



# Taking advantage of aquifer heterogeneity in designing construction dewatering systems with partially penetrating recharge wells

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Received: 6 December 2019 / Accepted: 10 August 2020 / Published online: 25 August 2020  
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## Abstract

During construction dewatering, artificial recharge with wells might be required to discharge the pumped groundwater. On the one hand, artificial recharge wells must be placed as close as possible to the construction site to limit the above-ground space for the dewatering infrastructure and the transport costs, while on the other hand, the distance from the dewatering site must be considerably large to reduce the hydraulic impact and minimize overall pumping costs. Commonly, artificial recharge wells are completed with filter screens that penetrate large portions of the aquifer. The present study investigates the efficiency and potential of artificial recharge with partially penetrating wells (PPWs; filter length of 1 m) in heterogeneous aquifers. This was done by conducting scenario modeling of simple dewatering schemes with one abstraction well and one artificial recharge well, as well as with experimental field tests. In these field tests, artificial recharge on a fully penetrating well (FPW), as well as on a PPW screened at a layer of relatively high permeability (300 m/day), is explored. The present study shows that the use of recharge PPWs screened at depth in high-permeability layers of the aquifer minimize the hydraulic impact at the dewatering site. Scenario modeling shows that recharge PPWs can be installed much closer to the dewatering site than FPWs. Assessment of the optimal screen depth of the PPW, as well as the mutual distance between the wells, requires a proper consideration of the vertical variability in the hydraulic conductivity of the aquifer.

**Keywords** Construction dewatering · Artificial recharge · Heterogeneity · Injection wells · Partially penetrating wells

## Introduction

In the last few decades, the use of underground space in urban areas has drastically increased for the construction of building foundations and infrastructure (e.g. Tan and Wang 2013; Fang et al. 2016). In areas with shallow groundwater levels, or in deep excavation pits, construction dewatering is widely used for lowering the water-table inside the excavation pit (Powers et al. 2007; Cashman and Preene 2013; Pujades et al. 2014). In general, dewatering procedures affect the design, the construction procedures, and overall costs of the construction project.

Hence, dewatering systems should be carefully designed using the knowledge of the soil characteristics, hydraulic properties and construction dimensions for a specific field site (Galeati and Gambolati 1988; Wang et al. 2002; Powers et al. 2007). Poor characterization of the subsurface in the design stage of dewatering systems can result in significant failure of the system and a drastic effect on the safety of the construction works, as well as increase the project costs by highly over-dimensioning the entire dewatering system (Powers et al. 2007; Cashman and Preene 2013; Jurado et al. 2012).

Discharge options of the pumped groundwater vary depending on the project scale and duration, as well as the hydraulic and environmental impact. Water can be discharged into sewerage systems, surface waters and in the subsurface by artificial recharge. Artificial recharge is done in the same dewatered aquifer, or in other target aquifers. In many countries, governing and environmental laws could largely restrict the recharge options in surface waters for dewatering designs (Powers et al. 2007; Preene and Brassington 2003). For example, dewatered water can contain suspended solids (e.g.

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precipitated iron oxides) which are highly visible and potentially harmful to aquatic flora and fauna. Potential petroleum and contaminant residues in the aquifer from leakage and spill events are a potential risk to the environment and require extensive, costly treatment if they are abstracted during dewatering, while the mobilization of such contaminants in the aquifer during dewatering have to be avoided (e.g. Preene and Brassington 2003). Often, the pumped groundwater from the construction site must be recharged into the same aquifer close to the construction site to minimize the overall hydraulic and environmental impact (Cashman and Preene 2013).

To obtain an optimal dewatering design, it is key to reduce both the hydraulic impact on shallow groundwater levels outside the dewatered excavation site, as well as the hydraulic impact due to artificial recharge. For example, the old city centres of Copenhagen (Denmark) and Amsterdam (The Netherlands) are built on weak soils and there is a strict ban on water-table lowering to limit potential damage to the wooden foundations of old buildings (Cashman and Preene 2013). Especially for large dewatered groundwater volumes at deep excavation sites, the hydraulic impact outside the excavation pit and the risk of settlement by groundwater lowering is large (e.g. Forth 2004; Roy and Robinson 2009; Pujades et al. 2017). A popular method to reduce the hydraulic impact of a dewatering project and prevent large groundwater influx rates in the excavation site involves the use of sealing curtains, such as sheet piling, diaphragm/slurry walls and soil grouting that penetrate partially into the pumped aquifer (Wang et al. 2013; Pujades et al. 2014; Wu et al. 2017). Consequently, the pumping-induced drawdown and settlement outside the excavation pit are reduced. However, vertical sealing curtains become cost-prohibitive for smaller shallow open or trench excavations where stable sides around the construction site can be attained throughout the entire construction operation to minimize the construction costs.

In addition, to mitigate groundwater lowering outside the dewatered site, artificial recharge by infiltration wells is used in the same aquifer to prevent ground settlement outside the excavation (Zhang et al. 2017; Zeng et al. 2019). However, the shallow groundwater-level rise due to artificial recharge should be taken into account to exclude potential damaging effects on surrounding underground building structures and infrastructure.

The negative hydraulic impact of artificial recharge near the excavation site on water-table lowering during dewatering is key for dewatering design. Higher pumping costs due to increased groundwater influx from the artificial recharge site into the dewatered excavation site need to be taken into account. In practice, the artificial recharge wells screen large parts of the aquifer to maximize recharge efficiency and minimize the rate of clogging over time (Powers et al. 2007). For planning and predicting the hydraulic impact of small

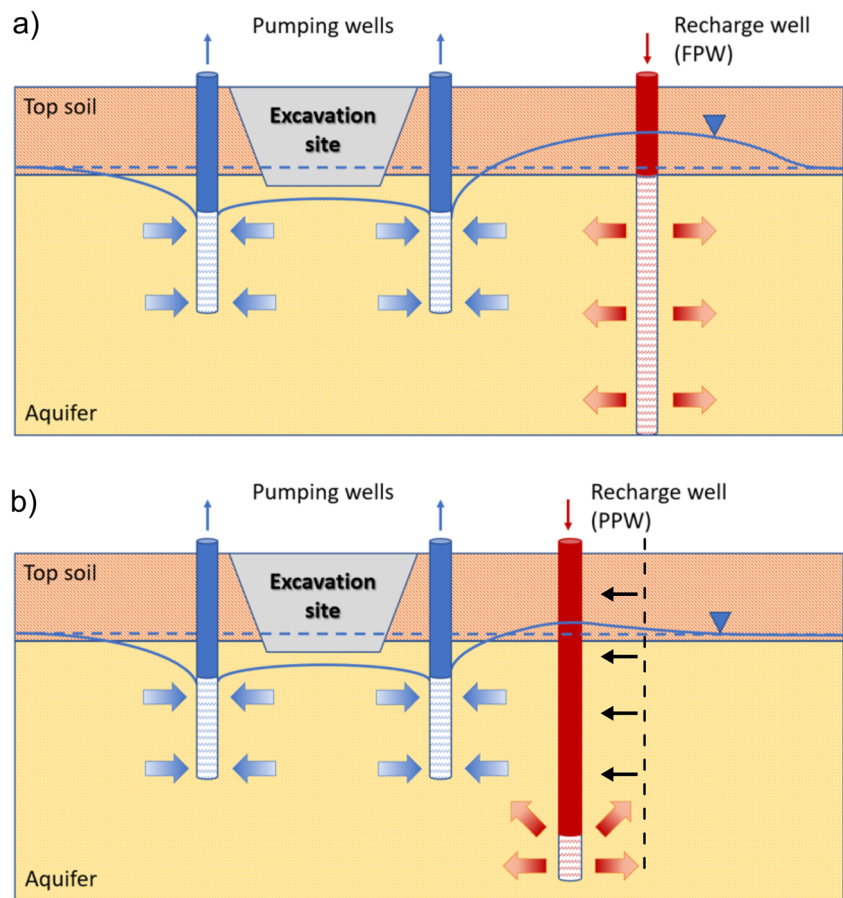
dewatering projects, two-dimensional (2D) analytical solutions or numerical models usually suffice since groundwater flow is strictly in the lateral direction in the entire aquifer while using such artificial recharge wells (Fig. 1a; e.g. Phien-vej et al. 1998; Powers et al. 2007; Zheng et al. 2018). The disadvantage of using fully penetrating wells (FPWs) for artificial recharge is that the hydraulic impact on shallow groundwater levels is relatively high. Hence, artificial recharge wells are ideally installed downstream of the extraction site at considerable distance such that the pumping and infrastructure costs are at an optimum (Powers et al. 2007; Cashman and Preene 2013). Particularly, in urbanized areas the available space at surface level is also a limiting factor in obtaining an optimal location for the artificial recharge.

