity is an indicator of the ease at which the water flows through the soil, it is a parameter of great importance for the IDW design. The larger the hydraulic conductivity, the faster the water generally infiltrates into the soil and the better the IDW functions. Low hydraulic conductivity values (<2 m/d) will not be included in the design method since IDWs would only be effective if they are very deep. To include these low values would make the design method increasingly complicated.

Lastly, the soil water content surrounding the wells does influence the functioning of the well, but this can be accounted for in the 3-dimensional geohydrological model. For this reason, it is not accounted for or included in the design method.

Conclusion from the simulations, the most important parameters that should be included in the design method are the depth and diameter of the well, the hydraulic conductivity of the soil and the connected surface area.

4 Field and lab experiments

4.1 Research area

To assess the practical performance of IDWs a case study area is used. This research is done in cooperation with the municipality of Hilversum. For the practical part of the research, IDWs were tested and samples were taken at locations in Hilversum. The sampling and the experiments with the wells are discussed in more detail in this chapter and the corresponding locations of samples and wells can be found in Figure 14.
The Hilversum urban area is situated at the Utrechtse Heuvelrug, a push moraine that was formed during the Saalian glacial around 150.000 years ago. This hill ridge consists of alluvial sand deposits of the Rhine and Meuse rivers. According to the geological survey of the Netherlands (TNO), most of the Hilversum subsurface consists of fine to coarse-grained sand. Moreover, in multiple areas of the city the groundwater table is situated relatively deep below the surface (4-5 m deep). This combination makes for favourable conditions for the implementation of IDWs. Figure 13 shows the location of the urban area within the Netherlands.

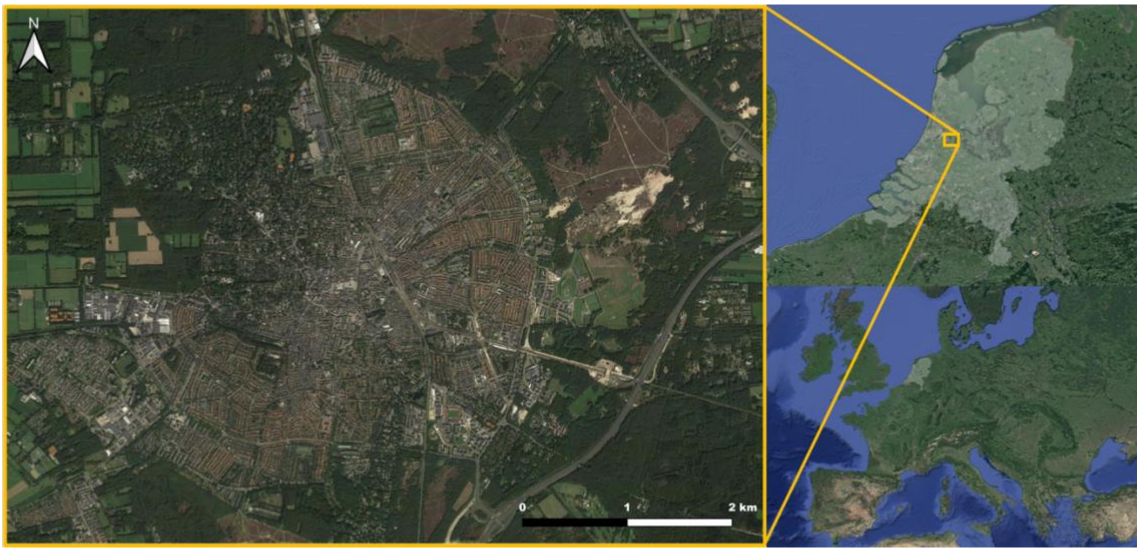


Figure 13: Location of the city of Hilversum in the Netherlands and Europe.

4.2 Objective

In order for the design method to be useful in practice, the model simulations have to be compared to the practical functioning IDWs. This is because the field conditions will be input parameters for the design method if it is being used in practice. The input parameters for the design method will be based on knowledge and experiments performed in a certain area. The hydraulic conductivity is an important parameter in the design process since it comprises the field conditions of a certain location or soil. Methods to determine the hydraulic conductivity in the field and additionally in the laboratory are reviewed and discussed in this chapter. The hydraulic conductivity of multiple samples was measured in the laboratory with a KSAT measurement device. This device is explained further in Section 4.4.1 KSAT measurements. The falling head experiments with 7 IDWs in Hilversum were performed to enable comparison to the simulated experiments and to

measure the hydraulic conductivity using the inverse auger hole method. All these experiments are further explained in the next sections.
Figure 14 depicts the locations of the IDWs within the urban area of Hilversum (in orange) and the location where soil samples were taken (in blue).

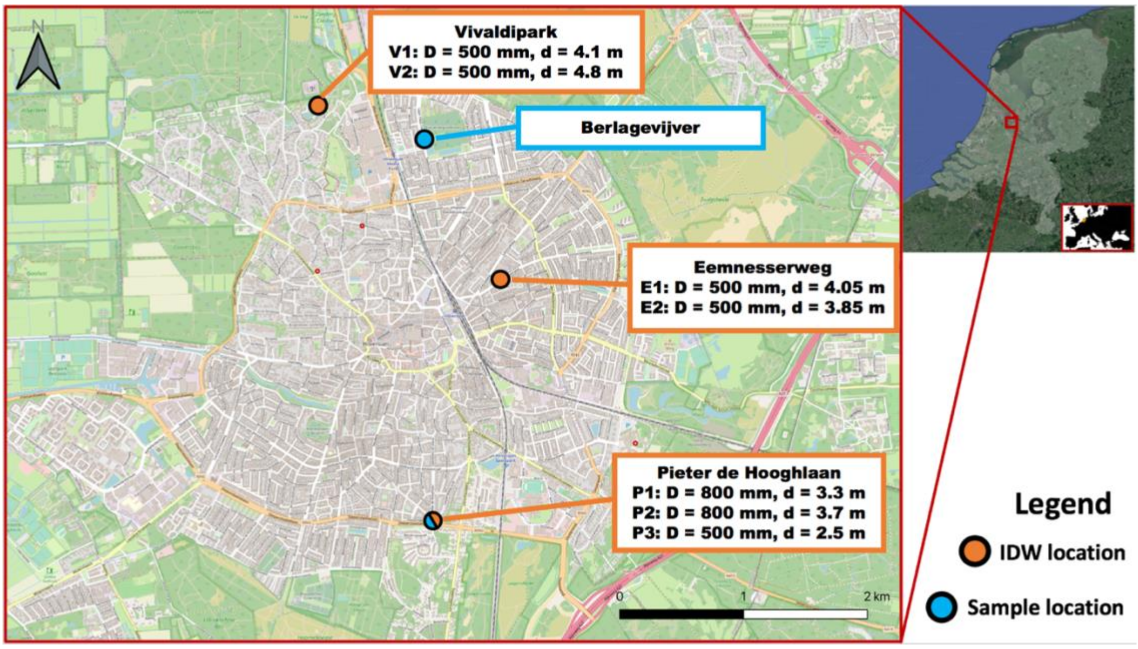


Figure 14: Location of the site where soil samples were taken next to Berlagevijver and the locations of the different wells used for the in practice falling head experiments

4.3 Hydraulic conductivity measurement methods

Generally, the saturated hydraulic conductivity (k_{sat}) is used in the design process of infiltration facilities. This is because for hydraulic design the facility is assessed on its functioning under saturated conditions. As in unsaturated soil part of the pores are filled with air, these pores will first be saturated completely after which the water can flow freely through the soil. Therefore, due to extra available pore volume, a hydraulic conductivity measurement in unsaturated soil results in overestimation. This can have severe consequences for the functioning of an infiltration facility design. If the infiltration rate of a facility is overestimated due to the measurement of the unsaturated hydraulic conductivity, the facility will frequently function insufficiently. To make matters even worse, the wells are performing the worst during intense rainfall events. Under these conditions the soil around the well will become saturated quickly, resulting in a decrease in infiltration rate. For safety reasons, it is therefore important to use the saturated hydraulic conductivity.
The measurement of the hydraulic conductivity of the soil was done in two different ways in this research. One is through laboratory experiments and one is through falling head experiments conducted with IDWs in Hilversum. The hydraulic conductivity is one of the most important parameters for the design of infiltration facilities (Sasidharan et al., 2020). The saturated conductivity of sand usually differs in horizontal and vertical directions (anisotropy), even for homogenous soils (Butler & Davies, 2000). In the 3-dimensional model this is not included to decrease the complexity of the model.

Both the field and lab measurements make use of Darcy's law to determine the soil hydraulic conductivity. In 1856 Henry Darcy performed an experiment concerning the water flow through a column of sand (Darcy, 1856). The setup of the experiment is shown in Figure 15.

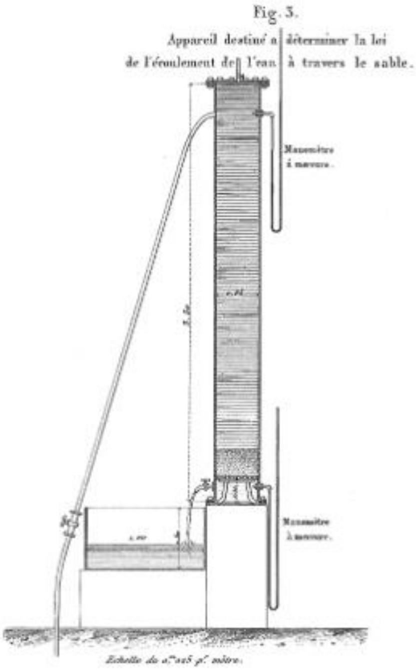


Figure 15: Experimental setup used by Darcy (1956)

A linear relation was found between the discharge of water through the soil column and the pressure gradient of the hydraulic head over the length of the soil columns. This empirical relation between water flow and soil characteristics is called Darcy's law. To come to this relation a constant was introduced, named the hydraulic conductivity k. This constant was defined as a coefficient of proportionality describing the rate at which a fluid can move through a permeable medium, depending on fluid and soil characteristics like density and permeability. Darcy's law states:

$$v = -k \cdot \frac{dh}{dl} \tag{8}$$

With *v* being the Darcy velocity or Darcy flux (alternatively called the specific discharge q) with *v* = *Q/A*, *k* is the hydraulic conductivity, *dh* the head loss over a certain length, *dl* the distance between 2 points where h is measured. This can be rewritten resulting in the following equation:

$$Q = -k \cdot A \cdot \frac{\Delta h}{L} \tag{9}$$

With *Q* being the amount of fluid flowing out of the column over time, *A* the area through which the water flows, *Δh* the hydraulic head or energy potential of the fluid in the column, *L* the length of the specific column of soil over which the change in h is measured.
This equation is used in a laboratory experiment to determine the saturated hydraulic conductivity of a soil sample. Furthermore, this relation forms the basis of the Porchet formula which is used to determine the hydraulic conductivity using falling head test data of an IDW. Both are further explained hereafter.

4.3.1 Laboratory KSAT measurements with soil samples

The KSAT device by Meter Group is an device that can accurately determine the saturated hydraulic conductivity (see Figure 17, right). A sample of 350 grams of dry soil is placed

in a ring container, see Figure 16. To mimic the physical circumstances (in the subsurface) of the sample the soil has to be compacted first. These containers are closed off by permeable lids that contain the soil inside but also promote the inflow of water into the soil. The containers are subsequently placed upside down in a vessel that is filled with demineralized water in such a way that the containers are not completely submerged under water, (Figure 17). This is done to prevent alteration in the sample due to water flow. Now the samples are contained for at least 24 hours to ensure they are completely saturated.
In the next step, the samples are placed in the measurement device in a watertight manner. Another lid is positioned on top which is attached to ensure solely flow of water through the sample. A falling head test is performed where the device measures the change in pressure, and so the water level in a water column. This indicates the water flow through the sample. In Figure 17 right, the setup of the measurement with the KSAT device is shown. On the left side of the figure, the water column is shown. From there the water flows through the bottom of the sample upwards out of the sample under gravity.

Figure 16: Preparing the sample containers

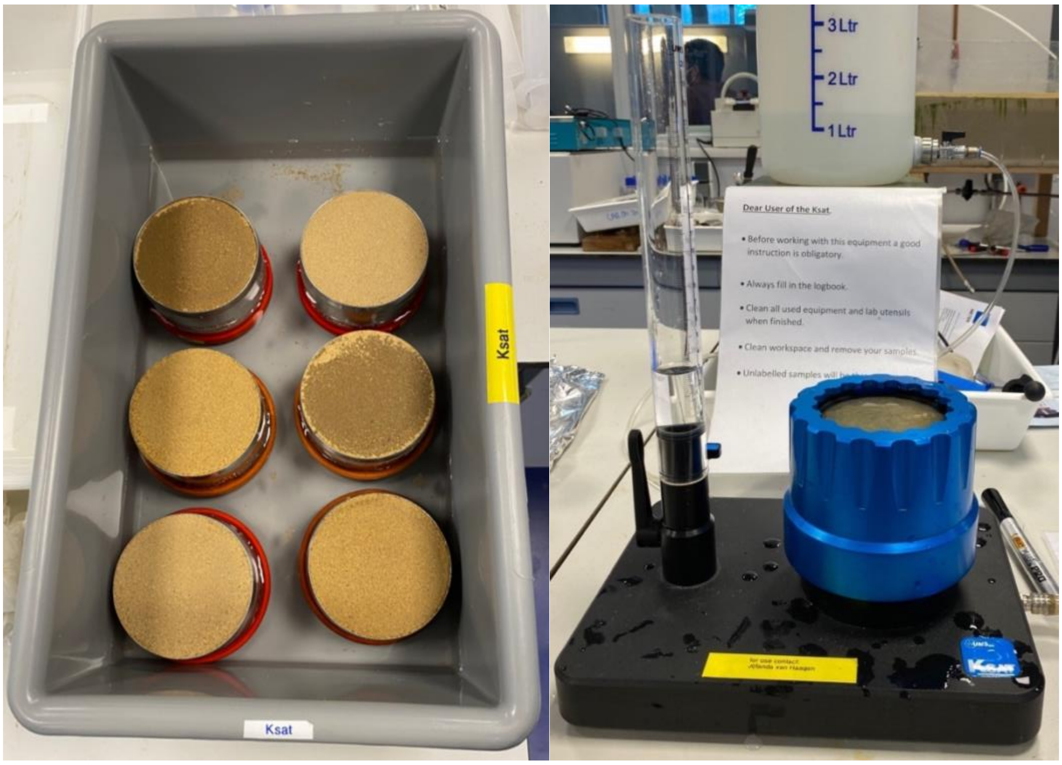


Figure 17: Left: samples placed in the vessel to saturate. Right: KSAT measurement device during falling head experiment

With the KSAT measurement device six samples from Pieter de Hooghlaan were measured (near P1, Figure 14). This is the location of three of the seven wells that were used for falling head experiments. At this location, a borehole was drilled from where six samples were extracted, from three different depths in the profile. In Figure 18 the soil profile from this borehole is illustrated. The profile seems to consist of generally homogenous soil.


Figure 18: Soil profile at borehole on Pieter de Hooghstraat

Next to the samples taken near the wells at the Pieter de Hooghlaan, five samples were taken at a site near the Berlagevijver. The location is indicated in Figure 14.

4.3.2 Falling head experiments

The 7 wells that were used for the falling head experiment are shown on the map in Figure 14. The map also contains information on the depth and diameters of all the wells.
The experiments were conducted by using a large water tank so to quickly fill the wells and perform the falling head tests. In this way, the infiltration from the well during a rainfall event can be mimicked. In the pictures in Figure 19, the experimental setup is shown for one of the seven IDWs in Hilversum. The water level was measured during the test using divers.

Figure 19: Images depicting the experimental setup

To test the functioning of the well under wet soil conditions, the water level in the well is kept somewhat constant for 20-30 minutes. After the soil around the well is assumed to be adequately saturated, the falling head experiment is conducted. For all wells multiple tests were conducted. The data of the last test for each well is used in the research since it ensures to soil around the wells was saturated to the largest degree.

4.3.3 Inverse auger hole method

Using the concept of the inverse auger hole method, it is possible to determine the hydraulic conductivity of the soil by the infiltration of water into the soil from a well, based on the falling head test. In this method, one important assumption is made. An additional experiment was conducted to test this hypothesis. This is further explained.

The inverse auger hole method is developed from the auger hole method that was first introduced by Diserens in 1934. Two years later Houghoudt improved the auger hole method (Hooghoudt, 1936), which was later improved again by several others. With this simple method, the saturated hydraulic conductivity can be determined by measuring the inflow in a borehole that is drilled in the groundwater table.
In 1979 Van Hoorn described the inverse auger hole method, which is a method to determine the hydraulic conductivity in the unsaturated zone, right above the groundwater table. This method, also known as the Porchet method, makes use of a volume balance and Darcy’s law.

To use the Porchet formula, it is assumed that the hydraulic gradient ($\frac{dh}{dl}$) in sandy soils is equal to approximately 1. This means that the wetting front in sandy soil will make an angle of approximately 45° in comparison to the horizontal plane (Ojha et al., 2018). In this way Darcy's law reduces to the following:

$$v = k \tag{10}$$

This was tested by the experimental setup in the lab which is shown in Figure 20. The setup consists of a container with two compartments, separated by filter material. In this experiment, the right compartment, which is transparent, is filled with soil sample material that was used for the hydraulic conductivity experiments, originating from the Berlagevijver, see Figure 14. The left compartment is filled with water up until a certain level. The water level is kept constant while the water infiltrates, through the filter, into the sand compartment. Here the water will flow through the sand and eventually drain through a second filter at the bottom of the right compartment. With this setup, the infiltration of water out of an IDW into the soil, and down to the saturated zone, is simulated. The hypothesis states that the wetting front will make an angle of 45° with the horizontal plane, as assumed in the inverse auger hole method.