The placement of partially penetrating wells (PPWs) in deeper parts of the aquifer can be beneficial compared to FPWs due to reduced hydraulic impact in the upper part of the aquifer (Fig. 1b). Using infiltration PPWs in a given portion of the aquifer results in a hydraulic gradient in both the lateral and vertical direction. Hence, groundwater flow also has a vertical flow component in the vicinity of the well (e.g. Barker and Herbert 1992; Houben 2015). In addition, anisotropy (lower vertical permeability) of aquifers results in preferential lateral flow paths at depth and hampers flow in the vertical direction. Consequently, reduced head rise at shallow groundwater levels can be obtained with artificial recharge by PPWs in deeper portions of the aquifer compared to equivalent recharge with FPWs, allowing for closer placement of artificial recharge PPWs to the dewatering site compared to FPWs (Fig. 1).

In dewatering projects, dewatering PPWs screened in the upper part of the aquifer are commonly used to target the hydraulic impact and reduce the shallow groundwater levels at a minimum of pumping costs (Powers et al. 2007). Dewatering systems for small excavation sites use well point systems at shallow aquifer depths, mostly in combination with diesel-powered pumps, to obtain the desired water-table lowering. For deep excavation sites and larger pumped groundwater volumes, deep wells with submersible pumps are used at targeted depth (Powers et al. 2007; Cashman and Preene 2013). Wang et al. (2013) and Wu et al. (2017) show that the use of partially penetrating pumping wells embedded by sealing curtains can change the lateral flow to optimize the dewatering design. Jin et al. (2016) suggest the use of a vertical circulation well to minimize the hydraulic impact at shallow subsurface levels during dewatering.

However, to the authors' knowledge, the use of artificial recharge PPWs to obtain the hydraulic impact at a targeted aquifer depth with favourable hydraulic characteristics during construction dewatering has rarely been done in practice. The conventional approach considers artificial recharge wells that screen large portions of the aquifer, while these wells need to be placed at considerable distance to ensure limited hydraulic

**Fig. 1** Schematic overview of construction dewatering with **a** a fully penetrating well (FPW) and **b** a partially penetrating well (PPW). Considering no background groundwater flow, the initial water table prior to the dewatering is given by the dashed line. The water table during combined pumping and artificial recharge is given by the solid line



impact on the dewatered excavation sites. Moreover, dewatering companies and consultancies often use simplified modeling schemes that do not account for detailed aquifer heterogeneity to design their combined construction dewatering and artificial recharge systems. In the last decade, dewatering companies in the Netherlands and Germany have started to use PPWs screened in deeper, high-permeability layers of the aquifer, which allow injection at larger well heads and volumetric flow rates, and lead to a more targeted hydraulic impact in the aquifer (Van Lopik 2020).

Therefore, in the present study, the potential use of recharge PPWs with limited screen lengths (1 m) instead of FPWs are tested experimentally and numerically for optimization of construction dewatering designs. At first, the potential of artificial recharge with PPWs is tested numerically with a simplified modelling scheme considering one abstraction well (dewatering) and one artificial recharge well (Fig. 1). The modeling is performed, focusing on different kinds of scenarios to calculate the hydraulic impact at the shallow groundwater level:

- 1 Different PPW and FPW configurations, assuming different horizontal distances between abstraction and recharge well, as well as different recharge depths for the PPW in a homogeneous anisotropic high-permeability aquifer.
- 2 Different aquifer characteristics of the target aquifer (e.g. aquifer thickness, unconfined/confined and anisotropy).
- 3 Heterogeneity. Often, predictive groundwater models for dewatering systems consider homogenous anisotropic aquifer characteristics and make use of one standard anisotropy ratio to account for aquifer heterogeneity. However, in practice, recharge PPWs will have filter screens in high-permeability layers in order to optimize dewatering systems. Hence, scenarios with heterogeneous layering in the target aquifer are considered, by accounting for vertical variability in the hydraulic conductivity instead of assuming one equivalent homogeneous anisotropic aquifer.

The efficiency and potential of artificial recharge with a PPW with limited filter length was tested experimentally in a heterogeneous layered sandy aquifer with constant recharge-rate tests. Similarly, constant recharge-rate tests on a fully penetrating well (FPW) that screened the entire depth of this target aquifer were performed. The spatial three-dimensional (3D) hydraulic head distribution of the PPW was measured and compared with the FPW. The importance of selecting the optimal screen depth of a PPW based on the aquifer heterogeneity is taken into account to minimize the required well head during recharge. The impact on the near-well hydraulic heads

using the PPW in the heterogenous layered target aquifer was compared to an hypothetical homogeneous aquifer by numerical simulations.

## Theory

In general, the Dupuit-Forchheimer assumption can be used for wells that screen a large part of an unconfined aquifer (Bear 2007). This assumption postulates that flow towards and from a well is lateral, and all vertical flow components are ignored. Many classical analytical solutions for stationary flow from and towards wells have been determined (e.g. for a confined aquifer by Thiem (1870) and for a semi-confined aquifer by De Glee (1930) and Hantush and Jacob (1955)). The De Glee-Hantush-Jacob equation is:

$$\Delta h(r) = \frac{Q_{in}}{2\pi KH} K_0\left(\frac{r}{L}\right) \quad (1)$$

where  $\Delta h(r)$  [m] is the hydraulic head increase,  $Q_{in}$  [m<sup>3</sup>/day] is the constant injection rate,  $K$  [m/day] is the horizontal overall conductivity of the aquifer,  $H$  [m] is the aquifer thickness,  $r$  [m] is the radial distance and  $L$  [m] is the leakage factor, which is defined as:

$$L = \sqrt{kHc} \quad (2)$$

where  $c$  [days] is the vertical resistance of the aquitard. The analytical De Glee-Hantush-Jacob equation (Eq. 1) is used to predict the FPW constant recharge-rate test result in the semi-confined aquifer at the field site. This is done to provide good estimates of the overall horizontal hydraulic conductivity of the target aquifer at the field site.

In the vicinity of PPWs, the vertical flow components close to the well screen are significant and therefore the Dupuit-Forchheimer assumption does not hold. Hence, Eq. (1) cannot be used for analysis of the constant recharge-rate tests with the PPWs (e.g. Tügel et al. 2016). The extent of the vertical flow component around the well depends on the characteristics of the aquifer (thickness, hydraulic conductivity, heterogeneity) and the well screen length.

## Field test on recharge wells

### Site and soil description

The fieldwork area is located near Rheindahlen (Fig. 2), about 10 km southwest of Mönchengladbach (Germany). The recharge tests were conducted prior to a dewatering project for the construction of an underground influx channel for discharge of rainwater from the urban area of Mönchengladbach to a rainwater retention basin.

The top soil consists of loam and loess deposits. Based on multiple small ramming drilling samples, up to a depth of 8 m below ground level (mbgl), the hydraulic conductivity of the top soil is in the range of 0.1–0.4 m/day. At the field site location, these deposits extend to a depth of approximately 5 m. The aquifer at the field site location is 20 m thick, with its bottom at approximately 25 mbgl. The aquifer consists of Pleistocene river terrace sediments of the Rhine River. The aquifer increases in thickness in the northeast direction with its bottom at 20 mbgl (15 m thickness) in the southwest of the area, and at 30 mbgl (25 m thickness) in the northeast of the area (Gell and Partner GbR, personal communication, 2009).

Soil samples were taken over the entire aquifer depth (up to 25 mbgl) at the recharge well location with a sampling interval of 0.5 m (Fig. 3). The hydraulic conductivity was estimated from the grain size  $d_{50}$  using the Kozeny-Carman relation:

$$K = \frac{g\rho}{\mu} \frac{d_{50}^2}{180} \frac{n^3}{(1-n)^2} \quad (3)$$

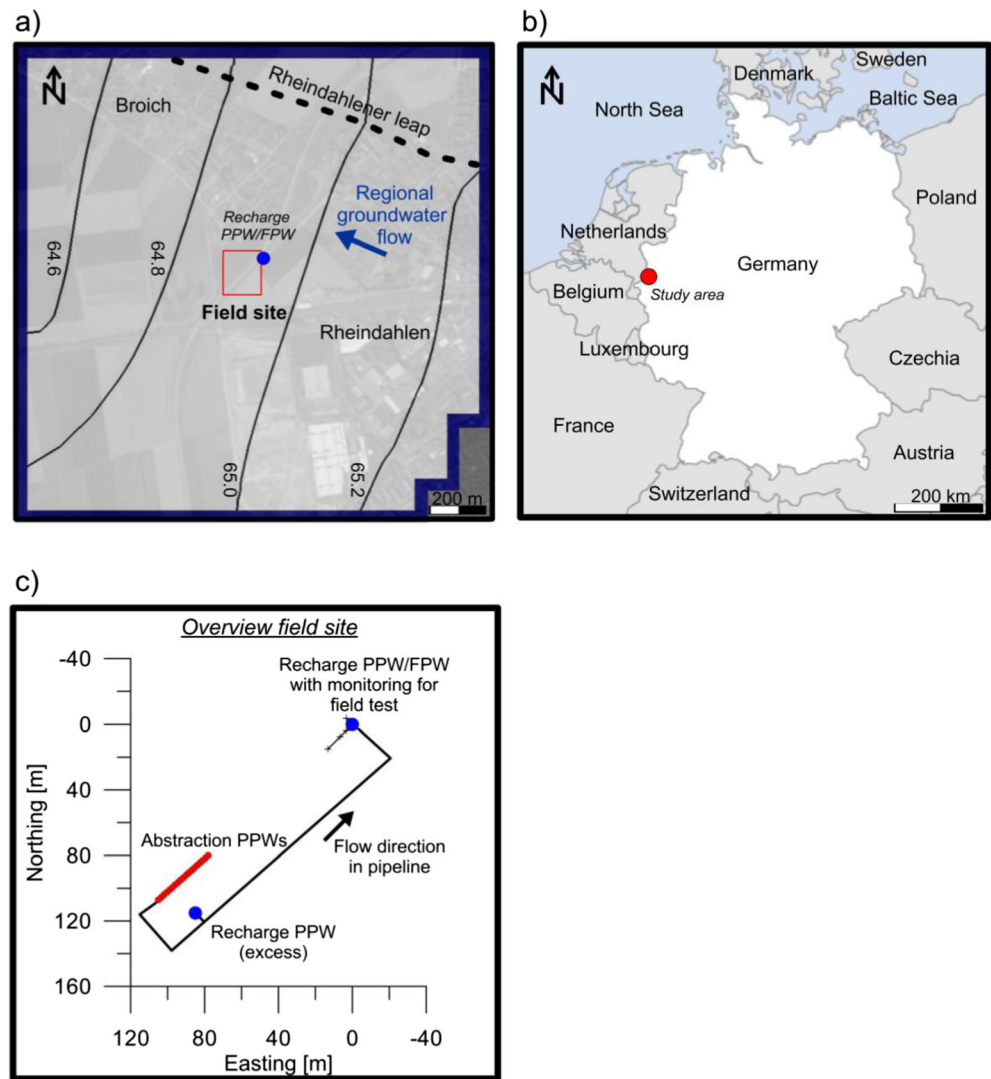
where  $\rho$  [kg/m<sup>3</sup>] is the fluid density,  $\mu$  [kg/(m s)] is the dynamic viscosity,  $g$  [m/s<sup>2</sup>] is the gravity acceleration constant,  $d_{50}$  [m] is the median grain size of the porous medium and  $n$  [–] is the porosity. An average porosity of 0.32 is assumed. The estimated hydraulic conductivity over depth is shown in Fig. 3c. According to the soil data, the average hydraulic conductivity of the aquifer is 73 m/day. However, the target aquifer is very heterogeneous, ranging from sandy fine/medium gravel layers with estimated hydraulic conductivities up to 480 m/day up to silty fine sand with estimated hydraulic conductivities around 1 m/day. Heterogeneous layering of the aquifer (pancake model) can be assumed based on borehole logs in the near-well vicinity of the PPW/FPW test site. The target aquifer is underlain by a confining unit of Pliocene clay of the Reuver formation of approximately 3–5 m thickness with a hydraulic conductivity smaller than  $9 \times 10^{-4}$  m/day. The top of this confining unit is considered as the base of the model described here.

The study area is located at the Venloer graben of the Lower Rhine Graben system. Northeast from the study area, the Rheindahlener leap extends into the Quaternary river terrace sediments of the target aquifer, resulting in a clear geological offset with Miocene marine fine sands ( $K \sim 1$  m/day) starting at a depth of approximately 7 mbgl at the northern part of the leap (Gell and Partner GbR, personal communication, 2009). The groundwater flow is in the northwest direction (see Fig. 2).

### Arrangement of the recharge and observation wells

The layout of the abstraction and recharge wells used for the recharge tests is shown in Fig. 2c. A vacuum dewatering system was used to pump groundwater from a set of abstraction

**Fig. 2** **a** The field area with the hydraulic head contours (meters above sea level, masl), and the regional groundwater flow direction. In the centre, the test site is shown with the exact well location of the recharge PPW/FPW. Hydraulic head contours are displayed for the first aquifer [masl]. **b** Field site location of Rheindahlen. **c** Schematic plan view of the recharge test site with the location of the used abstraction and recharge PPWs, as well as the monitoring wells



PPWs, screened at a depth of 6–8 mbgl, to the recharge well at the test site (Recharge PPW/FPW in Fig. 2c). Excess groundwater was recharged in the well labelled PPW (excess) in Fig. 2c, which is screened at 19.5–20.5 mbgl. The tested recharge PPW and FPW, as well as the observation points, are placed at a distance of 110 m from the abstraction PPWs. This was done to avoid hydraulic impact by dewatering on the recharge well during the aquifer tests. For this field experiment, first a recharge PPW was installed and tested, whereas subsequently, this recharge PPW was replaced by a recharge FPW.

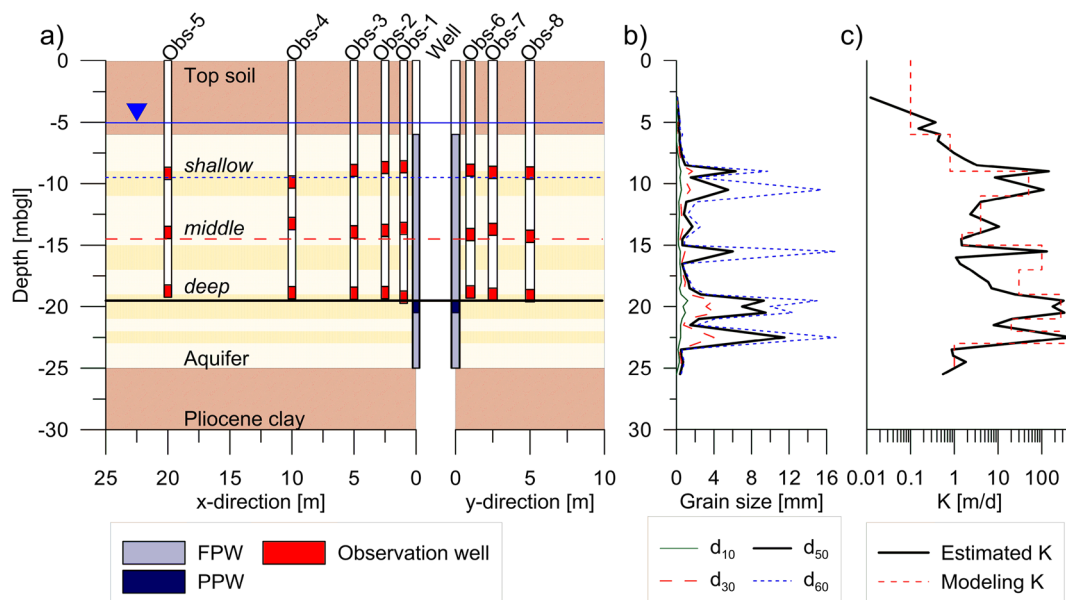
### Installation of the recharge PPW

Initially, a borehole was drilled for placement of the PPW using straight-flush rotary drilling with clean water as drilling fluid. The naturally developed PPW ( $\varnothing 110$  mm) without a filter pack was installed in the target aquifer with its filter screen in the high-permeability gravel layer at 19.5–20.5 mbgl (see Fig. 3). Prior to these recharge tests, an airlift pump was