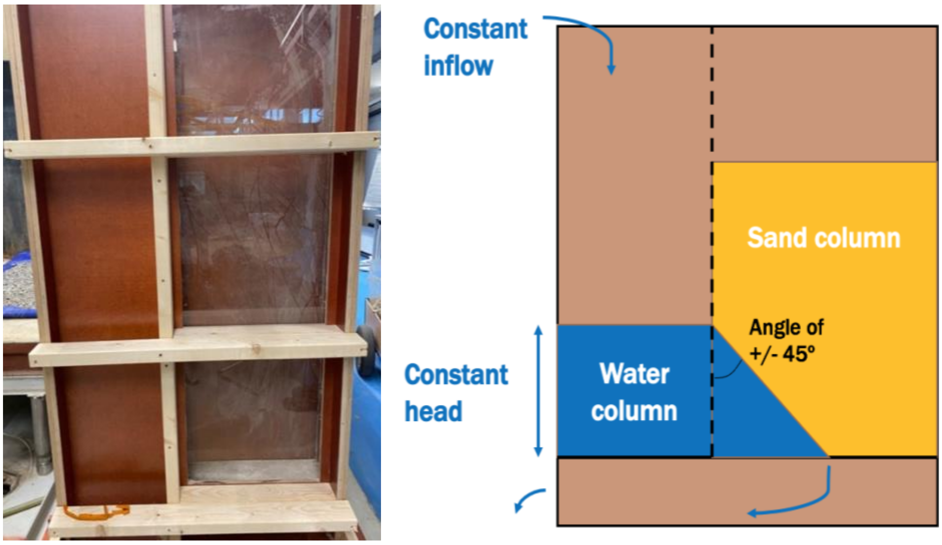


Figure 20: Set-up and schematic representation of gradient experiment

Figure 21 shows images that were taken during the experiment. It shows the resulting wetting front in the transparent container for the experiment setup. For clarity, a straight line was drawn on the glass that follows the wetting front of the water in the soil. In the right image of Figure 21, the angle of this straight line is illustrated. Strikingly, the wetting front of the water showed an angle of exactly 45°. It can be concluded that the hypothesis, which states a gradient of 1 in sandy soils, is correct.

Figure 21: Images depicting the wetting front during the experiment, including a ruler indicating the angle

With this knowledge, a simple volume balance can be made. This volume balance is used to find a relation between the hydraulic conductivity and the falling head in the well. With this relation, the hydraulic conductivity of the soil can be determined by the rate of the falling head in an IDW.

Since the gradient in sandy soils is equal to 1, it can be assumed that the infiltration velocity is equal to the hydraulic conductivity. It is stated that the discharge (Q) from a well is equal to the surface area of the well (A) times the change in head (dh) over time (dt). In other words, the volume change over time:

$$Q = A \cdot \frac{dh}{dt} \tag{11}$$

In a cylindrical well the surface area is given by: $A = \pi\, r^2$, resulting in:

$$Q = -\pi \cdot r^2 \, \frac{dh}{dt} \tag{12}$$

It can be assumed that the water flows away through the sides and the bottom. In this case the area of contact (A) is: $A = \pi \cdot r^2 + 2 \cdot \pi \cdot r \cdot h$ with r = radius of the well and h = the height of the side wall.

Disclaimer: in reality, some drywells do have infiltration through the bottom of the well and some not, due to clogging or closed off bottoms. In the calculations it will be assumed there is flow through the bottom since the area associated with the bottom is very small in comparison to the area of the walls of the well.

This can be demonstrated with a simple calculation. A well is considered with a depth of 4 m and diameter of 500 mm. The total area available for infiltration is: $A = \pi * r^2 + \pi * h * 2 * r = \pi * 0.25^2 + \pi * 4 * 2 * 0.25 = 6.48\ m^2$ from which only 0.2 m^2 is from the bottom. This is equal to 3% of the total surface area.

For the hydraulic conductivity value, the difference in this case would be 13.3 m/d for the situation with flow through the bottom (using inverse auger hole method), and 13.1 m/d for the situation without flow through the bottom. The influence on the calculation of the k value is slim, so for this reason flow through the bottom will be assumed.

Combining the formulas gives:

$$Q = k \cdot (\pi \cdot r^2 + 2 \cdot \pi \cdot r \cdot h) = -\pi \cdot r^2 \frac{dh}{dt} \rightarrow k \cdot 2 \cdot \pi \cdot r \left(h + \frac{r}{2}\right) = -\pi \cdot r^2 \frac{dh}{dt} \tag{13}$$

where $\pi \cdot r$ is cancelled out, leading to the following formula:

$$k \cdot 2 \cdot \left(h + \frac{r}{2}\right) = -r \frac{dh}{dt} \tag{14}$$

Rewriting gives (divide dh and dt to the sides):

$$\frac{k \cdot 2}{r} = -\frac{1}{\left(h + \frac{r}{2}\right)} \frac{dh}{dt} \rightarrow \frac{k \cdot 2}{r} dt = -\frac{1}{\left(h + \frac{r}{2}\right)} dh \tag{15}$$

By integrating both sides and combining the resulting constants to a single constant:

$$\frac{k \cdot 2}{r} \cdot t = -\ln\left(h + \frac{r}{2}\right) + C_3 \tag{16}$$

C_3 is determined by using the boundary condition that at t = 0 = t_0, h = h_0:

$$0 = -\ln\left(h_0 + \frac{r}{2}\right) + C \rightarrow C = \ln\left(h_0 + \frac{r}{2}\right) \tag{17}$$

This results in the overall formula being:

$$\frac{k \cdot 2}{r} \cdot t = -\ln\left(h + \frac{r}{2}\right) + \ln\left(h_0 + \frac{r}{2}\right) \tag{18}$$

Since log(A) – log(B) = log(A/B), the overall formula changes to:

$$k = \frac{r}{2 \cdot t} \ln\left(\frac{h_0 + \frac{r}{2}}{h + \frac{r}{2}}\right) \tag{19}$$

Since ln is the logarithm of the natural number 2.3, it is equal to 2.3log (x):

$$k = \frac{2.3 \cdot r}{2 \cdot t} \log\left(\frac{h_0 + \frac{r}{2}}{h + \frac{r}{2}}\right) \tag{20}$$

This eventually is simplified to <u>Porchet's formula:</u>

$$k = 1.15 \cdot r \cdot \frac{\log\left(h_0 + \frac{r}{2}\right) - \log\left(h_t + \frac{r}{2}\right)}{t - t_0} \tag{21}$$

This formula of Porchet will be used in the field experiments to determine the infiltration from the well into the soil and so, determine the saturated hydraulic conductivity.

4.4 Results

4.4.1 KSAT measurements

Table 7 shows the results of the hydraulic conductivity measurement in the laboratory for the samples from Pieter de Hooghlaan (near P1). Every sample was measured twice. It seems clear that the values near the surface are lower than in the layers below. From a depth of 1.5 m onwards the hydraulic conductivity value is generally in the same range.

Table 7: Hydraulic conductivity measurements of soil samples near Pieter de Hooghlaan

Hydraulic conductivity [m/d]			
Soil layer depth [m]	*Measurement 1*	*Measurement 2*	*Average*
1.0 – 1.5	3	3	3
1.5 – 2.0	19	21	20
2.0 – 2.5	17	17	17

In Table 8 the values of the hydraulic conductivity measurement for the samples at the Berlagevijver are shown.

Table 8: Hydraulic conductivity KSAT measurements of soil samples near Berlagevijver

	Hydraulic conductivity [m/d]			
Soil sample	*Measurement 1*	*Measurement 2*	*Measurement 3*	*Average*
1	12	14	16	14
2	32	28	32	31
3	19	20	19	19
4	23	25	22	23
5	32	41	37	37

4.4.2 Falling head experiments

The water level in the wells is depicted in the graphs in Figure 22, top. Since the water level in the second well at Vivaldipark (V2) was dropping at such a slow rate, the divers were left in the well overnight. For clarity it is plot in a separate graph from the other falling head tests (due to the long run time), see Figure 22, bottom.
An interesting observation is the rate at which the water level drops (falling head) is decreasing when the experiment is repeated more often. This complies with the expectations in Chapter 2 and 3 that the infiltration rate decreases as the soil water content increases.

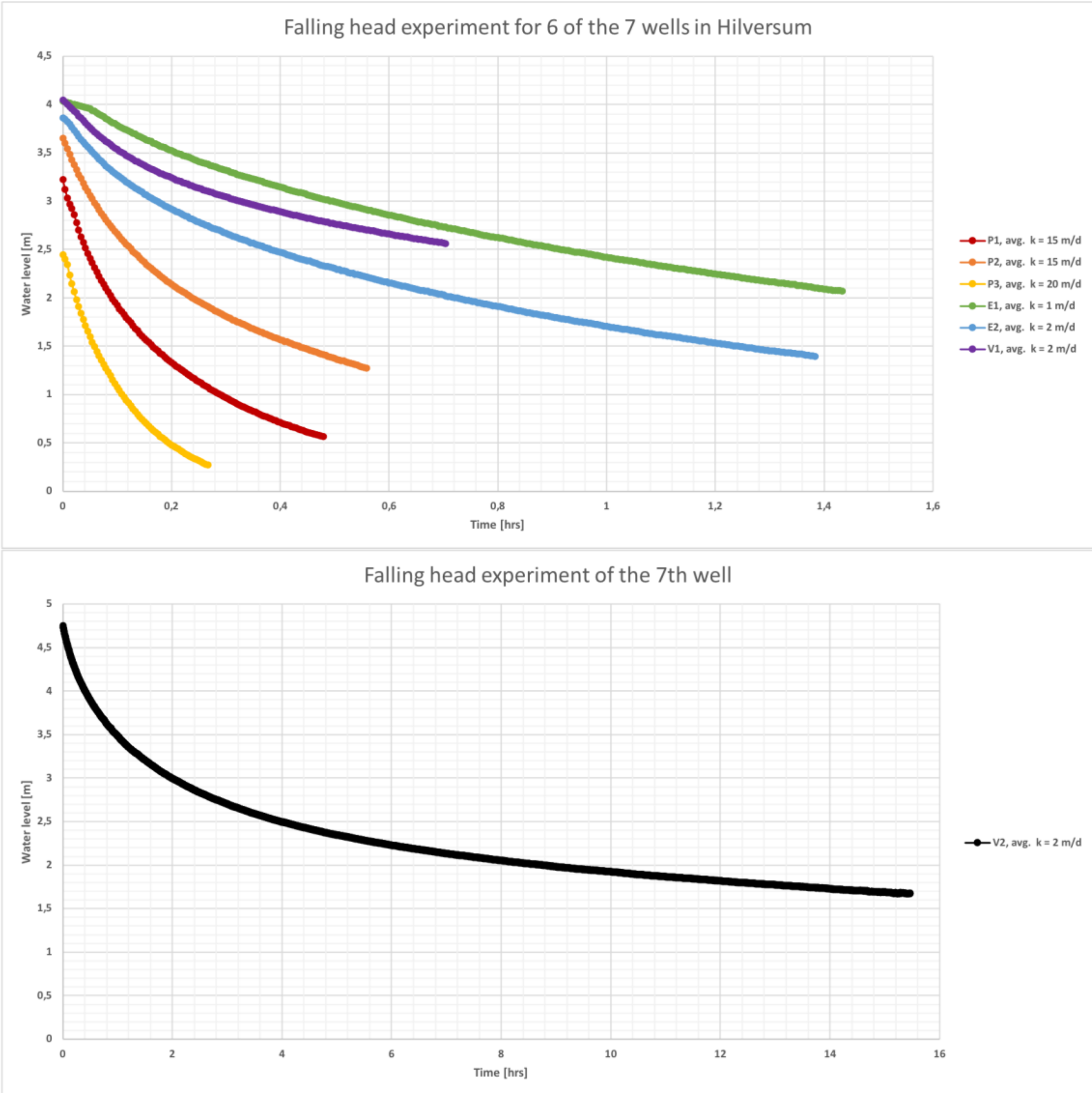


Figure 22: Top: falling head graph of the sampled wells. Bottom: graph of Vivaldipark well 2

4.4.3 Inverse auger hole method

Using the principle of the inverse auger hole method on the data obtained from the falling head experiment, the mean hydraulic conductivities are shown in Table 9. The minimal timestep for the measurements was 15 seconds. This is also used for dt in the Porchet formula since the timestep should be kept as small as possible to avoid miscalculations. Therefore, the mean hydraulic conductivities in Table 9 are calculated for dt = 15 seconds.

Table 9: calculated hydraulic conductivities using the inverse auger hole method on the falling head data

Well	Hydraulic conductivity [m/d]
P1	15
P2	15
P3	20
E1	1
E2	2
V1	2
V2	1

Figure 23 shows the distribution of the hydraulic conductivity measurements using the inverse auger hole method. The IDWs at the Pieter de Hooghlaan, showed a large value for the hydraulic conductivity but also a very large range in the calculated values. The smaller hydraulic conductivity values also had a smaller variation in the calculated values. A large variation in hydraulic conductivity values during an inverse auger hole experiment can indicate that the soil around the well had not been saturated enough. This is inherent to the permeability of a soil. If the test is performed at a location with low hydraulic conductivity, the soil surrounding the well is saturated faster. At a location with highly permeable soil, the water that is used to saturate the soil around the well will also flow down to the water table quicker. This means it is more difficult to saturate the highly permeable soil around a well.
Also, striking is the large number of negative values at V2. This is because the IDW drained at such a slow rate that the diver measured very small increases in the water level, resulting in negative hydraulic conductivity values.

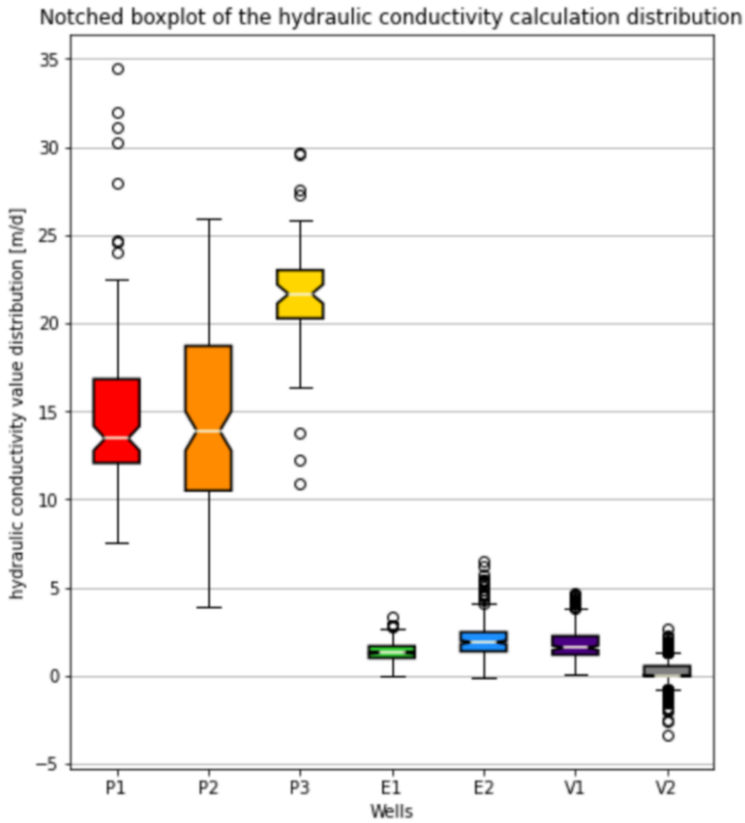


Figure 23: Notched boxplots of the hydraulic conductivity measurement using falling head test data of the IDWs in Hilversum

4.5 Conclusion on laboratory and field experiments

4.5.1 Laboratory experiments

The results obtained with the KSAT experiments for hydraulic conductivities agree with the soil profiles from TNO at this location and this region. According to this data, the subsurface in the city of Hilversum consists mainly of fine to coarse-grained sands, including the sample locations. The hydraulic conductivities resulting from the experiments are typical values of sand classified as fine to coarse-grained, see Table 10. It can be concluded that even for sandy soils the hydraulic conductivity can still vary greatly, as it also does in Hilversum. This means a large range of values for the hydraulic conductivity should be simulated for the design method in order for it to be useful. Furthermore, this also indicates that some locations within an urban area are less fit for implementation of IDWs.

Table 10: Common values for hydraulic conductivity of water trough soil from Verruijt (1970) and Bouwer (1978)

Material	Hydraulic conductivity [m/d]
Clay	< 0.0001
Sandy clays	0.0001 – 0.001
Peat	0.0001 – 0.01
Silt	0.001 – 0.01
Very fine sand	0.1 – 1
Fine sand	1 – 10
Coarse sand	10 – 100
Sand with gravel	100 – 1000
Gravel	> 1000

The large variation resulting from the KSAT measurements at the Berlagevijver is interesting since all the samples did come from the same location. This can indicate that the soil contains less conducting, fine layers. These layers restrict the functioning of an infiltration facility. Especially if the less permeable layers are situated underneath the IDW it can obstruct vertical flow to the groundwater. Overall, the values range somewhat between 15 to 30 m/day. In practice the difference between a value of 15 and 18 m/day is slim. This shows that the hydraulic conductivity should be considered as a range of values instead of an exact value. This is important to take into consideration for the design method.

Another interesting variation in the measurements was the difference in soil layers. The measurements of the different soil layers at the Pieter de Hooghlaan (P1) indicate that the top layer of the soil has a lower hydraulic conductivity. The organic matter in the topsoil causes a lowered hydraulic conductivity. Nemes et al., (2005) found a negative relationship between organic matter and the hydraulic conductivity.
It is also concluded from the KSAT measurements over the different soil depths that in order to determine the hydraulic conductivity of the soil, sampling should take place at a depth of 1.5 m or more to ensure the right hydraulic conductivity is measured.