used to increase the well efficiency of the PPW. Initially, the performance of the PPW was quite bad, resulting in relatively low discharge rates (up to only  $15 \text{ m}^3/\text{h}$ ). Airlifting was applied for approximately 1 hour, resulting in an increased well performance (enabling discharge rates up to  $25 \text{ m}^3/\text{h}$ ). Constant recharge rates in the recharge PPW (Fig. 3) were applied at approximately 23, 19.5 and  $14 \text{ m}^3/\text{h}$  for cases 1.P, 2.P and 3.P respectively (see Appendix).

### Installation of the recharge FPW

Subsequently, the PPW was pulled up and replaced by the FPW in the target aquifer. At the same location a borehole ( $\varnothing 300$  mm) was drilled using reversed-circulation rotary drilling. A conventional infiltration well ( $\varnothing 250$  mm) with filter pack (1.0–1.6 mm) screened between 5 and 25 (mbgl) was placed. A bentonite clay plug was used for the upper 5 m in the top soil. Several recharge tests at constant recharge rates of



**Fig. 3** **a** Cross-section of the soil in the *x*- and *y*-direction, where the observation wells 1–5 are aligned in the *x*-direction, while 6–8 are aligned in the *y*-direction (see Fig. 2b). The solid black (deep), dashed red (middle) and dashed blue (shallow) lines indicate the monitoring depths in the aquifer at 9.5, 14.5 and 18.5 m respectively. **b** The

determined grain sizes  $d_{10}$ ,  $d_{30}$ ,  $d_{50}$  and  $d_{60}$  of the soil samples by sieve analysis. **c** The hydraulic conductivity over depth estimated by Kozeny-Carman, as well as the hydraulic conductivity used for the numerical model

$32.3 \text{ m}^3/\text{h}$  (case 1.F) and  $16.7 \text{ m}^3/\text{h}$  (case 2.F) were conducted on the FPW.

### Monitoring configuration

In order to measure the hydraulic impact around the infiltration well, eight boreholes were drilled. In each borehole, three monitoring wells ( $\varnothing 32 \text{ mm}$ ) were placed, each with 1 m of filter screen length. The monitoring levels were at shallow (9.5 mbgl), middle (14.5 mbgl) and deep (19.5 mbgl) aquifer depths (Fig. 3a). Observation wells 1–5 were placed from northeast to southwest and observation wells 6–8 were placed from southeast to northwest (Fig. 2c). During each infiltration test the hydraulic head was monitored with pressure transducers (model 14 Mini-diver DI501, Schlumberger) with a measurement accuracy of 0.5 cmH<sub>2</sub>O. Manual hydraulic head measurements were done to check the diver data during the infiltration tests.

### Modeling set-up

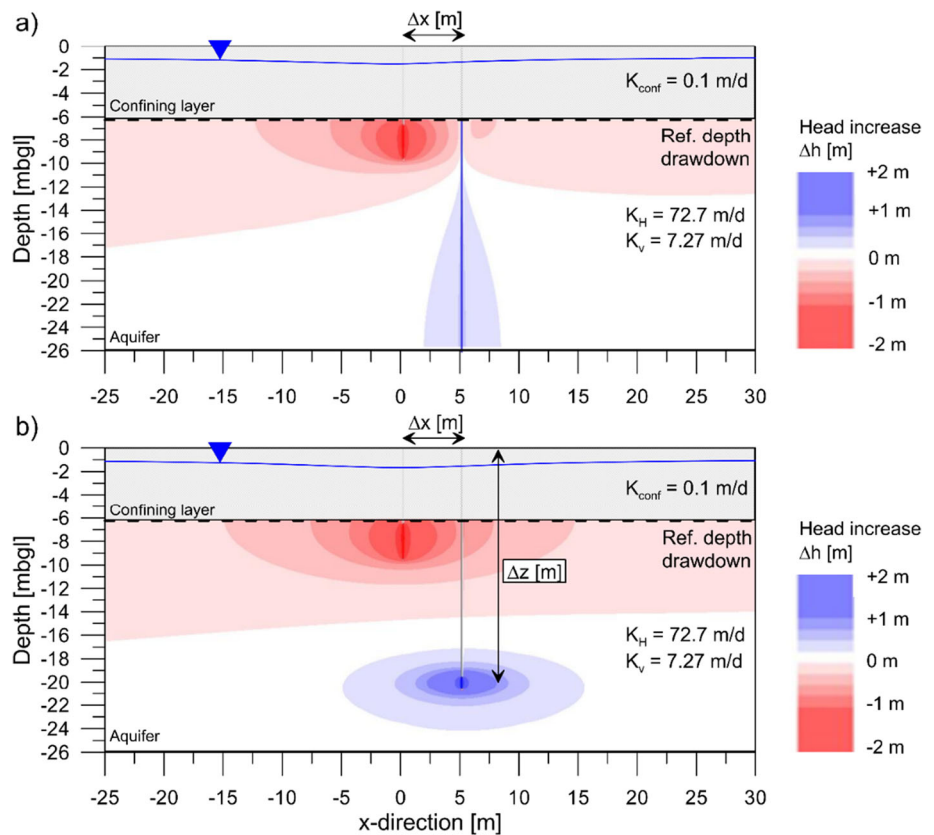
#### Scenario modeling on the efficiency of recharge PPW

The 3D steady-state groundwater flow in the simplified construction dewatering scheme is simulated with the groundwater model MODFLOW-2000 (Harbaugh et al. 2000). The groundwater flow is solved using the preconditioned conjugate gradient 2 (PCG2) solver. The

model domain is  $1.5 \times 1.5 \text{ km}$ . A semi-confined aquifer of 20 m thickness is assumed, overlain by a 6-m-thick confining unit (hydraulic conductivity of 0.1 m/day). No background groundwater flow is considered. Simplified dewatering systems with one abstraction well in the top of the semi-confined aquifer and PPWs/FPWs at various recharge depths ( $\Delta z$ ) and distances ( $\Delta x$ ) from the abstraction well were tested numerically in the centre of the model domain (see Fig. 4). The screen length of the abstraction PPW is 3 m and screens between 6.5 and 9.5 mbgl, while the length of the artificial recharge PPW is 1 m and screens at various depths with the centre of the well screen at depth ( $\Delta z$ ). For this model, a refined lateral and vertical discretization ( $\Delta x$ ,  $\Delta y$  and  $\Delta z = 0.1 \text{ m}$ ) was used in the vicinity of the wells to allow for steep hydraulic gradients in the zone of recharge and abstraction, with increasing grid sizes towards the model boundary (up to  $\Delta x$  and  $\Delta y$  of 100 m and  $\Delta z$  of 1 m). Vertical refinement of the mesh was only used for the PPWs, with  $\Delta z$  of 0.1 m ranging from 0.5 above the PPW to 0.5 m below the PPW and  $\Delta z$  of 0.5 m ranging from 0.5 to 1.5 m above, as well as below the PPW. At aquifer depths that are only screened by a FPW, the vertical grid refinement  $\Delta z$  is 1 m.

Equal volumetric abstraction and recharge rates ( $10 < Q < 50 \text{ m}^3/\text{h}$ ) are used to model combined abstraction for dewatering and artificial recharge. The drawdown  $s$  is calculated at shallow aquifer depth (6.25 m) to account for the hydraulic impact due to recharge with the different recharge well configurations (see Fig. 4). The ratio  $\lambda$  between

**Fig. 4** Head increase ( $\Delta h$ ) contours for combined abstraction at shallow aquifer depth and artificial recharge at a recharge rate of  $40 \text{ m}^3/\text{h}$  with **a** a FPW and **b** a PPW. This cross-section is the base case 1.1. The lateral distance between the two wells is indicated by  $\Delta x$  and the screen depth of the PPW is indicated by  $\Delta z$  for the cases 1.1–1.4



calculated drawdown ( $s$ ) and the volumetric abstraction/recharge rate is determined:

$$\lambda = s/Q \quad (4)$$

where  $\lambda$  [ $\text{h}/\text{m}^2$ ] is the relative drawdown. The relative drawdown is calculated for assessment of the scenario modeling and for scaling to required drawdowns for realistic volumetric abstraction/recharge rate in dewatering systems.

The model parameters used for the different cases are listed in Table 1. First, a detailed overview is given to analyze the effect on the drawdown in the upper portion of the aquifer using recharge PPWs in a homogeneous anisotropic semi-confined aquifer (based on the hydraulic characteristics of the field site aquifer, see section ‘Site and soil description’). Cases 1.1–1.4 were simulated to show the effect of varying the distance ( $\Delta x$ ) between the pumping well and PPW at the given recharge depths ( $10 < \Delta z < 25$  mbgl). This was done for distances  $\Delta x$  of 5, 10, 20 and 50 m.

Besides the dimensional aspect of the dewatering design with artificial recharge PPWs (cases 1.1–1.4), the hydraulic impact while using an artificial recharge PPW at  $\Delta x = 20$  and  $\Delta z = 20$  m instead of a FPW in different kinds of aquifers is analyzed. This is done for an unconfined aquifer (case 2), a fully confined aquifer (case 3), a thinner and thicker aquifer thickness (cases 4–5), an aquifer with a higher and lower hydraulic conductivity (cases 6–7; Table 1). In addition, the

effect of overall aquifer anisotropy on the dewatering-recharge system is tested (see cases 8–9).

In nature, aquifers are heterogeneous and the vertical variation in hydraulic conductivity can be large. However, too often, conceptual models are considering homogenous anisotropic aquifer characteristics and make use of one standard anisotropy ratio (i.e. anisotropy ratio of 2–10 (Kasnow 2010), such as is done in cases 1–9) to account for aquifer heterogeneity. However, the full-potential of PPWs, in order to optimize dewatering systems, should account for a much more detailed approach. Ideally, drilling operators and dewatering companies will complete PPWs in relatively high-permeability layers in the aquifer to improve the specific well capacity of the PPW. Hence, the effect of heterogeneous layering in a high-permeability aquifer ( $K_h = 72.7$  m/day) is investigated for dewatering and recharge with PPWs in the conceptual model (cases 10–14, see Table 2). In these cases, a hypothetical set of target aquifers is selected, all containing horizontal layers that alternate in hydraulic conductivity over depth and with an equal average horizontal conductivity as used in the reference case 1. Various high-permeability layers of 300 (cases 10–12 and 14) and 500 m/day (case 13) are considered as target layers for artificial recharge with a PPW.

The aim of this part of the scenario modeling is to assess the implications of simplifying the target aquifer to one homogeneous anisotropic aquifer. For each case, the heterogeneous

**Table 1** The aquifer characteristics for different types of homogeneous aquifers used for the scenario modeling. Values in italic represent the adjusted parameter values in the modelling of the different scenarios. The adjusted parameter values for the various cases:  $\Delta x$  [m] is the

distance between the pumping well and artificial recharge PPW,  $K_h$  [m/d] is the horizontal hydraulic conductivity,  $K_v$  [m/d] is the vertical hydraulic conductivity,  $a$  [-] is the anisotropy ratio and  $H_{aq}$  [m] is the aquifer thickness

Case	Adjusted parameter value	Top soil		Aquifer			
		$K_h$ [m/day]	$K_v$ [m/day]	$H_{aq}$ [m]	$a$ ( $K_h/K_v$ )	$K_h$ [m/day]	$K_v$ [m/day]
Case 1	Ref. scenario	0.1	0.1	20	10	72.7	7.27
Case 1.1	$\Delta x = 5$ m	0.1	0.1	20	10	72.7	7.27
Case 1.2	$\Delta x = 10$ m	0.1	0.1	20	10	72.7	7.27
Case 1.3	$\Delta x = 20$ m	0.1	0.1	20	10	72.7	7.27
Case 1.4	$\Delta x = 50$ m	0.1	0.1	20	10	72.7	7.27
Case 2	Unconfined aquifer	72.7	7.27	20	10	72.7	7.27
Case 3	Confined aquifer	–	–	20	10	72.7	7.27
Case 4	$H_{aq} = 15$ m	0.1	0.1	15	10	72.7	7.27
Case 5	$H_{aq} = 25$ m	0.1	0.1	25	10	72.7	7.27
Case 6	$K = 10$ m/day	0.1	0.1	10	10	10	1
Case 7	$K = 5$ m/day	0.1	0.1	10	10	5	0.5
Case 8	Isotropy ( $a = 1$ )	0.1	0.1	10	1	72.7	72.7
Case 9	Anisotropy ( $a = 2$ )	0.1	0.1	10	2	72.7	36.35

**Table 2** The hydraulic conductivities of the layered heterogeneous aquifer and the equivalent homogeneous anisotropic aquifer

Case	Heterogeneous layered aquifer case (10.1–14.1)				Homogeneous anisotropic aquifer case (10.2–14.2)	
	$H_{aq}$ [m]		$K_h$ [m/day]	$K_v$ [m/day]	$K_h$ (Eq. 5) [m/day]	$K_v$ (Eq. 6) [m/day]
	Top	Bottom				
Case 10	6	19	47.45	47.45	72.7	51.81
	19	21	300	300		
	21	26	47.45	47.45		
Case 11	6	11	72.7	72.7	72.7	47.62
	11	19	37.73	37.73		
	19	21	300	300		
	21	26	37.73	37.73		
Case 12	6	11	72.7	72.7	72.7	8.44
	11	12	40.84	40.84		
	12	13	0.5	0.5		
	13	19	40.84	40.84		
	19	21	300	300		
	21	26	40.84	40.84		
Case 13	6	11	72.7	72.7	72.7	10.31
	11	19	6.96	6.96		
	19	21	500	500		
	21	26	6.96	6.96		
Case 14	6	11	72.7	7.27	72.7	4.762
	11	19	37.73	3.773		
	19	21	300	30		
	21	26	37.73	3.773		



layering of the target aquifer is considered, as well as the scenario with an equivalent homogeneous anisotropic aquifer. Isotropic layers in the target aquifer are considered for cases 10–13 (see Table 2). In case 14, the same heterogeneous layering as in case 11 is assumed considering anisotropy of 10.

Often, the variation in permeabilities of stratified aquifers with altering sand or gravel layers is simplified to an equivalent homogeneous anisotropic aquifer (e.g. Kasenow 2010). The horizontal conductivity ( $K_h$ ) of a layered aquifer is the arithmetic mean of the hydraulic conductivity of each layer:

$$K_h = \frac{\sum(K_{h,i}b_i)}{H} \quad (5)$$

where,  $H$  [m] is the aquifer thickness,  $b$  [m] is the thickness of layer  $i$ ,  $K_{h,i}$  [m/day] is the horizontal of layer  $i$ . The vertical hydraulic conductivity ( $K_v$ ) can be calculated using the thickness-weighted harmonic mean:

$$K_v = \frac{H}{\sum \frac{b_i}{K_{v,i}}} \quad (6)$$

where  $K_{v,i}$  is the vertical hydraulic conductivity of layer  $i$ .