4.5.2 Field experiments

The graphs clearly show distinct falling head rates for the different wells. It can be assumed that this is due to the difference in hydraulic conductivity of the soil surrounding the wells. Other important parameters are the well diameter, depth and possibly even clogging. The effect of the diameter is clearly visible from the falling head tests at Pieter de Hooghlaan (P1, P2 and P3). These wells were installed in proximity to each other. While all wells showed a rapid decline in the water level, the third was the quickest (P3). This well has a diameter of 500 mm while the others both have a diameter of 800 mm. This complies with the conclusion of the well diameter variation simulations of Chapter 3 Modelling drywell infiltration.

Another important factor was that the wells at the Pieter de Hooghlaan were never used after construction, so no clogging could have occurred. This in combination with high permeable soil clearly shows the wells easily drain the water. This is in contrast to the wells at the Eemnesserweg (E1 and E2) that were brand new and not clogged, but the water drained with less ease. The calculated hydraulic conductivity is small which indicates fine-grained sand is present at this location. Another reason could be the proximity to which the wells were installed next to each other. The four wells installed at this location were only located a few metres from one another (approximately 3 m apart). When there

is a substantial horizontal flow of water from the wells into the soil (due to a strong anisotropy) obstruction of the water flow from the well into the soil can occur. This could cause an increased resistance of flow from the well into the soil.
The wells at Vivaldipark however have been in use for over a decade. This either indicates that the soil has a small hydraulic conductivity, or it indicates that extensive clogging of the facility has occurred over time or both. From the data obtained however it is difficult to distinguish which of these factors is the main contributor to decreasing infiltration rates from the well. It is expected both play an important role in this process. These effects are discussed in the section on Clogging.

4.5.3 Comparison of in practice and theoretical falling head tests

The falling head tests from the field experiments can be compared to the simulations (virtual experiments) to get a better understanding of the processes that affect the functioning of an IDW. The comparison is done for the three wells at the Pieter de Hooghlaan (P1, P2 and P3) since the soil was sampled and analysed at this location, giving additional data on the soil characteristics at this location. In accordance with to the laboratory measurements, the hydraulic conductivity of the soil at this location varied between 17 and 21 m/d (measured at the appropriate depth). According to the inverse auger hole method it varied between 15 and 21 m/d. In Figure 24 the falling head data of the first well is compared to the simulated falling head with a similar saturated hydraulic conductivity. The falling head measurements fit the simulations of a well with homogenous soil of k = 5 m/d a lot better than one with k = 15 m/d. It seems the well is not performing as well as it should, based on the measured hydraulic conductivity and the geohydrological model. This indicates other factors influence the functioning of this well. Since all the wells at Pieter de Hooghlaan are not connected to the sewer system, the wells are practically unused. For this reason, no extensive clogging can have occurred over the years. Possible reasoning for the discrepancies between simulations and measurements are discussed in Chapter 6 Discussion.

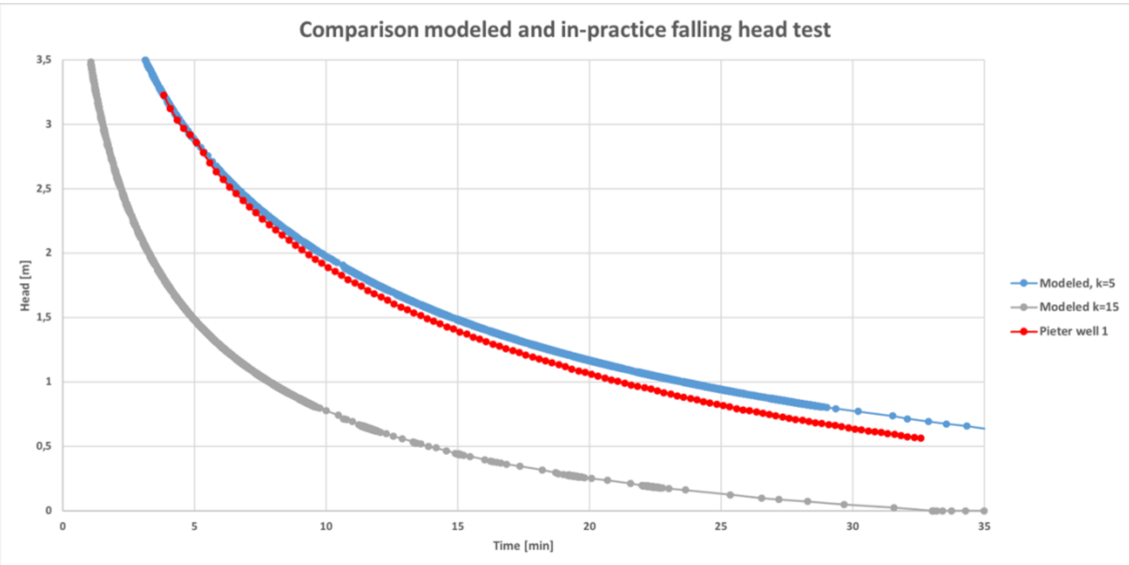


Figure 24: Comparison of falling head test at Pieter de Hooghlaan well 1 to simulated falling head test

If the same comparison is done for the second well (P2) at this location, we see a similar pattern. In Figure 25 the falling head graph is again compared to the simulations. In this case, the graph fits a modelled falling head experiment in soil with k = 2 m/d. Interestingly both wells P1 and P2 have the same diameter, are installed in the same soil and have similar depths. Also, the average hydraulic conductivity determined by the inverse auger hole method was the same for both wells. However, both IDWs function rather differently.

This indicates that the calculation of the average hydraulic conductivity based on a falling head test in an IDW is incorrect since it gives the same value for different measurements.

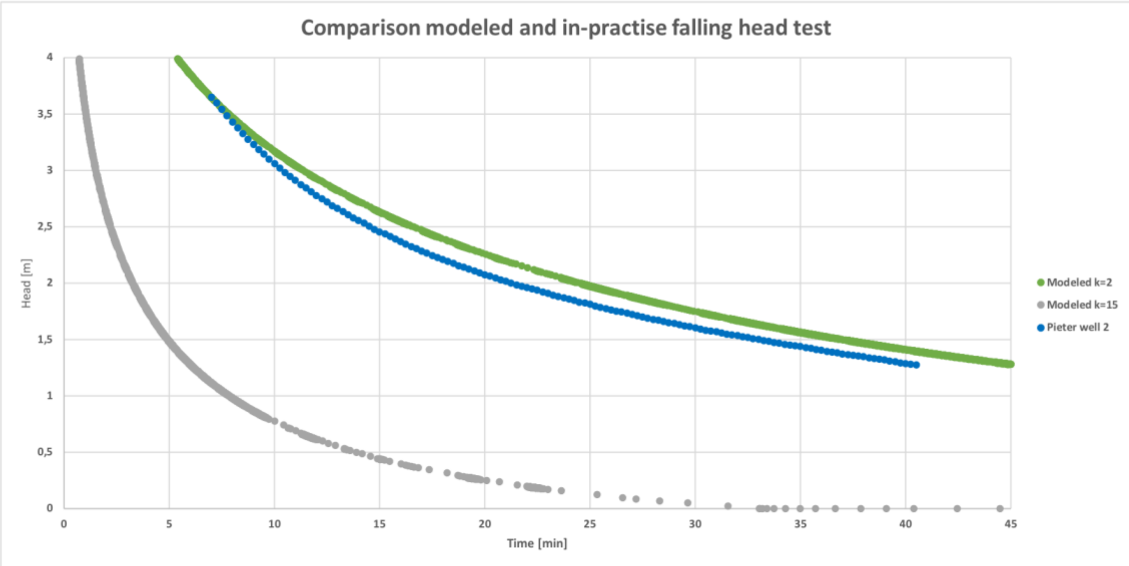


Figure 25: Comparison of falling head test at Pieter de Hooghlaan well 2 to simulated falling head test

A similar situation applies to the third well at this location (P3), which leads to the same conclusions. Again, the falling head test does not fit the simulated falling head for soil with k = 15 m/d. The best fit of this data is to soil with k = 10 m/d. This is visible in Figure 26.

For the other wells that were tested similar patterns occur. In the next chapter, the generic design method is presented based on the geohydrological model. In Chapter 6 Discussion, the results and shortcomings of the research are discussed.

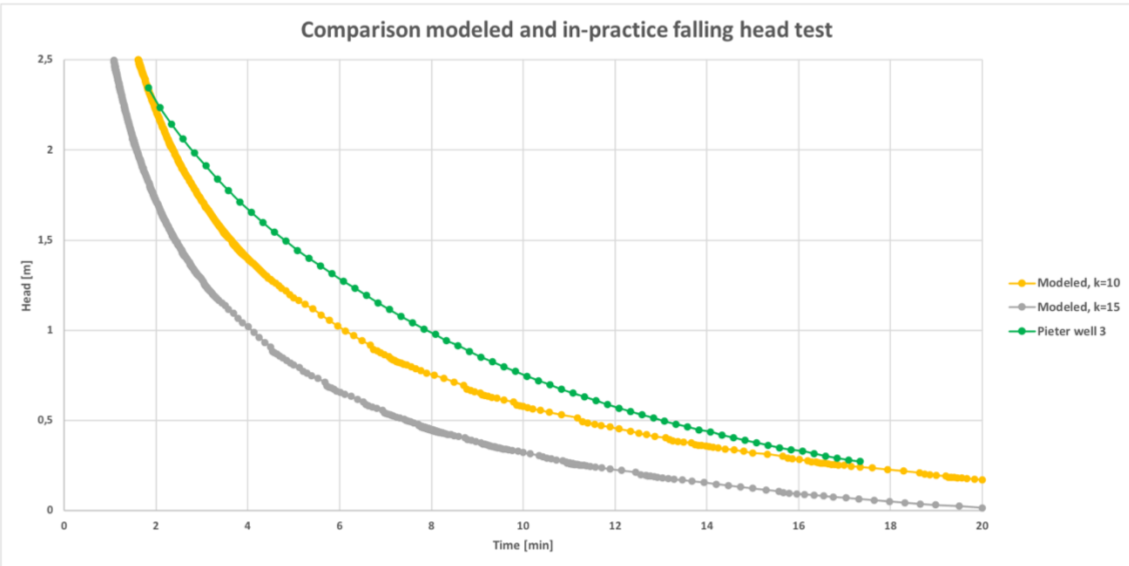


Figure 26: Comparison of falling head test at Pieter de Hooghlaan well 3 to simulated falling head test

5 IDW design method

In this chapter, the created design method is presented and explained. Furthermore, the use of the design method is clarified. This chapter can be consulted by anyone who wants to use the infiltration drywell design method.

5.1 Parameters for the design method

In this research, it was established that the most important parameters of the functioning of an IDW are the depth [m], diameter [mm] and hydraulic conductivity [m/d]. Another important parameter for the design method is the possible connected surface area [m^2]. This is the anticipated surface area that is to be connected to one are more IDWs. The design storm that is used to assess the hydraulic functioning of the well is the last parameter. In this part, the different parameters of the design method are discussed.

Diameter
Throughout this research, three diameters have been used to subdivide the different IDWs. This is based on in practice vertical pipe dimensions. For the design method the contour plots are again divided into these three different diameters: 300 mm, 500 mm and 800 mm. For clarity purpose the contour plots for the three different well diameters are separated, and not plotted in the same figure. This enables easier use of the contour plots for design.

Depth
As concluded from the research, shallow wells are not encouraged. This is because the falling head test pointed out that the rate of drop in water level for shallow wells is slim. For this reason, wells with a depth below 2 m are not included in the design method. Furthermore, it is important to point out that the depth that is included in the design method is the average groundwater depth and not the well depth. This is because groundwater depth is the limiting factor for IDW implementation.
In agreement with Sasidharan et al. (2019), the bottom of the well is set 1 m below the bottom of the IDW to account for this. This is to prevent stagnating water in the well. Especially during winter, when the groundwater usually reaches its highest level, this could lead to malfunctioning of the wells.
For the case study area of Hilversum, the groundwater monitoring data indicate that the groundwater level fluctuates between 0.5 – 1 m from the year-round average. So, for this case, the average your-round groundwater depth could be used for the design method. In regions where there is a larger fluctuation of the groundwater level, this should be kept in mind when consulting the design method.

Hydraulic conductivity
Again, it was determined that low hydraulic conductivities are unfavourable for IDW implementation. Therefore, only low hydraulic conductivities (< 2 m/d) are not included in the design method.

Design storm
In this research, the performance of IDWs is analysed for three different design storms. Eventually, the intense short rainfall event is used for the design method since this is the limiting factor of the three design storms. This intense rainfall event is a design storm of 21 mm in 10 minutes.

Connected surface area
The possible connected surface area data was based on model simulations and a design storm. With the data, the connected surface area of a well could be calculated and so, the required number of wells for a certain surface area. This data is included in the design method contour plot for different combinations of well dimensions and soil characteristics.

It is important to keep in mind that in this design method the maximum depth of the well is a set quantity since it is dependent on the groundwater depth. Also, the hydraulic conductivity is a set quantity that is measured in the field or in the laboratory. The diameter however is a design preference that can be altered after the desired result. The connected surface area can similarly be altered during the design process e.g., if a plan is not feasible or when a larger surface area could be connected. In the design method created out of this research, a specific design storm is applied. The use of a different design storm would undeniably result in a different design method. This is an important feature to take into account. In general, for a design storm with lower intensity, than 21 mm/10 min, this design method can still be used. If a more intense design storm is desired, the values in the design method should be taken conservatively.

5.2 Finalized design method

The main objective of this research is to set forth a generic design method for IDWs in sandy soils. This research demonstrated the important processes and parameters that determine the functioning of an IDW. Together with the created geohydrological model, this enables the construction of a generic design method for IDWs. With the acquired knowledge and data from model, field and laboratory experiments, a contour plot was created. This contour plot is based on the results of the geohydrological model since the simulations provided adequate data for the function of IDWs for varying sets of parameters. As explained in Section 3.3.3 it is possible to determine the necessary number of wells for a certain connected surface area, based on soil characteristics and well parameters. In the same section, it was concluded that a short intense rainfall event is the limiting factor for the possible connected surface area. Using a long-duration design storm, the possible connected surface area was significantly larger, which could lead to malfunctioning of facilities during intense rainfall events. Therefore, the contour plot is based on a design storm of 21 mm in 10 minutes. Furthermore, it was established that shallow wells (< 2 m) and soils with low hydraulic conductivity (< 2 m/d) result in insufficiently functioning IDWs. On these grounds, the design method is limited to include wells larger or equal to 2 m deep and hydraulic conductivities of 2 m/d or larger.

Figure 27 depicts the finalized contour plot of the design method for IDWs with a diameter (D) of 800 mm. The contour plots for diameters of 300 mm and 500 mm are included in Appendix C. The contour plot is based on a connected surface area of 100 m^2. Based on the hydraulic conductivity and groundwater depth the required number of wells for a connected surface area of 100 m^2 can be read from the contour plot. Note that the contour colour changes with every 0.5 number of wells. The white lines represent every 0.25 well. This means that a difference of 1 well is represented by 2 contour colours and 4 contour lines. These lines and colour changes enable an easy determination of the number of wells that are necessary based on a specific connected surface area. For connected surface areas larger or smaller than 100 m^2 the outcome can simply be multiplied. To explain the general use of the design method, an example is included in the next section. Note, it is advised to always round the total number of IDWs up when determining the design. This increases the margin of safety and prevent insufficient design.

The advised minimum distance between wells is summarized in Table 11.

Table 11: Advised minimum distance between IDWs

Diameter well [mm]	Recommended distance between wells [m]
300	2
500	3
800	5

Due to uncertainties in the model and results plus the absence of testing of the design method it is advised to include an in-depth design of the specific facility and research into soil characteristics before implementation of IDWs.

Design method example

A surface area of 350 m^2 is to be connected to one (or multiple wells). The measured hydraulic conductivity of the soil is approximately 7 m/d and a year-round average groundwater depth of 4.9 m is measured. A diameter of the wells of 800 mm is selected. The required number of wells is somewhere between 0.25 and 0.5 wells. In this case, it is rounded up to 0.5. Since the surface area is 350 m^2 and not 100 mm^2, the number of wells is multiplied by 3.5. This results in 1.75 wells. Again, it is advised to round this number up to 2 wells in total. The wells should be installed at least 5 m apart from each other, so the flow regimes do not obstruct each other in the subsurface.