### Numerical analysis of field data

Similar to the numerical set-up for the scenario modeling, a model domain of  $1.5 \times 1.5$  km is used (see Fig. 2). The PPW and FPW recharge tests are simulated assuming the derived aquifer heterogeneity as shown in Fig. 3c, as well as an equivalent homogeneous aquifer with the average calculated hydraulic conductivity (Eq. 5) of 72.7 m/day (see Table 3). The aquifer characteristics and well locations as described in section ‘Site and soil description’ are used. Constant head boundaries results in an equivalent background groundwater

flow as observed in the field. Again, lateral and vertical grid refinement ( $\Delta x$ ,  $\Delta y$  and  $\Delta z = 0.1$  m) was used in the vicinity of the wells to allow for steep hydraulic gradients in this zone, which resulted in convergence of the numerical solution. Grid refinement of 1 m for  $\Delta x$  and  $\Delta y$  is applied at  $660 < l_x < 810$  and  $700 < l_y < 860$  m.

Both the discharge rate at the abstraction PPWs and the recharge rate at the PPWs and FPW are given in Table 4 for all cases. A constant head has been used for numerical simulation of steady-state recharge. This allows for nonuniformly distributed flow rate over the entire well screen, as is usually the case for PPWs and heterogeneous aquifers (e.g. Ruud and Kabala 1997; Houben and Hauschild 2011; Houben 2015; Tügel et al. 2016).

## Results

The results of this study are presented in two parts. First, the numerical results from the scenario modeling are presented, in which various simple pumping-recharge set-ups in different aquifer types and with PPW configurations are analyzed. This is done to predict the potential hydraulic impact reduction using recharge PPWs instead of FPWs for dewatering designs. It can be used for upscaling to dewatering procedures at the field scale with multiple abstraction and recharge wells. Finally, the efficiency of the proposed PPW configuration (of 1-m filter length) in the numerical scenario modeling is tested experimentally in a heterogeneous target aquifer. The monitored hydraulic head rise in the near-well vicinity at constant recharge rate for both experimentally tested FPW and PPW configurations are analyzed numerically, while assessing the importance of aquifer heterogeneity.

### Scenario modeling on the efficiency of PPWs during dewatering

#### The spatial 3D use of recharge PPWs

The effect of artificial recharge on the dewatering drawdowns in the surrounding area with a FPW and PPW is tested in this section. The results showed a significant increase in draw-down at shallow aquifer levels using the PPW at a depth of 20 mbgl (Fig. 4b), compared to the use of a FPW at similar distance  $\Delta x$  from the pumping well (Fig. 4a). The multiple numerical simulations (cases 1.1–4) done for the recharge PPWs placed at different  $\Delta x$  and  $\Delta z$  show that for PPWs screened at depths of  $\Delta z > 10$  mbgl a significantly lower hydraulic impact in the vicinity of the dewatering well is obtained compared to the equivalent scenario with a FPW at a similar recharge distance  $\Delta x$  (see Fig. 5). With increasing distance from the pumping well at  $\Delta x = 50$  m, the difference of

**Table 3** Parameters used for the numerical modelling of the PPW and FPW recharge tests

Parameter	Value	Unit
Dimensions ( $l_x$ , $l_y$ )	1500, 1500	m
Depth $l_z$	0–30	m
Discretization, $\Delta x$	0.1–100	m
Discretization, $\Delta y$	0.1–100	m
Discretization, $\Delta z$	0.1–6	m
Porosity	0.32	–
$K_h$ , $K_v$ (aquitard)	0.1	m/day
$K_h$ , $K_v$ (aquifer North R. leap)	1	m/day
$K_h$ , $K_v$ (aquifer)	Homogen.	72.7
$K_h$ , $K_v$ (aquifer)	Heterogen.	0.8–480

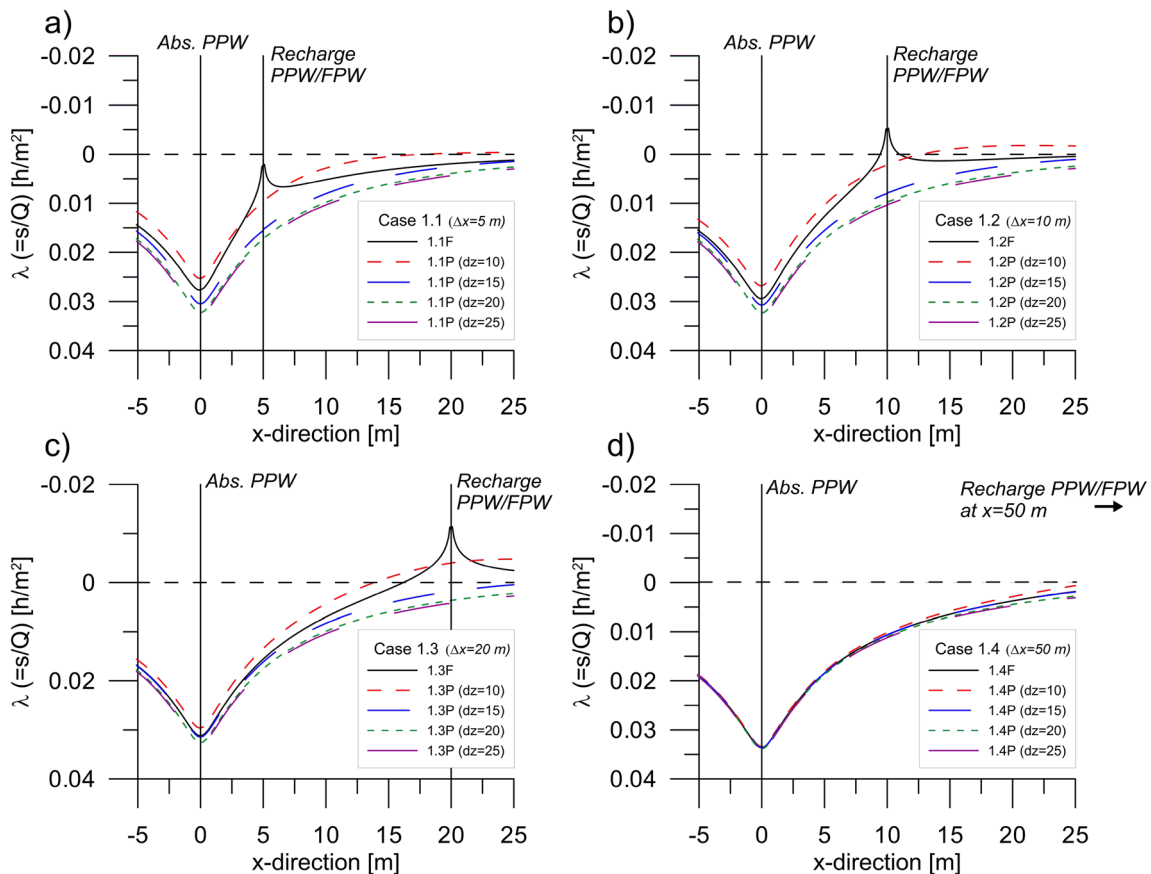
**Table 4** The discharge and recharge rates on the abstraction wells (PPWs), the excess water recharge PPW, and the recharge at PPW and FPW at the test site. The required well heads ( $\Delta h_{FPW}$  and  $\Delta h_{PPW}$ ) to simulate the recharge rates at FPW/PPW are also given

Scenario		Abstraction PPWs [ $m^3/h$ ]	Excess recharge PPW [ $m^3/h$ ]	Recharge [ $m^3/h$ ]		Well head[m]	
				FPW	PPW	FPW: $\Delta h_{FPW}$	PPW: $\Delta h_{PPW}$
<b>FPW</b>							
Case F.1	Heterogen.	-32.4	-	32.4	-	0.77	-
Case F.2	Heterogen.	-32.4	15.7	16.7	-	0.39	-
<b>PPW</b>							
Case P.1	Heterogen.	-23	-	-	23	-	1.51
	Homogen.	-23	-	-	23	-	4.66
Case P.2	Heterogen.	-23	3.5	-	19.5	-	1.28
	Homogen.	-23	3.5	-	19.5	-	3.95
Case P.3	Heterogen.	-23	9	-	14	-	0.91
	Homogen.	-23	9	-	14	-	2.8

hydraulic impact between the PPW and the FPW becomes negligible (case 1.4).