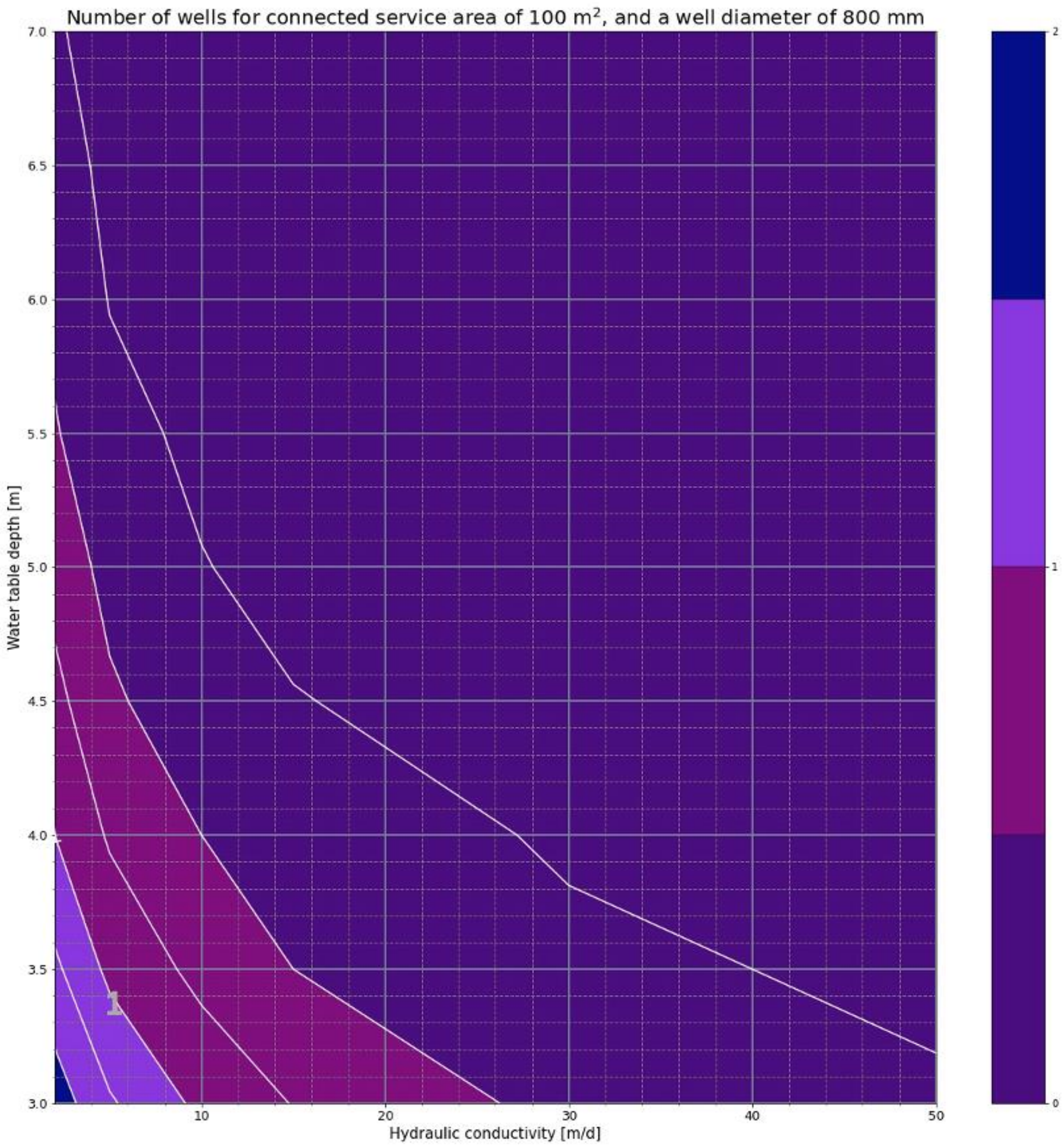


Figure 27: The design method depicts a contour plot of the required number of wells dependent on different input parameters. the average groundwater depth [m], the measured soil hydraulic conductivity [m/d], and the proposed connected surface area of this plot is 100 m^2. The contour plots for diameters of 300 mm and 500 mm can be found in Appendix C. Note: the depth of the well is always 1 m less than the groundwater depth since the bottom of the well is situated 1 m above the average groundwater level.

6 Discussion

6.1 Discrepancy between theoretical and practical functioning

As discussed in the Chapter 4 Field and lab experiments, there is a discrepancy between the falling head test data of the practical and theoretical experiments. This can have multiple reasons which are discussed in more detail in this chapter. These reasons could be on the end of the model or the field and laboratory experiments (or a combination). Additionally, the created design method is discussed. This discussion also leads to advice for further research in the field of IDW design.

6.1.1 Model

Homogeneity versus heterogeneity

A large contributor to the complexity of the geohydrological processes is the heterogeneity of the subsurface. Soil can vary greatly on spatial scale but similarly due to layering or macropores (Robinson & Ward, 1990). The subsurface characteristics can greatly differ from place to place. Since it is almost impossible to determine the subsurface composition at every specific location for the IDW design, it is also difficult to account for this variety when making representative models. In this research, the created model consists of homogenous soil or heterogeneous soil with a lumped hydraulic conductivity. The reasoning for this is the objective of creating a generic design method that is functional for different kinds of sandy soils. If a very specific heterogeneous soil is simulated in the model, it could give a better fit at certain locations with similar soil characteristics. However, if the soil characteristics, or for example stratification, differ from the model the fit would be worse. In studies like Sasidharan et al., (2019), Hydrus 2D was used to determine the heterogeneity of a specific well at a specific location. Here Hydrus was used to determine hydraulic soil characteristics surrounding an existing IDW and the model was calibrated using infiltration test data. This shows it is possible to get a good fit of the (2D) model for a heterogeneous soil. However, this study was aimed at finding a perfect fit for a specific IDW. It is aimed at creating a generic design method useable for wells with different parameters. Nonetheless, further research has to be conducted to determine the effect of heterogeneity on the IDW performance and especially the design method.

Anisotropy versus isotropy

The model created in this research does not include anisotropy of the soil. This limits the complexity of the model and the research. The anisotropy of the soil is an important soil characteristic that is present in especially sandy soils (Robinson & Ward, 1990). In hindsight, the inclusion of anisotropy could have resulted in a better model since it models physical properties in sandy soils to a better extent. To this end, a few additional simulations were run to get an indication of the effect of anisotropy on the functioning of IDWs. In Figure 28 the effect of anisotropy is illustrated. In these simulations, the anisotropy was set to a value of 6. This is an average value for moderately homogenous sandy soils in the Netherlands (Bot, 2016). It means the horizontal hydraulic conductivity is 6 times larger than the vertical hydraulic conductivity. For clarification, the value for the hydraulic conductivity (k) used in these simulations represents the horizontal value. In the case the simulated (horizontal) k = 1 m/d, this means the vertical k = 0.167 m/d.
The runs were executed for 3 different values of the hydraulic conductivity, and a well with a diameter of 500 mm, the falling head experiments are illustrated in Figure 28. In first instance, the drop of the water level in the wells seems to only result in little contrast between isotropy and anisotropy. Considering drain time, there are some variations. For an IDW with soil characteristics of k = 10 m/d, isotropy shows the well is emptied in about

31 minutes. The same well with anisotropy is emptied in an hour. This is twice as long. The falling head graph levels out more when anisotropy is added to the geohydrological model. This is expected since the vertical hydraulic conductivity is lower than a similar model with isotropy, while the horizontal hydraulic conductivity is equal. This shows that even though the water level in the wells is very similar, the time it takes to completely drain an IDW is longer. However, the differences in falling head data are not to such an extent that it is expected to result in a completely different design method. Therefore, the gained data from the isotropy simulations are used for the creation of the design method. Future research could test this assumption.

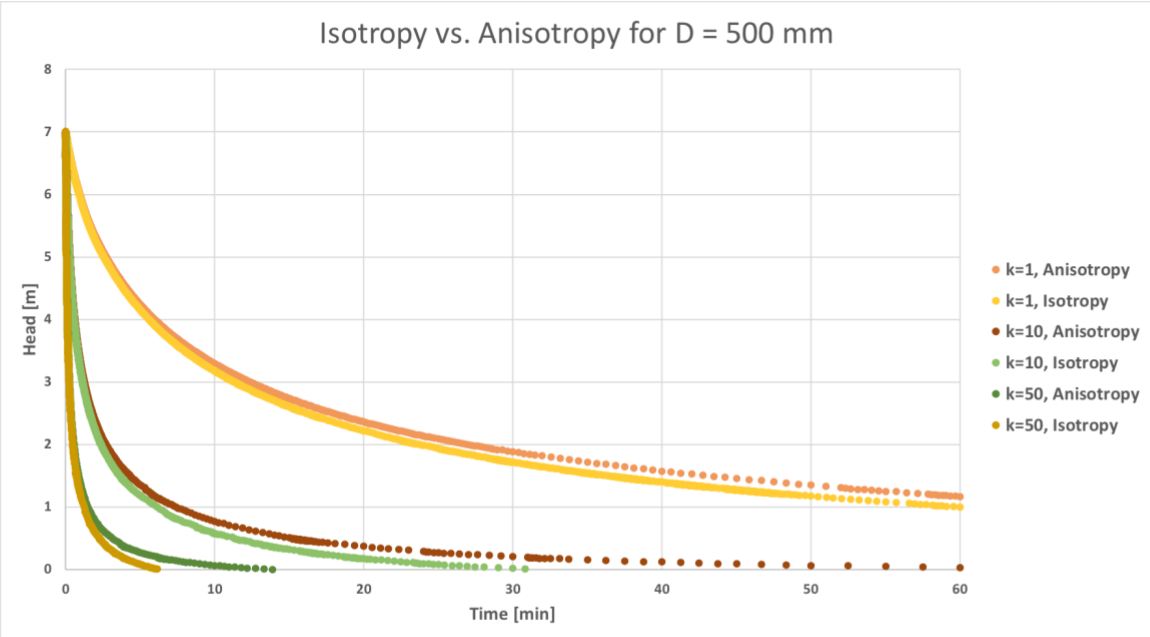


Figure 28: Difference in falling head test for isotropy and an anisotropy of 6 for D = 500 mm

Additional wall resistance

Another contributor to the discrepancy of model fit is the absence of the wall structure that is present in vertical infiltration pipes in practice. This is because there could be additional resistance by the wall of the well, limiting the outflow of water. In the model, the boundary condition can only be set to seepage interface. This means the water from the well can infiltrate into the soil at any given point at this boundary. This is a therefore a model limitation. In reality, the openings in the well wall are only situated at certain locations. Most of the wall surface area is impermeable. Because of this, the water cannot drain freely into the soil from the well. To investigate the effect of the well wall an additional experiment was conducted. In this experiment, a vertical infiltration pipe, equal to the ones used for IDWs, was erected in the open air. The idea was to fill the pipe instantly with water and conduct a falling head experiment. The pipe is surrounded by the atmosphere instead of soil. In this way, a so-called baseline measurement could be conducted. This falling head data could subsequently be used to get a physical representation of the wall resistance, e.g., in the form of a maximum possible saturated hydraulic conductivity. Unfortunately, the pipe drained in such a quick way that no valid measurement of the falling head was measured.

6.1.2 In practice

Sampling

The hydraulic conductivity of the samples that were analysed in the laboratory did not match the hydraulic conductivity of the model simulations. This could be explained by the fact that sampling can cause an error in soil characteristic measurements. First of all, the

sample is removed from the natural environment when it is analysed in the laboratory. This alone alters the conditions of the sample. A piece of soil situated 2 m below the surface is compacted due to the presence of the surrounding soil. When the piece of soil is sampled and transported to the laboratory, the environment is different. This can result in modified hydraulic conductivity measurements compared to natural conditions. Secondly, related to this reasoning, the sample that is measured in the laboratory is completely dried and mixed after which it is completely saturated before the measurement. This eliminates possible anisotropy of the sample since the soil particles do not have the same orientation as under natural circumstances. As concluded by Bagarello, Sferlazza & Sgroi (2009), structure alteration in a sampled soil can yield deviating results. Furthermore, macropores and stratification in the sample are eliminated.
Even though some measurement error the true hydraulic conductivity of the laboratory samples is probably similar to the measured values. However, for determining the hydraulic conductivity in the field, other methods could be more precise. Also, the measurement under field conditions is advised. Some examples are infiltrometer method (vertical hydraulic conductivity), auger hole method (horizontal hydraulic conductivity) and inverse auger hole method using an auger hole instead of an IDW to do the measurements.

Inverse auger hole method

The Porchet formula or inverse auger hole approach was used in this research to approach the hydraulic conductivity in practice based on falling head test data in IDWs. As concluded in Chapter 4 Field and lab experiments, there is a discrepancy between the in practice and theoretical falling head tests. To give an example two distinct falling head graphs (P1 and P2) resulted in the same mean value of saturated hydraulic conductivity.
An important concept for using the Porchet method is the assumption that the gradient in sandy soils decreases linearly over the distance and so, is equal to 1. This means that the wetting front makes an angle of 45 degrees with the horizontal plane when infiltrating into sand. This hypothesis was accepted by conducting an experiment which is explained in Section 4.3.3 Inverse auger hole method. However, this experiment was done in a two-dimensional setup where the movement of the wetting front is observed in the x and y directions. When considering the infiltration of water from a cylindrical well into the subsurface, the hypothesis of a gradient of 1 does not comply.
This concept can be explained with a simple figure. Consider the situation in Figure 29, from the illustration it is evident that the cross-sectional area is enlarged with increasing distance from the well. Area A1 is smaller than area A2 due to an increase in the total circumference when moving away from the well. This means when the water is infiltrating from the well into the soil in a radial direction the cross-sectional surface area over which the water flows becomes increasingly large. For this reason, there will not be a gradient of 1, and a linear wetting front. It means the gradient decreases more rapidly when moving away from the well and so; the wetting front turns down more rapidly when moving away from the well. This is also visible in the model simulations when looking at the wetting front, see Figure 30. The wetting front has a larger angle than 45° with the horizontal plane.

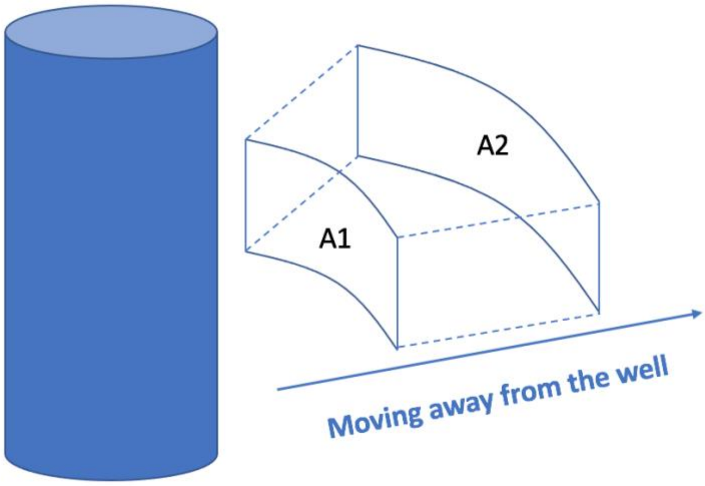


Figure 29: Schematization of increase in cross-sectional area with increasing distance from the well

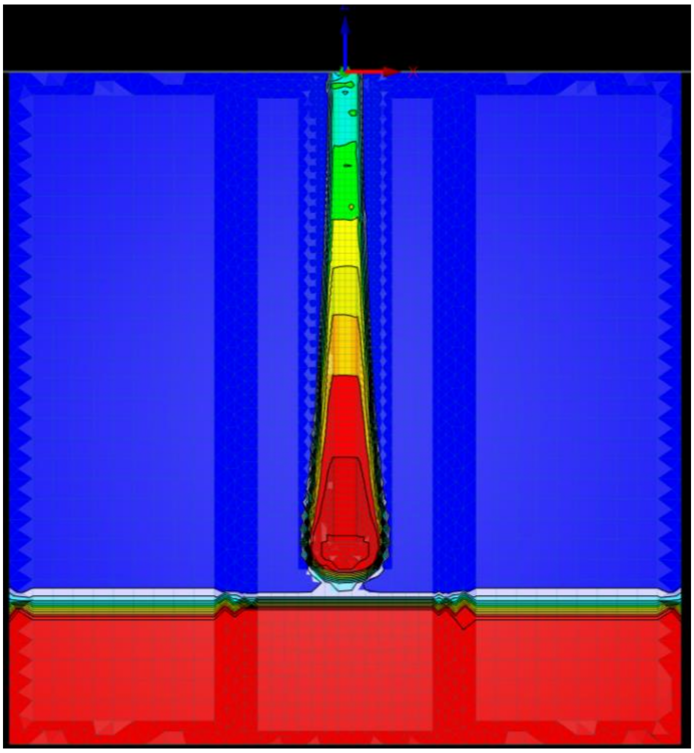


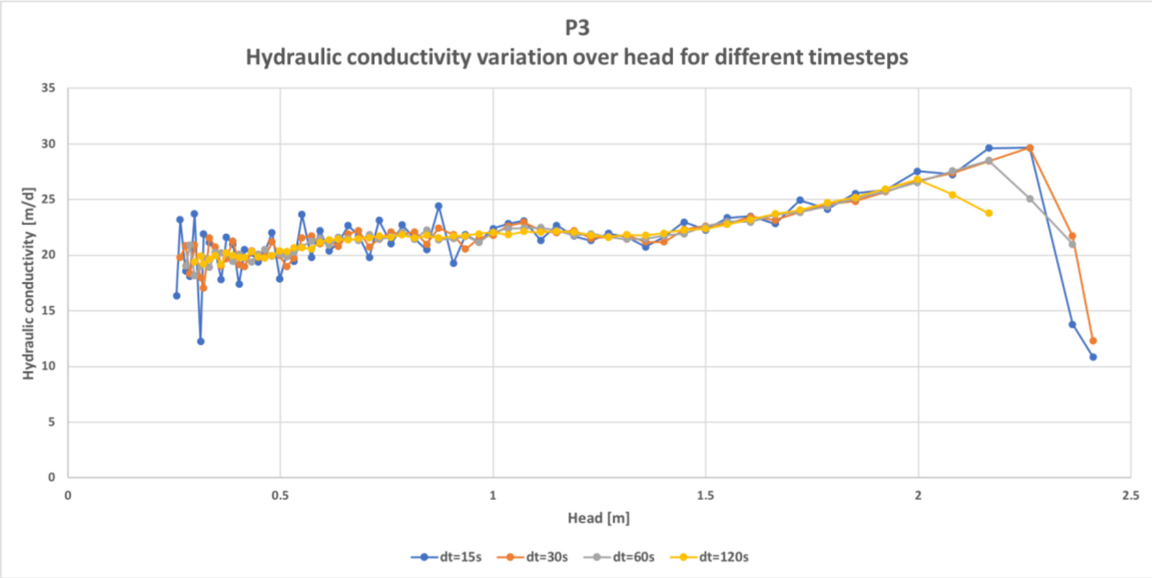
Figure 30: Moisture content surrounding a D = 300 mm IDW after a falling head simulation of 1 hour.

The inverse auger hole method makes use of falling head test data in boreholes. In this research, a slightly different approach was taken by the utilization of IDW falling head test data. The principle of the volume balance of water flowing out of the well through the sides and bottom of the well should be the same. However, the effect of using a much larger 'borehole' can affect the usability of the inverse auger hole method. When utilizing this method, it is advised to use an actual auger hole instead of an IDW.

Furthermore, when performing a saturated hydraulic conductivity measurement using Darcy's law the cross-sectional area is constant. Considering a radial domain, the area perpendicular to the flow increases with distance from the well. This enables a larger volume of water to flow through this increasing area and therefore an overestimation of the discharge. In turn, this results in an overestimation of the hydraulic conductivity.

This is also visible considering the data obtained from the theoretical and in practice falling head experiments. The in practice falling head test data showed a good fit to simulated falling head tests of the model. However, the calculated hydraulic conductivities using the in practice falling head test data gave larger values than the simulated data with a similar fit. This indicates that the Porchet formula results in an overestimation of the calculated hydraulic conductivity.

An additional problem of using the Porchet formula was the determination of the overall saturated hydraulic conductivity using the falling head test data. It could be argued that the minimum value of k is to be selected. Unfortunately, this resulted in some very low (E1), and even negative (E2 and V2) values. When utilizing larger timesteps for the calculations these outliers are evened out. However, this still resulted in large variations of the calculated hydraulic conductivity, especially when plotted against the hydraulic head (water depth) in the well. It seems like every well had a very distinct 'fingerprint' when considering the falling head tests and calculated hydraulic conductivity. In general, the hydraulic conductivity decreases throughout the test and so, decreases with the drop of the water level in the well. In Figure 31 the variation of the hydraulic conductivity is plotted against the corresponding hydraulic head in the well. For the graphs of the other wells, see Appendix D. Both graphs are very different. The graph of well P3 (for location see Figure 14) shows a very steady and almost constant value for the hydraulic conductivity. Also, looking at the different timesteps that were used for the calculation, the values are somewhat constant for all timesteps. The bottom graph of Figure 31 shows the hydraulic conductivity variation of the falling head test in well V1. For this experiment, the calculated hydraulic conductivity varies greatly with the water depth in the well. For large depths, at the start of the experiment, the calculated hydraulic conductivity is significantly larger than for lower depths, at the end of the experiment. Due to this large variation, it is difficult to determine the overall hydraulic conductivity at this location. This outline of the graph is also present (to some extent) for the wells at other locations. This indicates a relationship between the water depth in the well and the calculated hydraulic conductivity.
Concluding, the Porchet formula resulted in overestimations of the hydraulic conductivity due to an increase in the radial domain of the cross-sectional area. This in combination with a significant variation of the calculated hydraulic conductivity values over the falling head test data, proves that the inverse auger hole method (and the Porchet formula) should not be used to determine the hydraulic conductivity using IDWs.



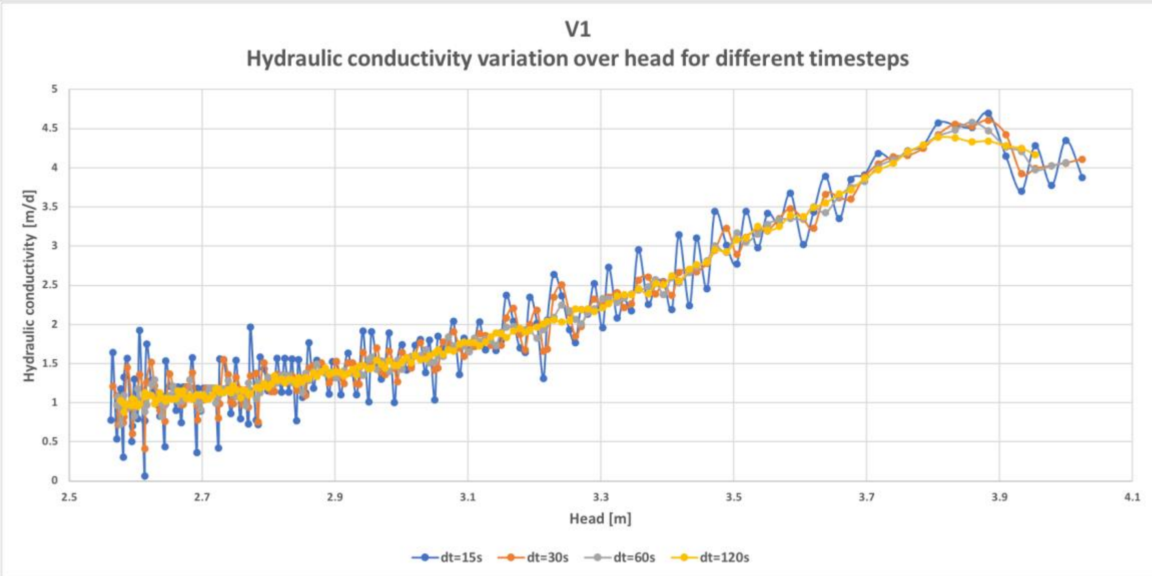


Figure 31: The variation of the hydraulic conductivity over the depth for two different wells. For the calculation various timesteps were used. The graphs of all the wells can be found in Appendix D

Clogging

Clogging can have a profound impact on the falling head tests conducted with the IDWs. Furthermore, clogging is difficult to determine since the effects of clogging are similar to a smaller soil hydraulic conductivity. To determine the degree of clogging of an IDW, extensive research of the well and the soil properties should be conducted. Since this is significantly outside the scope of this research, additional future research is recommended.

To illustrate the impact of clogging an additional visit was paid to the municipality of Arnhem. Here inspection of poorly functioning IDWs was performed. The IDWs had been in use for almost 10 years. In Figure 32 the effects of clogging after 10 years of use are visible. The geotextile surrounding the well (left picture) is completely blackened (the textile used to be white). This blackening also increased downwards on the geotextile. Also, the soil surrounding the well was profoundly discoloured (right picture). When analysed in the laboratory the soil surrounding the well was found heavily polluted with lead and zinc. Also, heightened values of copper, mercury, molybdenum, nickel and other toxic compounds were measured. These chemicals originate from car exhaust pipes and weathering of car components like tires. During rainfall events, these chemical compounds are flushed and end up in the IDW. When entering the soil, the particles become inactive and cause clogging of the layer surrounding the well. To combat this regular maintenance and inspections of the IDWs are important. Bouwer (2002) also suggests ‘resting’ of clogged IDWs if the clogging is predominantly organic. It is done by drying the well over a long period of time (a year or longer). However, also here additional research could quantify and help combat the effects of clogging

Figure 32: Traces of clogging around IDWs in Arnhem

6.2 Design method contour plot

For the design method various approaches were executed to visualize the concluding data of this research. Due to the discrepancy between theoretical falling head experiments and the in practice falling head experiments the design method is solely based on the Hydrus 3D model. Because the model simulations form a reliable basis to form the design method on in contrast to the experiments in practice. A drawback of the fact that the design method is solely based on a model is that it has not been tested in practice yet. Future implementation of the design method will prove the level of reliability.

6.3 Recommendations

Implementation and use of design method

Since it is difficult to determine the hydraulic conductivity for an entire area and especially at different depths it is recommended to perform multiple hydraulic conductivity measurements at different depths and different locations to see if there is a large variation. From here on the design method of this thesis can be consulted. Since the design method is not yet tested in practice and the discrepancy between theoretical and practical functioning, it is advised to use it in the early stages of designing IDW facilities. The design method aids in indicating the feasibility of an IDW (network) plan.
It is recommended to use the quantities in the design method conservatively if e.g., a more intense design storm is preferred or when the soil has a large anisotropy.

Future research

The design method set forth is a starting point that can still be improved in the future. To start, the design method has not yet been tested in practice. The comparison between practical and theoretical functioning of the wells that was discussed in this report was used to get an indication of the functioning of IDWs and the processes surrounding infiltration. In the future the design method should be tested in practice with additional research on the performance of these wells. It is expected that there will be a difference between practical and theoretical functioning to some extent. The big question is how useful the design method will be in practice.

Furthermore, the geohydrological model can be optimized further. Some suggestions are the inclusion of anisotropy, heterogeneity and some form of wall resistance. To this end, a setup could be imagined similar to the one described in Section Additional wall resistance. When including heterogeneity in the model it is recommended to use Staringen series (Wösten et al., 1986, Heinen et al., 2018) for the selection of the Van Genuchten parameters in the Netherlands. These are normalized soil characteristic model parameters for different kinds of soils and the accompanying soil horizons. In this series 18 topsoils and 18 subsoils of the Netherlands are described. This data can greatly aid in the implementation of heterogeneity in the geohydrological models. However, the inclusion of heterogeneity will only optimize the model for locations that have similar heterogeneity in the soil. A suggestion is to create various models, each with a specific kind of heterogeneity included that could represent the different kinds of soils present in the Netherlands (or even abroad). Future research could also include the effect of varying the anisotropy in the model simulations to indicate the effect of anisotropy.

One important factor to be researched is the effect of infiltration drywell clogging. This has a profound effect on the useability of, not only infiltration drywells but also infiltration facilities in general. The large downside of research into clogging is the time component connected to it. Some of these wells have been in use for years or before serious clogging affects the functioning of the well. It is therefore suggested to create a setup of an IDW where consecutive falling head experiments are conducted over a long time series. In this way, the deterioration of the functioning of the well over time can be observed. This deterioration occurs due to the repetitive infiltration of stormwater and drying out of the soil. The reasoning behind this is the inactivation of chemical compounds when the soil dries.

If this kind of research is carried out it is important to use polluted water that contains similar pollutants as flushed stormwater. Since the clogged layer around the wells in Arnhem were highly polluted by heavy metals it is expected that these have a large part in clogging of the soil and geotextile.

6.4 Water quality

A part of the discussion about IDW design that has not been included yet concerns the water quality. This research is quantitative research which means the water quality is left outside the equation. Since water quality has a profound impact on the usability of IDWs, it must be considered. A review of multiple articles on water quality concerning infiltration facilities by Weiss et al. (2008), concluded that pollutants in urban stormwater runoff can potentially contaminate groundwater. After longer periods of use, the receiving soils exceeded the levels of pollution for certain pollutants. These studies showed that urban stormwater runoff usually contains high concentrations of nutrients, suspended solids, heavy metals, pathogens, petroleum hydrocarbons and salts. Nutrients can cause eutrophication of receiving water bodies, which can be a risk for aquatic life and the overall biodiversity. However, even though infiltration was related to some of the pollution risks, conventional methods, like combined sewer overflows (CSOs), result in higher risks of contamination.

7 Conclusion

The main objective of the research is to create a generic design method for infiltration drywells (IDW). These wells enable the draining of stormwater into the subsurface in urban areas. In this way the groundwater is replenished, and the sewer system can be relieved during precipitation. To meet this objective, an extensive analysis of IDW functioning in theory as in practice was conducted. The research shows that the performance of an IDW is largely dependent on the depth and diameter of the IDW and on soil hydraulic conductivity. The input parameters of the design method are depth (dependent on groundwater level), diameter, hydraulic conductivity, connected surface area and design storm. The output parameter is the required number of wells. The groundwater level, and so maximum well depth, and the hydraulic conductivities are soil characteristics that are fixed quantities at each location. The diameter and connected surface area are variables that can be altered depending on the desired result.
All these parameters are included in the design method. This design method can be used by any party that wants to implement IDWs in the urban environment.

The design method is based on simulations of IDWs with different depths, diameters and hydraulic conductivities in Hydrus 3D. With the results of these simulations, empirical contour plots were created. For the contour plots for the three different diameters, see Appendix **C**. In this way, users can easily determine the type and number of wells to use for stormwater drainage. The design method can be of great value in the early stages of IDW design. It indicates on the viability of a plan to construct multiple or single IDWs. However, it should be mentioned that due to several uncertainties with the creation of the design method and the geohydrological model that it is based on, further research is recommended to come to an optimized design method. The optimized design method could e.g., include anisotropy of the soil. The applicability of the created design method has to be researched by the installation of IDWs in practice.

It is important to keep in mind that the use of IDWs becomes unprofitable when the groundwater level is close to the surface (< 3 m) and especially for soils with very low hydraulic conductivity (< 2 m/d). For areas with shallow groundwater tables, larger infiltration facilities like soakaway crates are advised. This is because soakaway crates have a large storage volume that can be used to store the stormwater before infiltration. Shallow IDWs only have a small storage volume and would fill up quickly during a storm. For this reason, a large number of wells is necessary to drain even smaller surface areas. For soils with hydraulic conductivity below 2 m/d the wells empty at a low rate. This will result in water ponding in the wells for a long time which obstructs the functioning of the well at later stages. An advantage of using numerous smaller IDWs is that the contact area between water and soil is significantly larger compared to the use of soakaway crates. However, since the wells empty slowly, and contain only a small storage volume, it is a trade-off between slightly faster infiltration (with small wells) and a large storage volume (with soakaway crates).

The use of a design storm to test the design of hydraulic structures is a practice that is used throughout the field of engineering. In this research, a short intense design storm appeared to be the limiting factor on design in contrast to design storms with longer periods of precipitation. For this reason, a design storm of 21 mm in 10 minutes was used to create the design method. This design storm has a statistical return period of once every 25 years in the Netherlands. In sewer design, usually a shorter return period is used for design storms. However, assuming precipitation patterns will intensify in the future it is wise to use a longer return period for the design storm.

An important conclusion from this research is that the inverse auger hole method does not suffice in determining the hydraulic conductivity from falling head test data of IDWs. Due to the radial infiltration domain around an IDW, where the cross-sectional surface area of flow in the radial direction increases with increasing distance to the well, an overestimation of the calculated hydraulic conductivity occurs. Laboratory measurements of the hydraulic conductivity showed varying results. It is therefore recommended to determine the hydraulic conductivity in the field, with a different method than the auger hole method using IDW falling head test data.

To summarize, this extensive research into infiltration drywells showed the added value of using infiltration facilities for urban stormwater management. Furthermore, the created design method is an attribute for the implementation of infiltration drywells and enables an easier design of these facilities than before.

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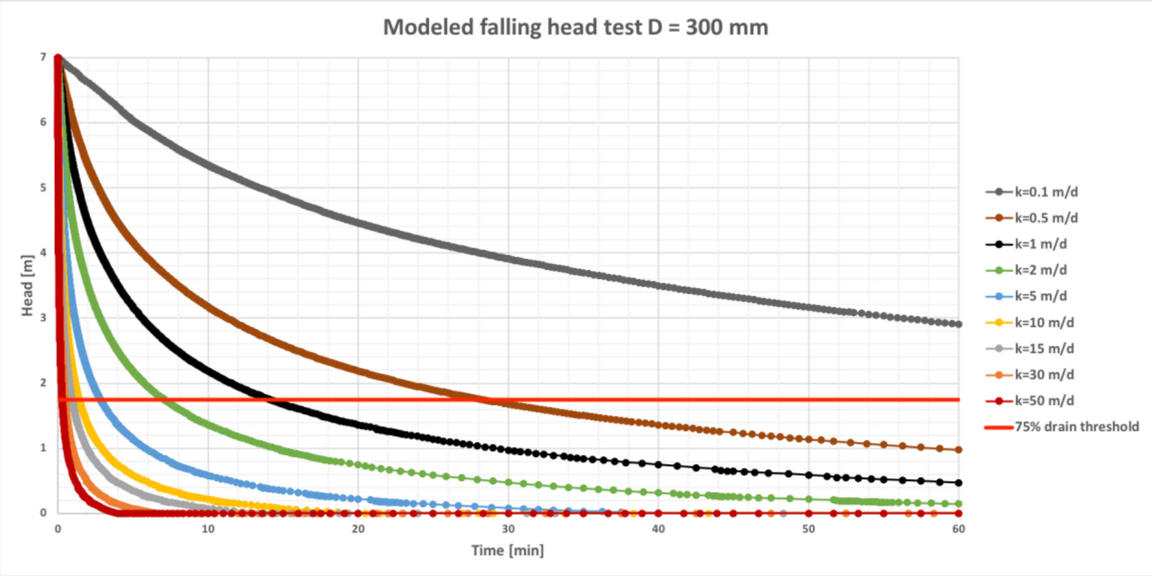
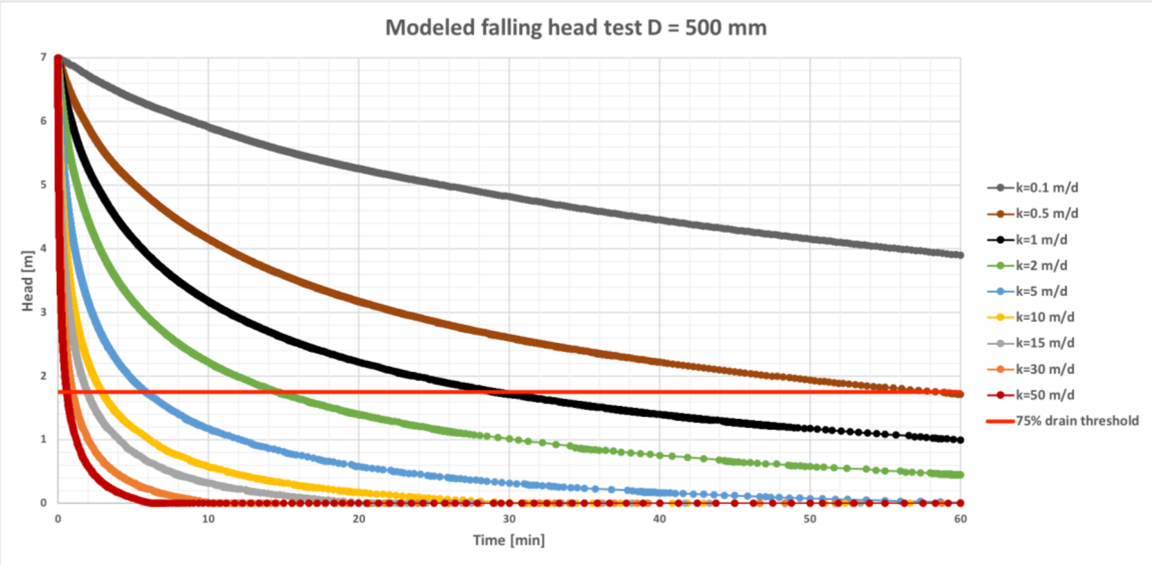
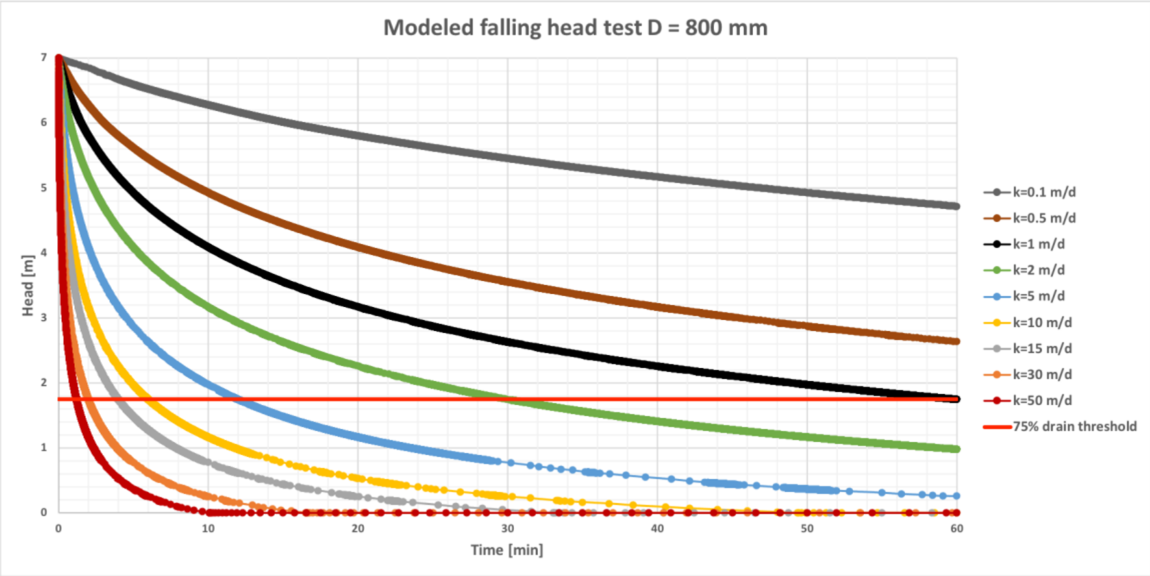
Appendices