The abstraction-recharge well scheme shown in Fig. 4 can be used to roughly estimate the drawdown for a simplified

construction dewatering design, assuming the dewatering site is a bulk abstraction well and the recharge site is a bulk injection well. For example, the required drawdown within a radius of 10 m from the abstraction well(s) ( $\Delta x = 10$ ) is  $\sim 2.5$  m for



**Fig. 5** The ratio between drawdown and volumetric flow rate  $Q$  ( $\lambda = s/Q$ ) over distance  $x$  in the top of the aquifer at a depth of 6.25 mbgl. (see Fig. 4). Equal abstraction and recharge rates ( $Q$ ) are considered. The ratio  $\lambda$  is given for both the scenarios with artificial recharge using the FPW, as

well as with the PPWs (at different screen depths  $\Delta z$  ranging between 10 and 25 mbgl). This is done with the recharge PPW placed at a distance of **a**  $\Delta x = 5$  m, **b**  $\Delta x = 10$  m, **c**  $\Delta x = 20$  m and **d**  $\Delta x = 50$  m from the abstraction well

construction dewatering, while considering combined abstraction and recharge at a rate of  $Q = 250 \text{ m}^3/\text{h}$ . The recharge PPWs are screened at 20 m depth ( $\Delta z$ ) and at approximately 20 m distance ( $\Delta x$ ) from the pumping wells. Using this well configuration and ratio  $\lambda$  from Fig. 5c at  $\Delta x = 10$ , the aimed drawdown is attained ( $s_{\text{PPW}} = \lambda Q = 0.01 \times 250 = 2.5 \text{ m}$ ), while for a FPW placed at  $\Delta x = 20$  the drawdown is only 1.75 m ( $s_{\text{FPW}} = \lambda Q = 0.007 \times 250 = 1.75 \text{ m}$ ). To obtain a similar drawdown at  $\Delta x = 10 \text{ m}$  with a FPW, the required distance from the pumping wells ( $\Delta x$ ) must be 41 m (lower than the value at 50 m;  $s_{\text{FPW}} = \lambda Q = 0.0106 \times 250 = \text{m}$ ).

### Scenario modeling for homogeneous anisotropic aquifer conditions

This section describes the use of PPWs tested in different kinds of aquifers. The artificial recharge PPW is screened at  $\Delta x = 20 \text{ m}$  and  $\Delta z = 20 \text{ m}$  and compared to FPWs at a similar distance ( $\Delta x = 20 \text{ m}$ ). For all scenarios (cases 1–7) homogeneous anisotropic aquifer conditions are considered with an anisotropy of 10. Cases 1–3 show that the hydraulic impact in an unconfined aquifer significantly differs from the reference scenario (Fig. 6a). The use of recharge PPWs instead of FPWs results in a relative drawdown reduction ( $\Delta\lambda$ ) of only  $0.0017 \text{ h/m}^2$  instead of  $0.0029 \text{ h/m}^2$  for the fully and semiconfined scenarios at  $\Delta x = 10 \text{ m}$ . The reduction in hydraulic impact by using a recharge PPW at 20 mbgl instead of a FPW is higher in a thinner aquifer. The reduction in relative drawdown ( $\Delta\lambda$ ) is  $0.0036 \text{ h/m}^2$  for case 4 ( $H = 15 \text{ m}$ ), while this is only  $0.0023 \text{ h/m}^2$  for case 5 ( $H = 25 \text{ m}$ ) at a recharge distance  $\Delta x$  of 10 m (Fig. 6b). Due to decreasing aquifer thickness, the required well head of the FPW for the given volumetric recharge rate increases. Hence, recharge at depth with a PPW becomes increasingly significant and the smaller penetration ratio of the PPW at given target depth results in more reduction in hydraulic impact at shallow aquifer levels.

For the cases with different hydraulic conductivities and a given anisotropy ratio of  $a = 10$  (cases 6 and 7), the predicted drawdown by the ratio  $\lambda (=s/Q)$  at a given distance scales with Darcy's law, so  $\lambda$  for case 6 and case 1 is respectively a factor of 1/2 and 1/14.5 lower than for case 7 (Fig. 6c). However, for construction dewatering with recharge PPWs at depth, the anisotropy is an important parameter for predicting the hydraulic impact at shallow groundwater levels. Cases 8–9 show that the hydraulic impact reduction of using PPWs instead of FPWs on the dewatering site is significantly reduced for more isotropic target aquifers, with  $\Delta\lambda = 0.0006 \text{ h/m}^2$  for homogeneous aquifer conditions (case 8) and  $0.0029 \text{ h/m}^2$  for anisotropic ( $a = 10$ ) aquifer conditions (case 1; Fig. 6d). For these cases, a more spherical 3D head distribution and reduced lateral preferential flow paths are observed while using the PPW.

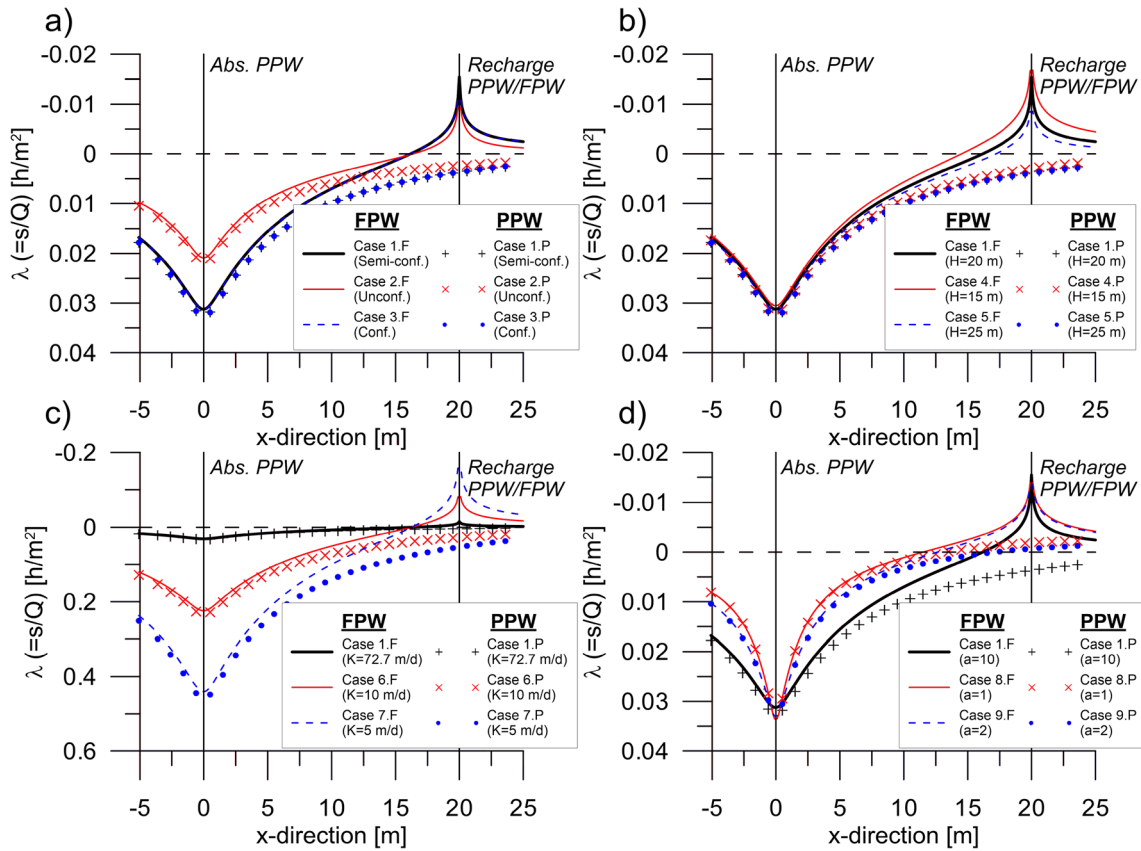
### Scenario modeling of PPWs under heterogeneous aquifer conditions

Cases 10–14 assess the importance of aquifer heterogeneity. This was done by making a clear distinction between equivalent homogeneous anisotropic aquifer conditions, which are often assumed in the design stage of smaller dewatering projects, and heterogeneous conditions with alternating layers with different hydraulic conductivities. Case 10.1 considers a high-permeability layer of 300 m/day, while one single homogeneous isotropic formation is considered for the upper and lower part of the aquifer (Table 2). The resulting hydraulic conductivity ( $K = 47.45 \text{ m/day}$ ) calculated for the upper portion of the aquifer is significantly lower than for the reference case ( $K = 72.7 \text{ m/day}$ ), while considering similar transmissivity values for both aquifers (Table 2). Hence, the drawdown due to abstraction in the upper portion of the aquifer is significantly higher around the abstraction PPW in case 10.1 (Fig. 7). Therefore, a hydraulic conductivity of  $72.7 \text{ m/day}$  is considered at pumping depth for the cases 11–14 in order to compare the drawdowns between all cases.