A

Falling head tests of Hydrus 3D model simulations

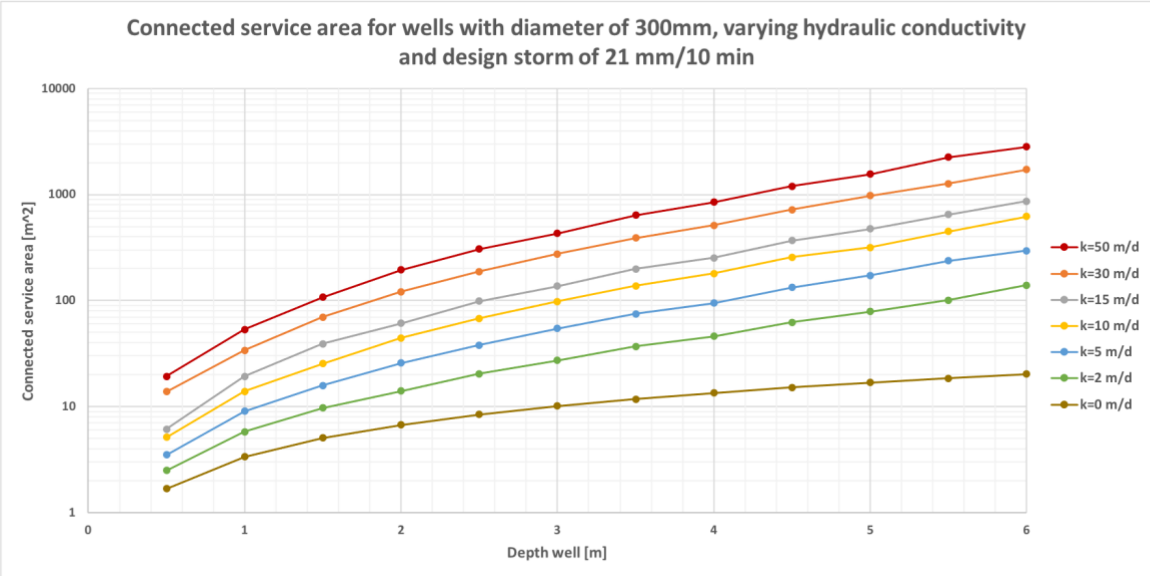


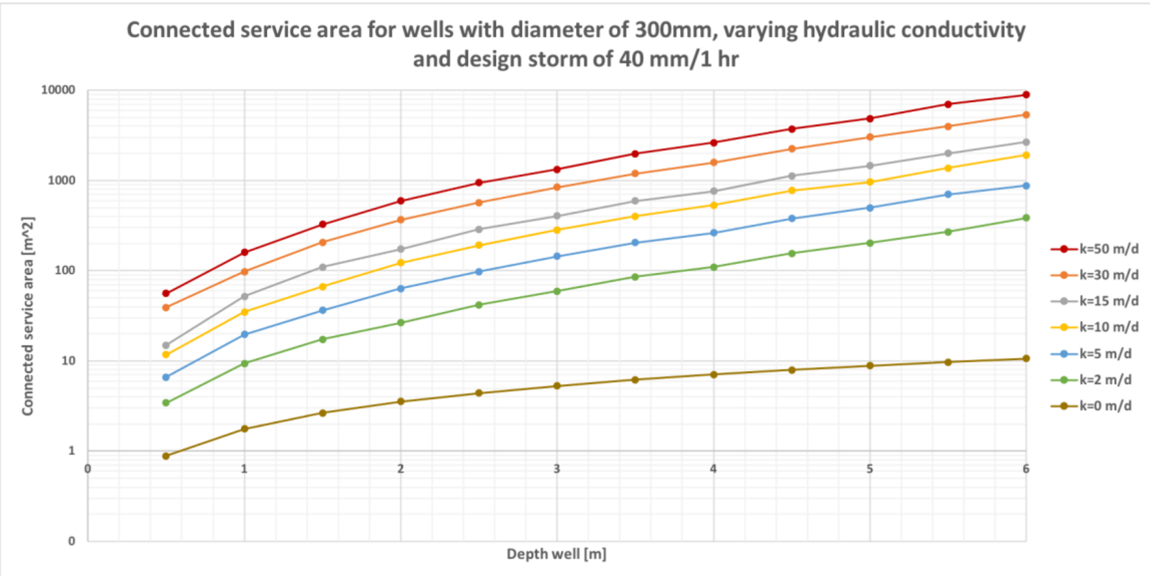


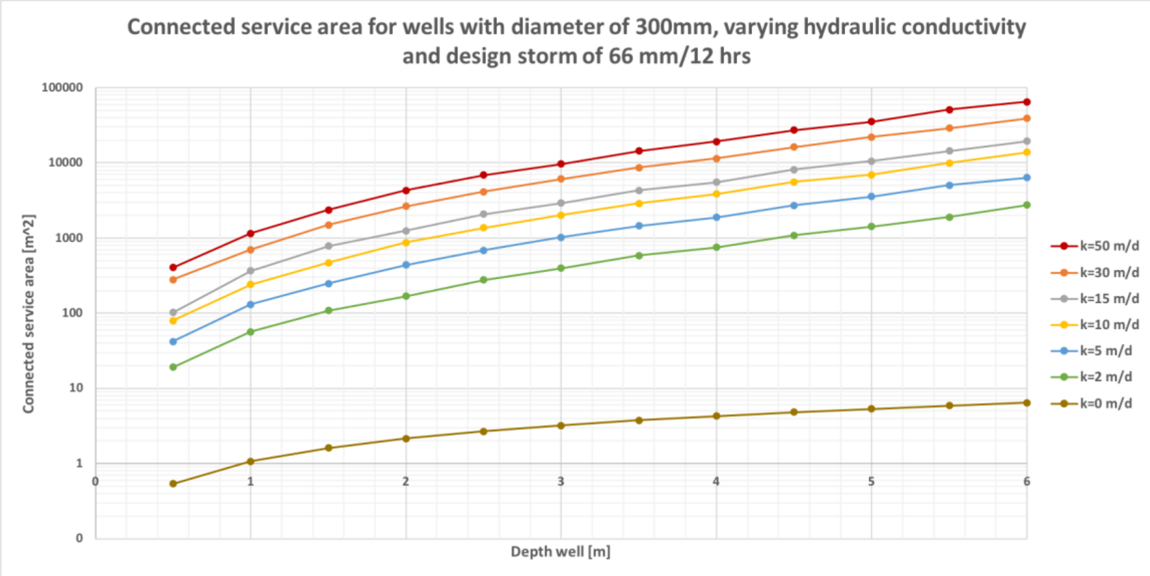

Modeled falling head test D = 800 mm
Head [m]
Time [min]
0
1
2
3
4
5
6
7
0
10
20
30
40
50
60
k=0.1 m/d
k=0.5 m/d
k=1 m/d
k=2 m/d
k=5 m/d
k=10 m/d
k=15 m/d
k=30 m/d
k=50 m/d
75% drain threshold

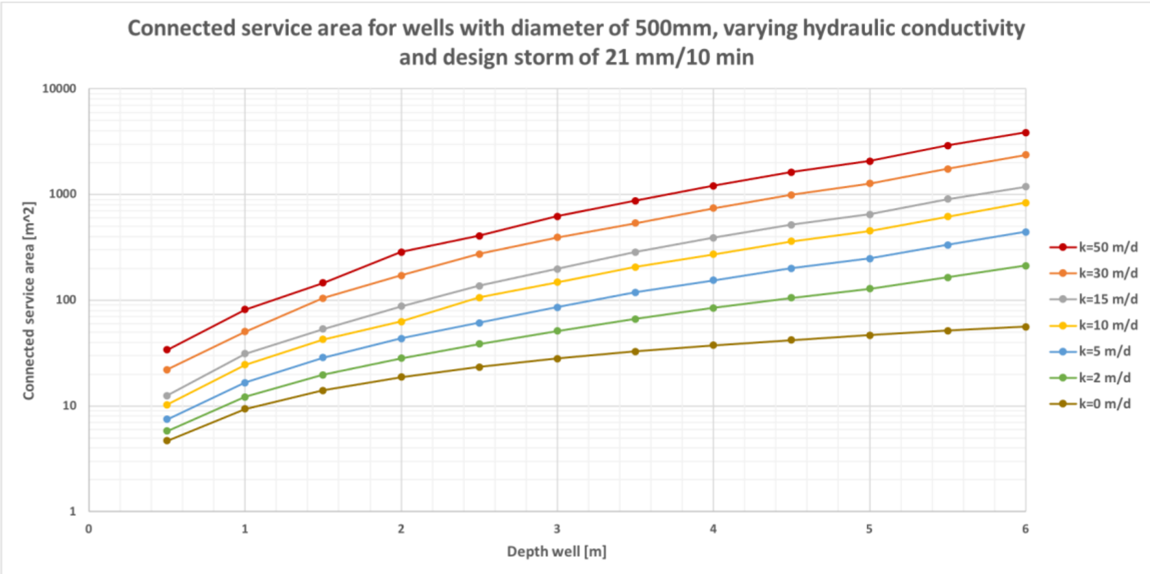
B

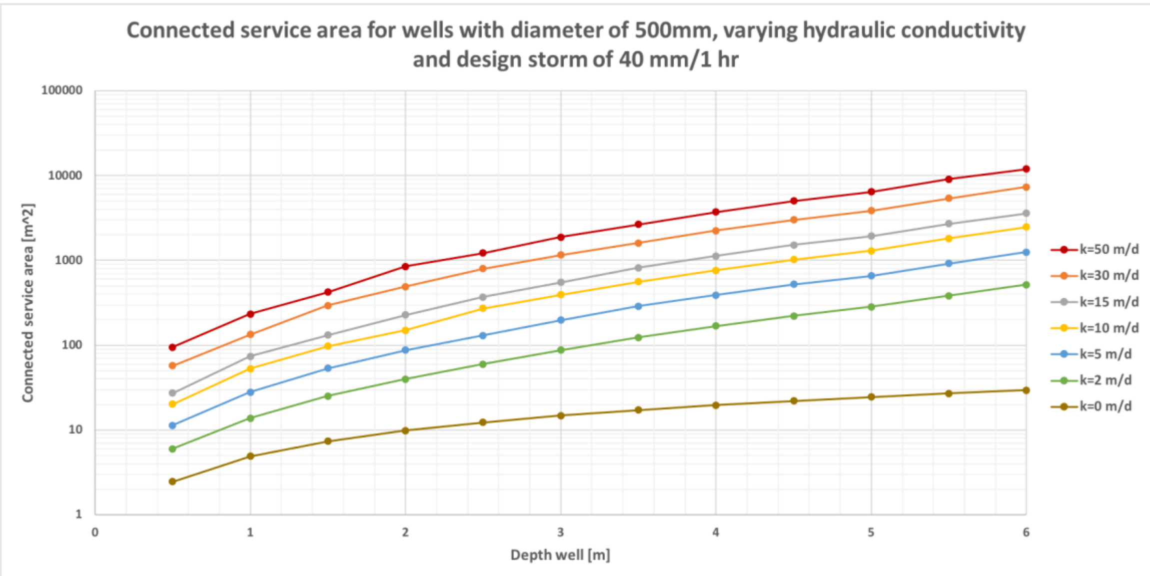
Possible connected surface area for different well depth, diameter, and hydraulic conductivity and different design storms

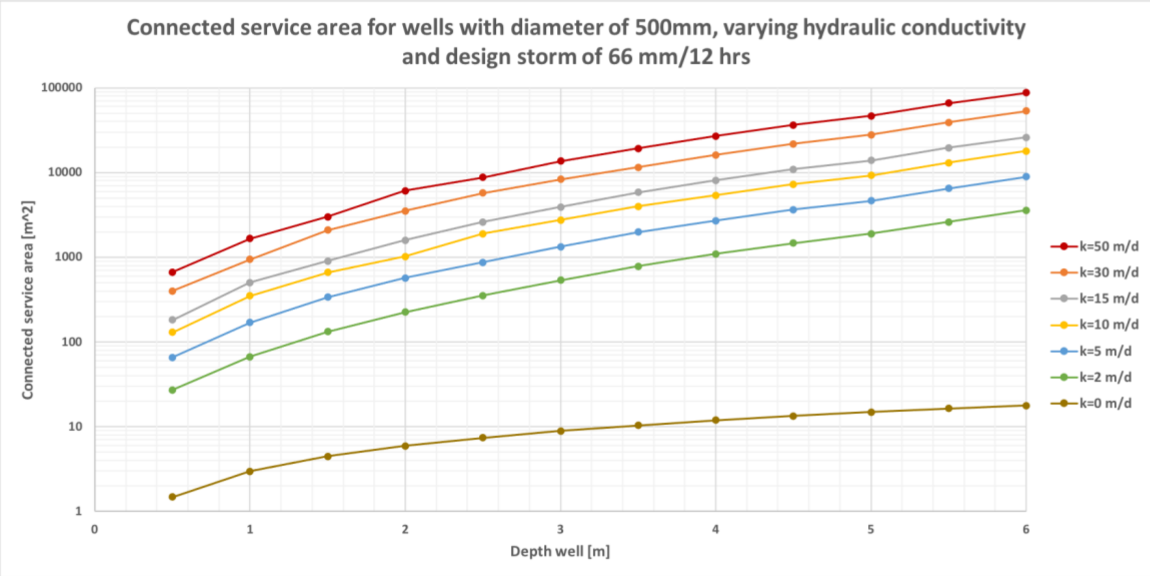


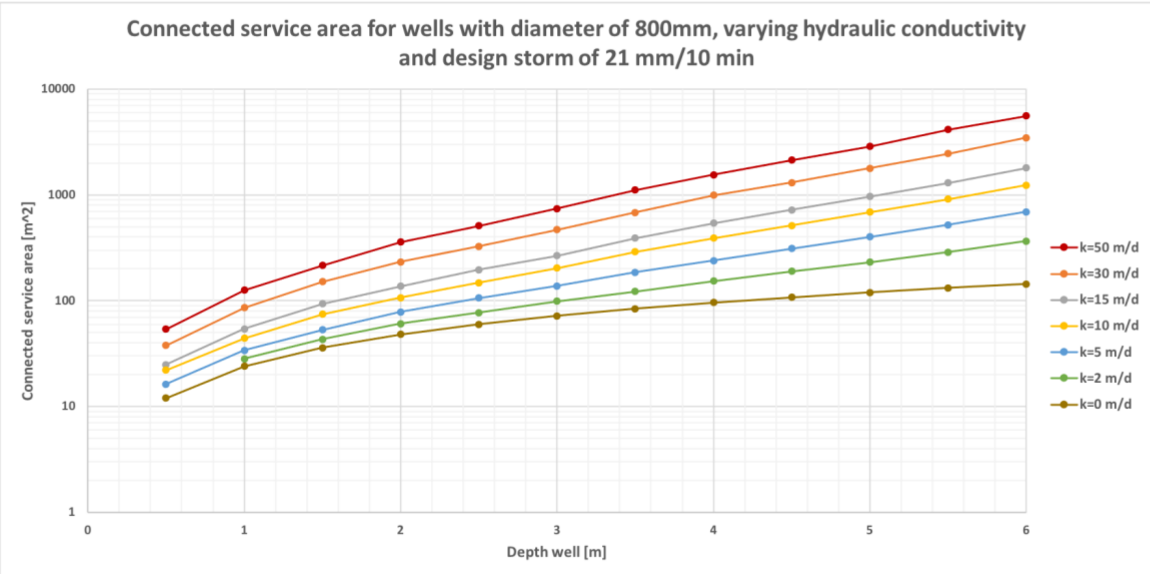


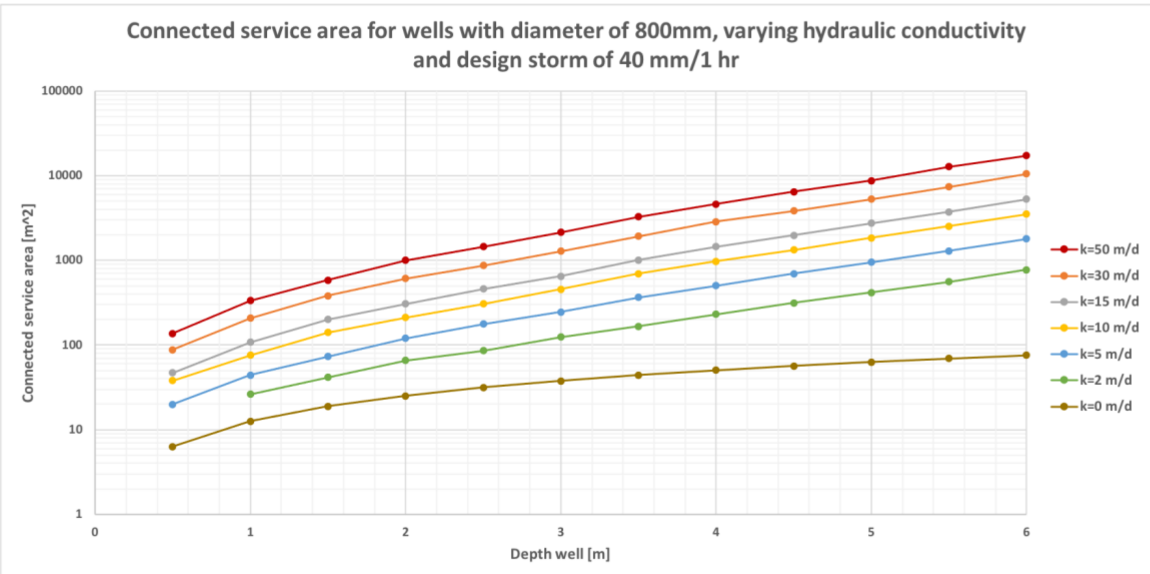

Connected service area for wells with diameter of 300mm, varying hydraulic conductivity and design storm of 66 mm/12 hrs
Connected service area [m^2]
Depth well [m]
k=50 m/d
k=30 m/d
k=15 m/d
k=10 m/d
k=5 m/d
k=2 m/d
k=0 m/d

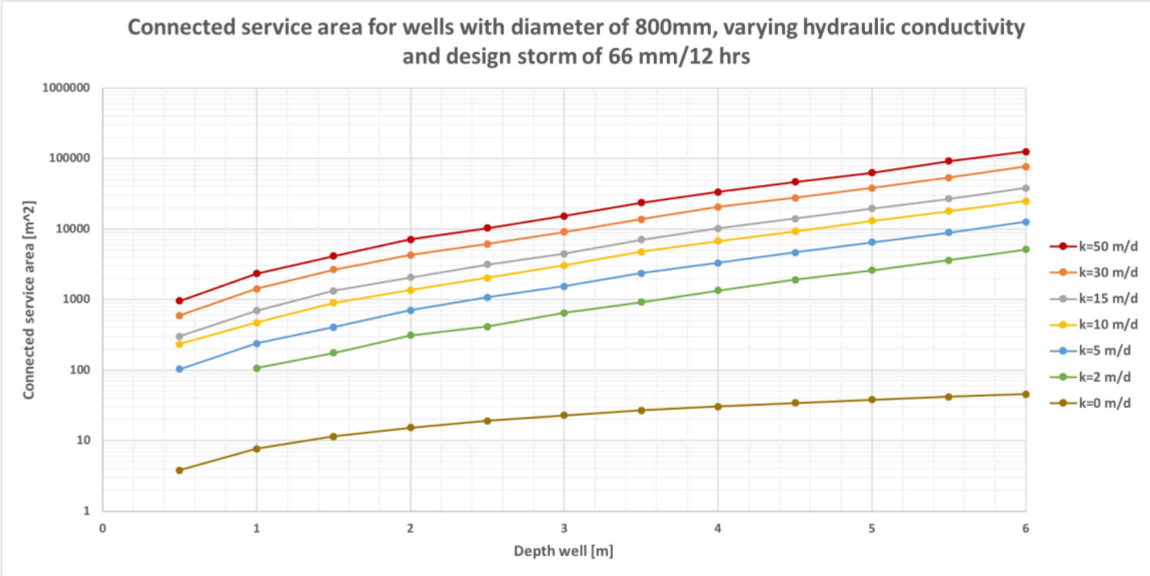

Connected service area for wells with diameter of 500mm, varying hydraulic conductivity and design storm of 21 mm/10 min
Connected service area [m^2]
Depth well [m]
k=50 m/d
k=30 m/d
k=15 m/d
k=10 m/d
k=5 m/d
k=2 m/d
k=0 m/d


Connected service area for wells with diameter of 500mm, varying hydraulic conductivity and design storm of 40 mm/1 hr
Connected service area [m^2]
Depth well [m]
k=50 m/d
k=30 m/d
k=15 m/d
k=10 m/d
k=5 m/d
k=2 m/d
k=0 m/d


Connected service area for wells with diameter of 500mm, varying hydraulic conductivity and design storm of 66 mm/12 hrs
Connected service area [m^2]
Depth well [m]
k=50 m/d
k=30 m/d
k=15 m/d
k=10 m/d
k=5 m/d
k=2 m/d
k=0 m/d

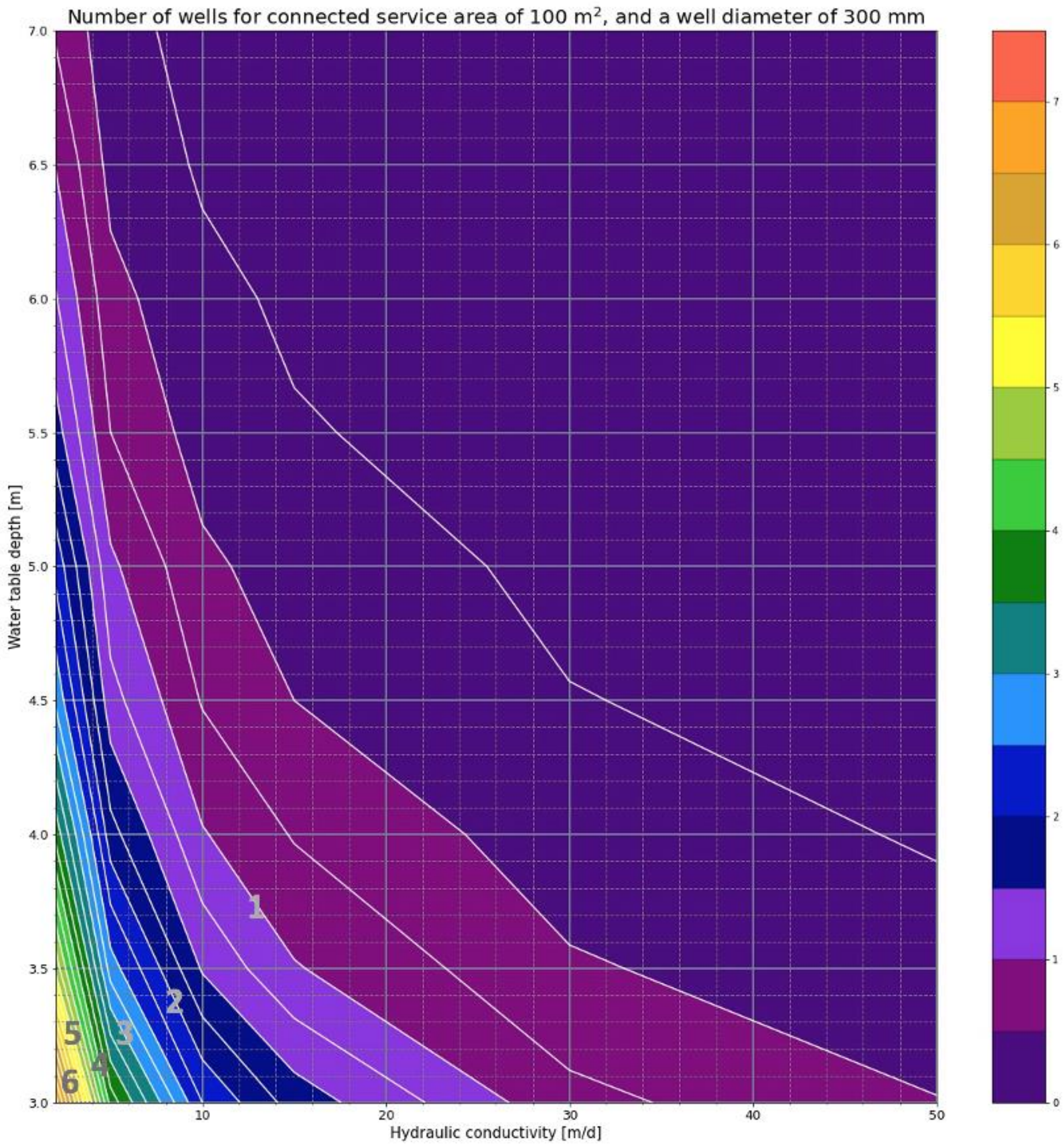

Connected service area for wells with diameter of 800mm, varying hydraulic conductivity and design storm of 21 mm/10 min
Connected service area [m^2]
Depth well [m]
k=50 m/d
k=30 m/d
k=15 m/d
k=10 m/d
k=5 m/d
k=2 m/d
k=0 m/d

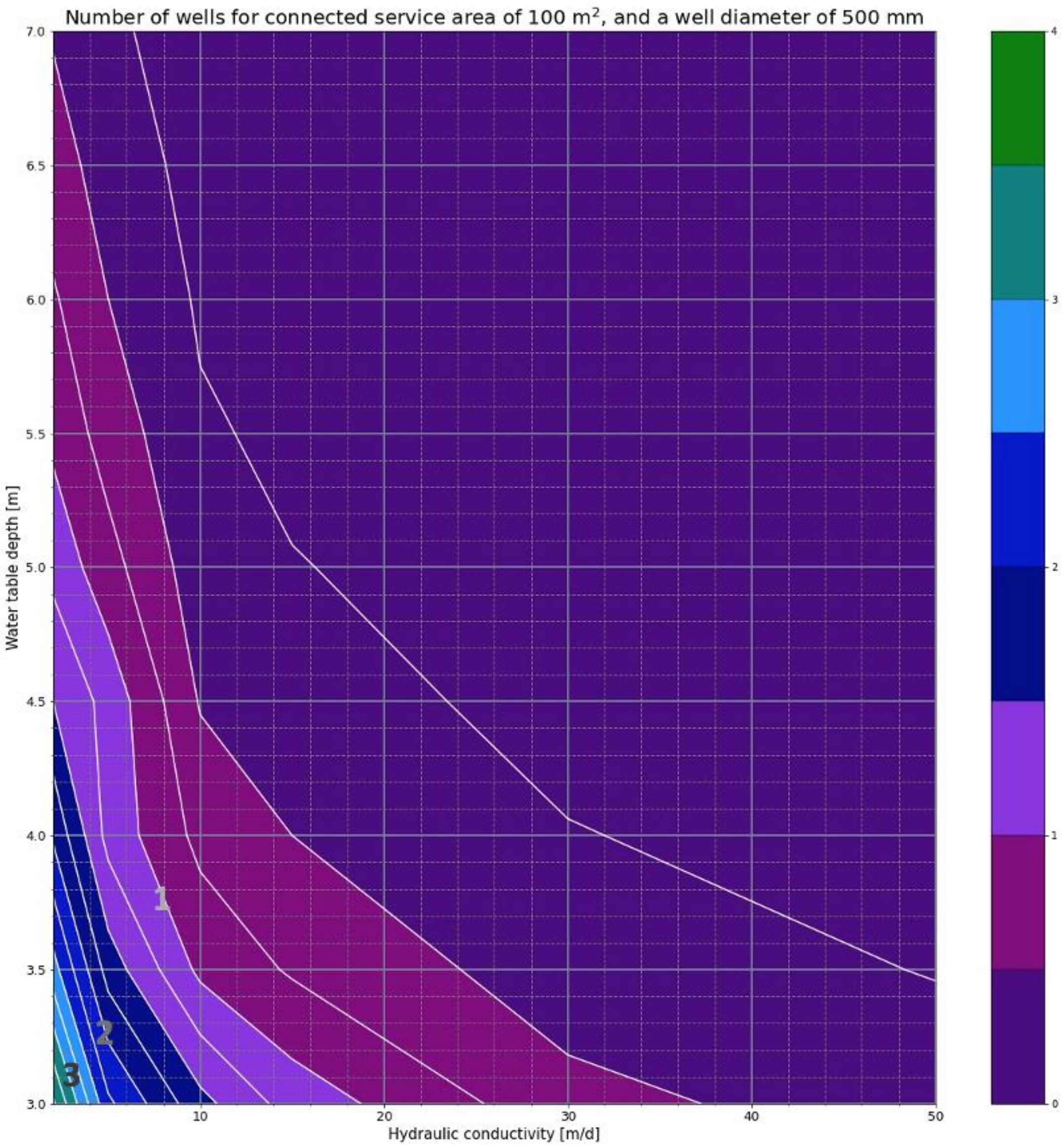

Connected service area for wells with diameter of 800mm, varying hydraulic conductivity and design storm of 40 mm/1 hr
Connected service area [m^2]
Depth well [m]
k=50 m/d
k=30 m/d
k=15 m/d
k=10 m/d
k=5 m/d
k=2 m/d
k=0 m/d


Connected service area for wells with diameter of 800mm, varying hydraulic conductivity and design storm of 66 mm/12 hrs
Connected service area [m^2]
1000000
100000
10000
1000
100
10
1
0
1
2
3
4
5
6
Depth well [m]
k=50 m/d
k=30 m/d
k=15 m/d
k=10 m/d
k=5 m/d
k=2 m/d
k=0 m/d

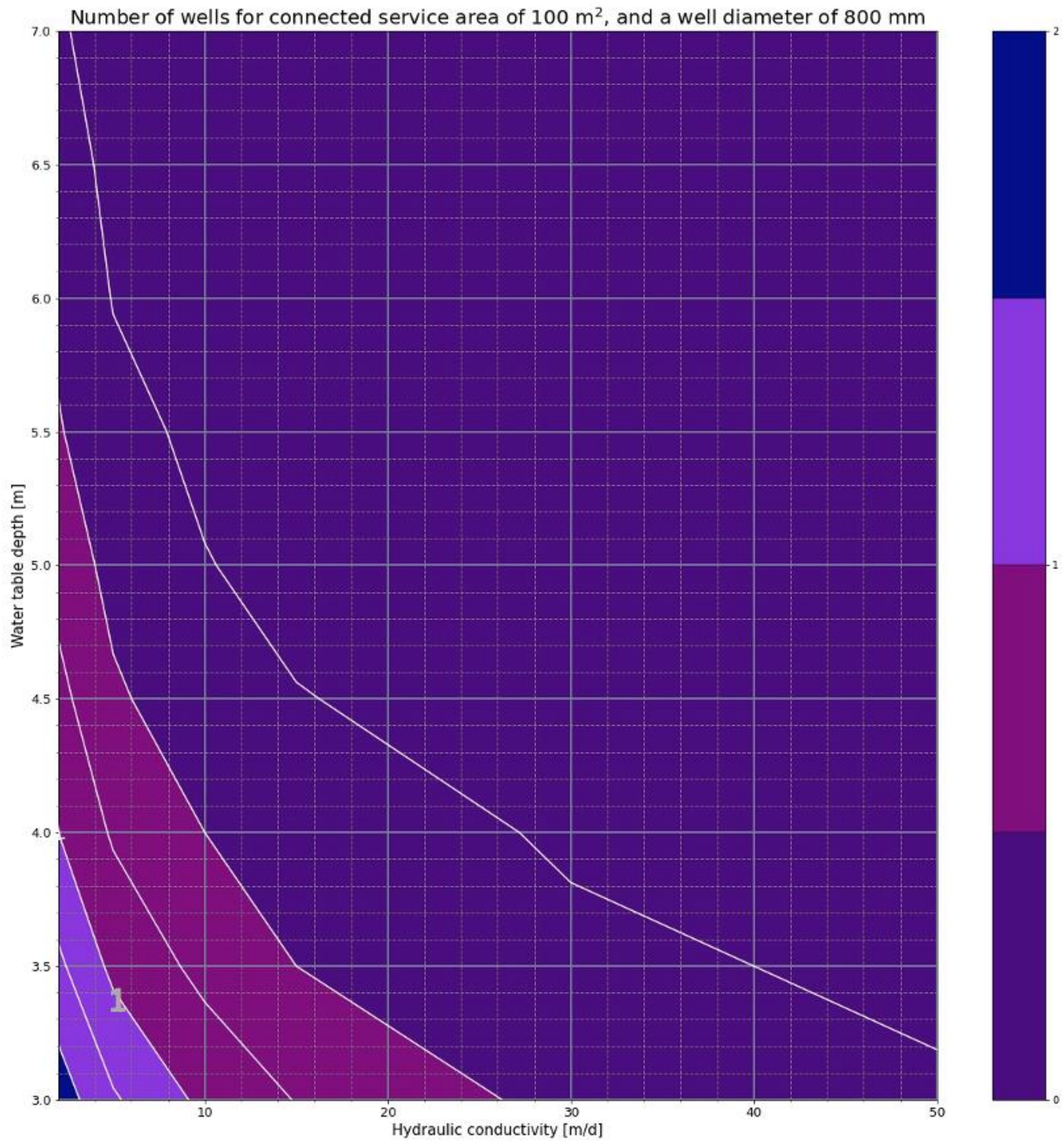
C

Contour plots for design method of all diameters



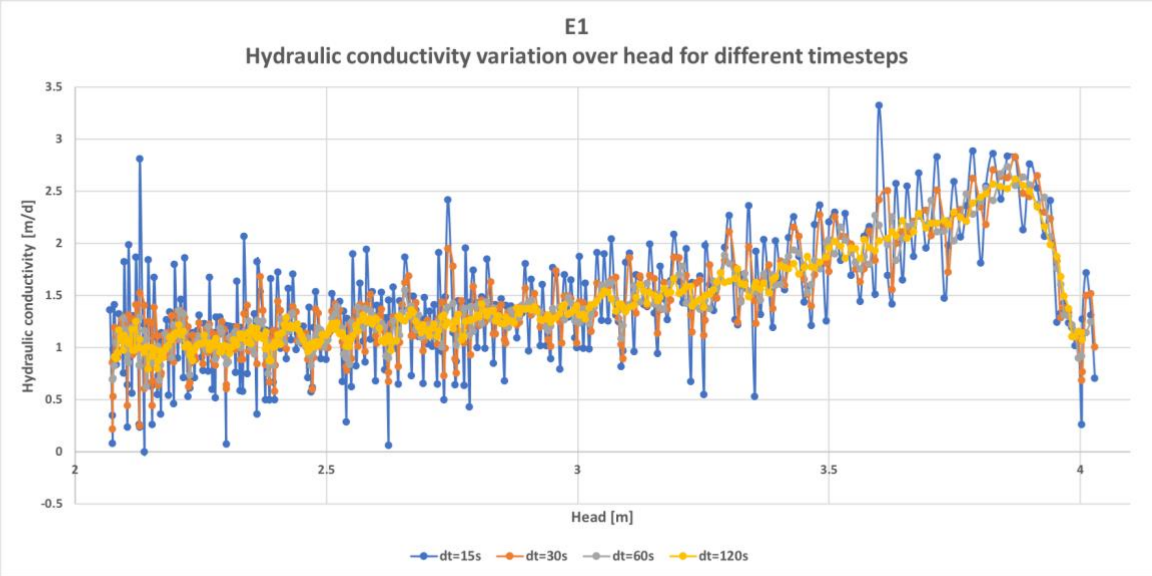


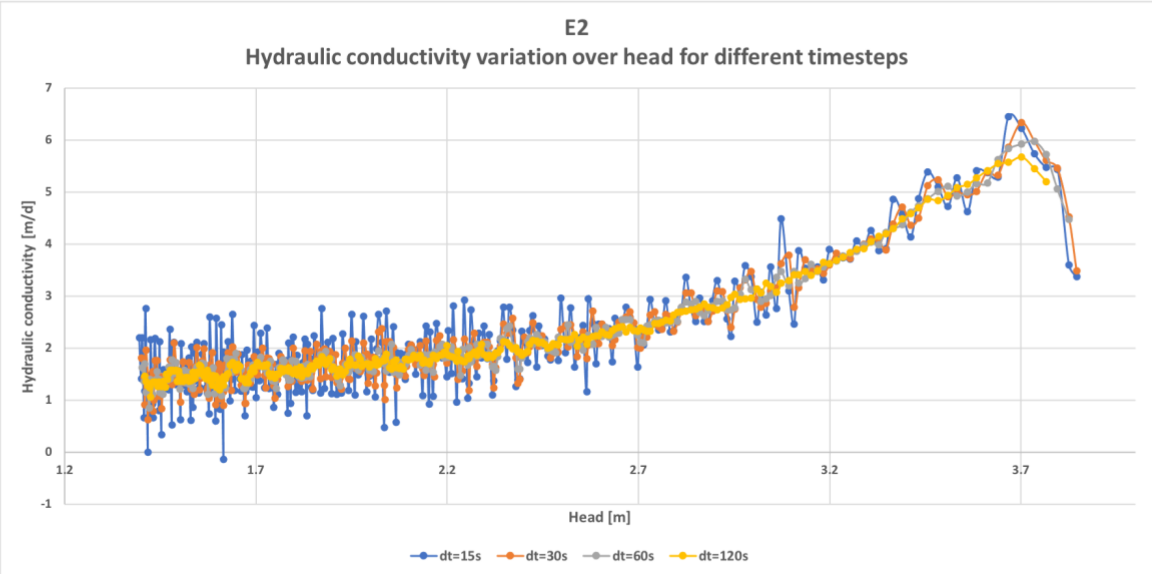
Number of wells for connected service area of 100 m², and a well diameter of 500 mm
Water table depth [m]
Hydraulic conductivity [m/d]
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
10
20
30
40
50
1
2
3
4
3
2
1
0

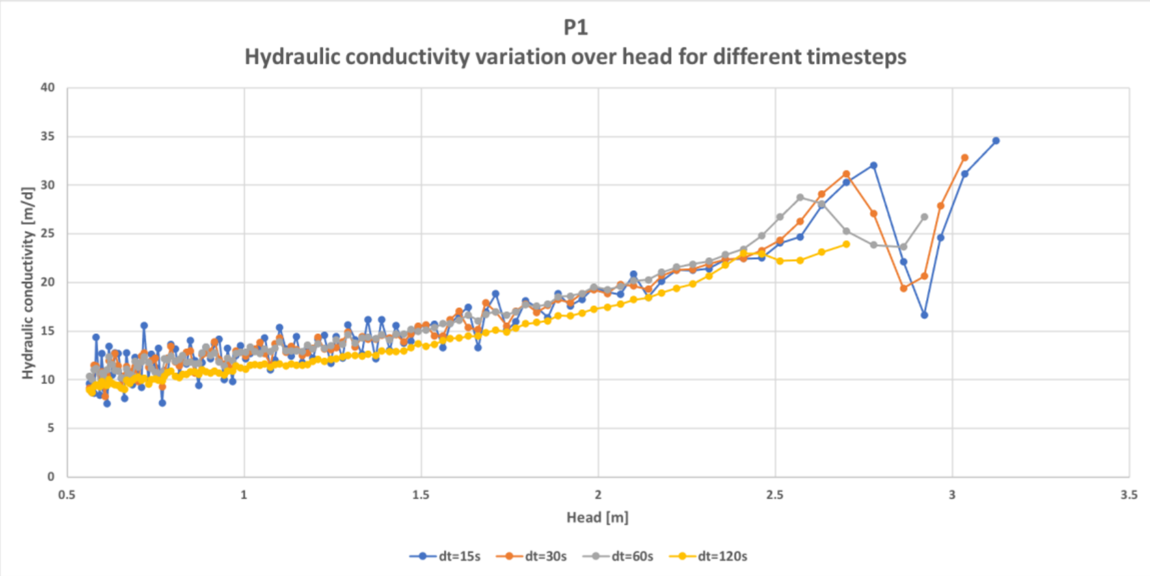

Number of wells for connected service area of 100 m², and a well diameter of 800 mm
Water table depth [m]
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
Hydraulic conductivity [m/d]
10
20
30
40
50
1
2
1
0

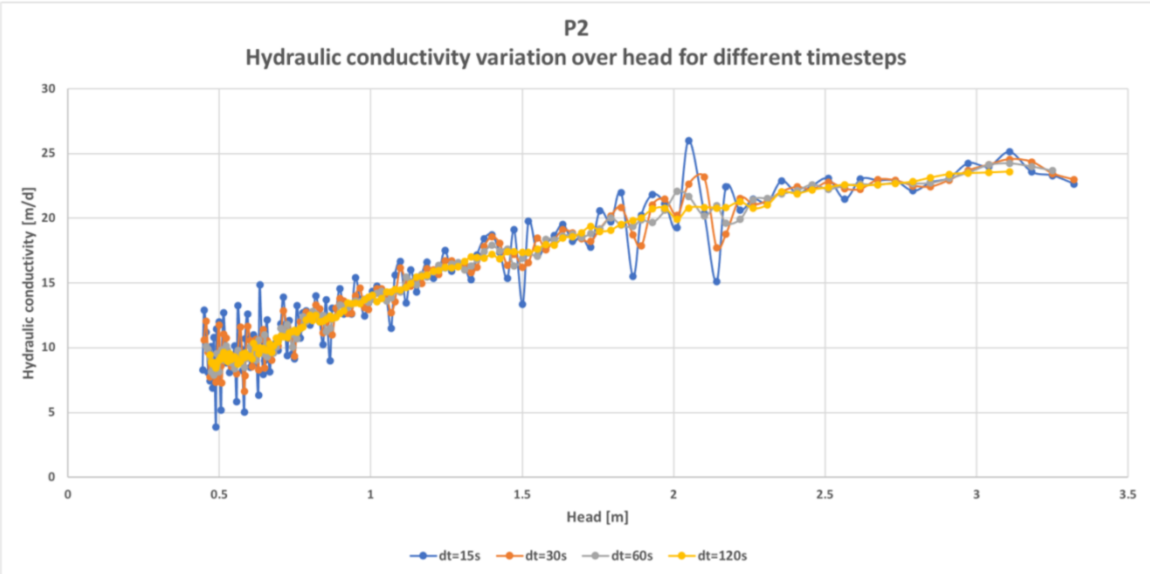
D

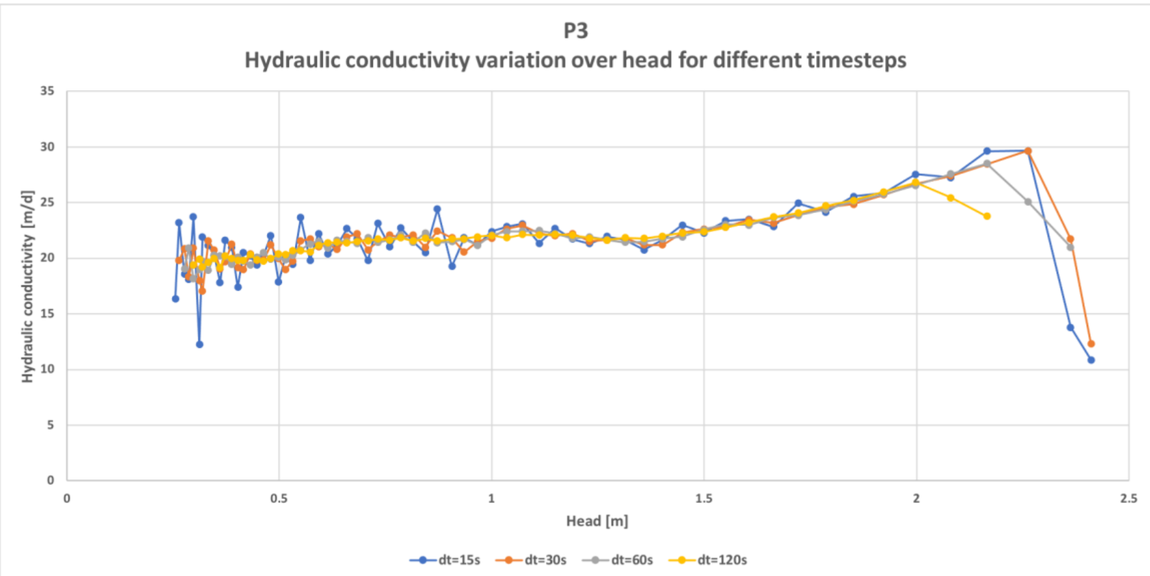
The calculated hydraulic conductivity (from IDW falling head data) for 4 different time steps plotted against the hydraulic head (water depth) in all the wells

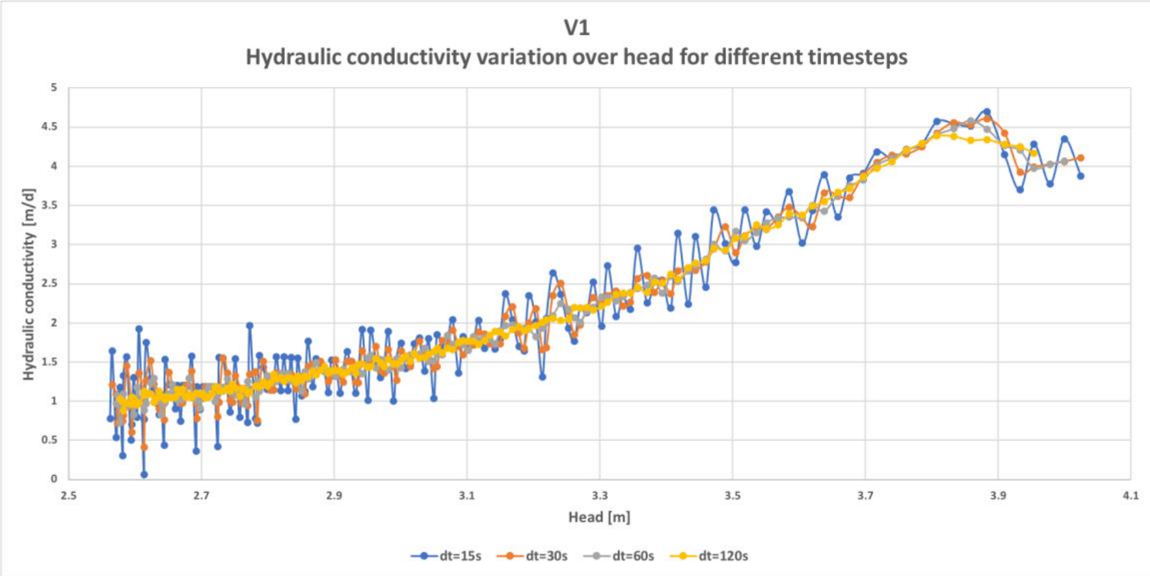


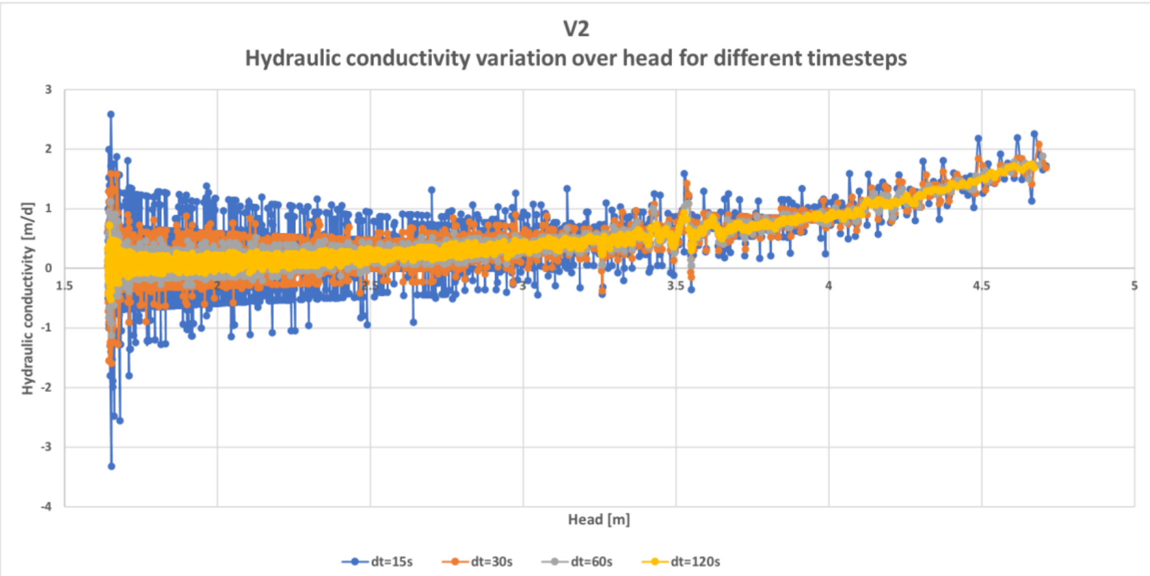



P1
Hydraulic conductivity variation over head for different timesteps
Hydraulic conductivity [m/d]
Head [m]
0
5
10
15
20
25
30
35
40
0.5
1
1.5
2
2.5
3
3.5
dt=15s
dt=30s
dt=60s
dt=120s


P2
Hydraulic conductivity variation over head for different timesteps
Hydraulic conductivity [m/d]
Head [m]
0
5
10
15
20
25
30
0
0.5
1
1.5
2
2.5
3
3.5
dt=15s
dt=30s
dt=60s
dt=120s


P3
Hydraulic conductivity variation over head for different timesteps
Hydraulic conductivity [m/d]
Head [m]
0
5
10
15
20
25
30
35
0
0.5
1
1.5
2
2.5
dt=15s
dt=30s
dt=60s
dt=120s


V1
Hydraulic conductivity variation over head for different timesteps
Hydraulic conductivity [m/d]
Head [m]
dt=15s
dt=30s
dt=60s
dt=120s


V2
Hydraulic conductivity variation over head for different timesteps
Hydraulic conductivity [m/d]
Head [m]
dt=15s
dt=30s
dt=60s
dt=120s