For Cases 11, 13 and 14, a reasonable prediction can be obtained with one single homogeneous anisotropic aquifer. Again, considering an abstraction rate at the excavation site of  $250 \text{ m}^3/\text{h}$  and recharge with PPWs at approximately 20 m distance from the dewatered area, the drawdown at  $\Delta x = 10 \text{ m}$  from the abstraction site is calculated. Using the simplified scheme as shown in Fig. 7, equivalent homogeneous anisotropic conditions instead of heterogeneous layering slightly underestimate drawdown predictions for cases 11, 13 and 14 at drawdown differences of respectively 0.09, 0.11 and 0.03 m.

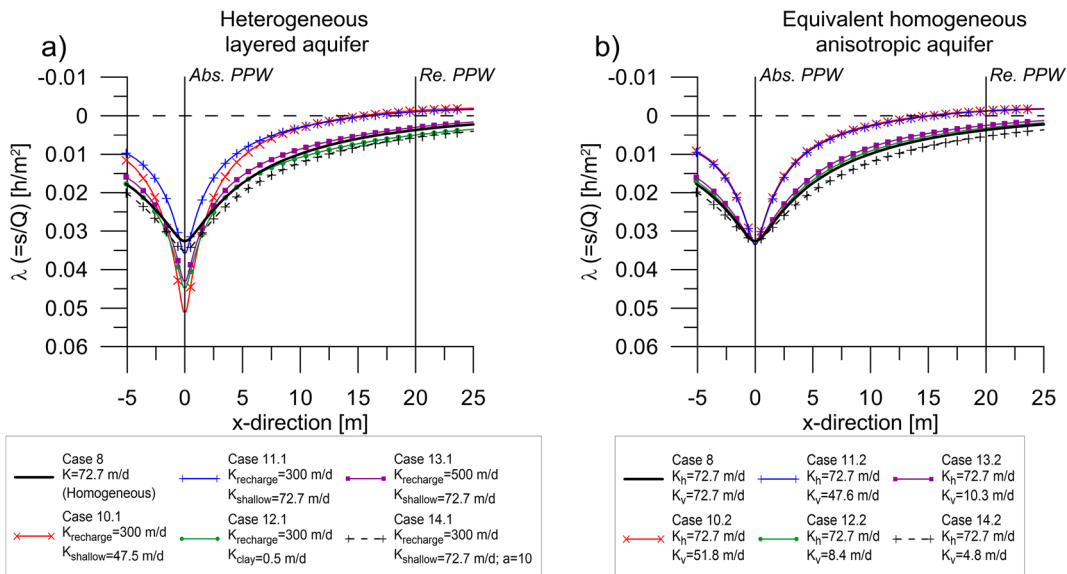
However, the required well head of the recharge PPWs screened in high-permeability layers is significantly overestimated if homogeneous anisotropic conditions are assumed for heterogeneous layered aquifers. The well head values ( $\Delta h_{\text{PPW}}$ ) are in the range of 8.3–11.5 m for the homogeneous anisotropic aquifer conditions (Table 5) and hence unfavorable well hydraulics are predicted. However, taking into account the actual high-permeability layer, reasonable well heads with values of 1.4–2.9 m are predicted.

In many groundwater models, the existence of small lenses of clay, peat, silt or loam are not considered. However, the 3D flow paths during pumping and recharge in a given target aquifer can be strongly affected by such small lenses. The calculated relative drawdown at  $\Delta x = 10$  is  $0.0092 \text{ h/m}^2$  for the anisotropic case 12.2 and  $0.0108 \text{ h/m}^2$  for the heterogeneous case 12.2 with a small clay layer. Considering an abstraction rate of  $250 \text{ m}^3/\text{h}$  at the excavation site, this results in 0.41 m more drawdown for the heterogeneous case. Therefore, the existence of small low-permeability lenses in the subsurface could have a significant reduction in hydraulic impact on the drawdown, while using PPWs instead of an



**Fig. 6** The ratio between drawdown and volumetric flow rate  $Q$  ( $\lambda = s/Q$ ) over distance  $x$  in the top of the aquifer at a depth of 6.25 mbgl (see Fig. 4). Equal abstraction and recharge rates ( $Q$ ) are considered. The recharge FPW and PPW are placed at a distance ( $\Delta x$ ) of 20 m from the pumping

well. The PPW is screened at 20 mbgl. This is done for the **a** unconfined and confined aquifer, **b** different aquifer thicknesses (cases 4–5), **c** different hydraulic conductivity values (cases 6–7), **d** different anisotropy values (cases 8–9)



**Fig. 7** The ratio between drawdown and volumetric flow rate  $Q$  ( $\lambda = s/Q$ ) over distance  $x$  in the top of the aquifer at a depth of 6.25 mbgl (see Fig. 4). Equal abstraction and recharge rates ( $Q$ ) are considered. **a** The effect on the ratio  $\lambda$  is shown for different heterogeneous layering in the target

aquifer considering an overall  $K_h$  of 72.7 for all cases 10–14.1, as well as **b** for considering a single equivalent homogeneous anisotropic aquifer for numerical simulation (cases 10–14.2)

**Table 5** Required well heads for the recharge PPW screened at  $\Delta x = 20$  m and  $\Delta z = 20$  m with volumetric recharge of  $Q = 40$  m<sup>3</sup>/h in both the heterogeneous aquifer layering and the equivalent scenario considering homogeneous anisotropic conditions (see Eqs. 5 and 6) for cases 10–14

Case	Heterogeneous aquifer characteristics	Required well head in heterogeneous layered aquifer, $\Delta h_{PPW}$ [m]	Required well head in equivalent homogeneous anisotropic aquifer, $\Delta h_{PPW}$ [m]
Case 10	$K_{recharge} = 300$ m/day	1.42	8.26
Case 11	$K_{shallow} = 47.5$ m/day	2.26	8.33
Case 12	$K_{recharge} = 300$ m/day $K_{shallow} = 72.7$ m/day	2.75	10.16
Case 13	$K_{recharge} = 300$ m/day $K_{clay} = 0.5$ m/day	1.51	9.78
Case 14	$K_{recharge} = 500$ m/day $K_{shallow} = 72.7$ m/day	2.85	11.46
Case 14	$K_{recharge} = 300$ m/day; $a = 10$ $K_{shallow} = 72.7$ m/day; $a = 10$		

FPW. This means that small heterogeneities should be considered while selecting the optimal dewatering scheme.

Overall, the results show that heterogeneous layering of the aquifer should be considered to predict the drawdown due to abstraction and artificial recharge. Especially, the hydraulic conductivity at abstraction level, where pumping wells are screened, as well as the hydraulic conductivity at recharge PPW depth should be taken into account. Section ‘Hydrogeological characterization and the recharge PPW depth’ discusses how dewatering companies identify heterogeneous layering of the aquifer with a minimum amount of hydrogeological information.

## Experimental field test

### Artificial recharge with FPWs

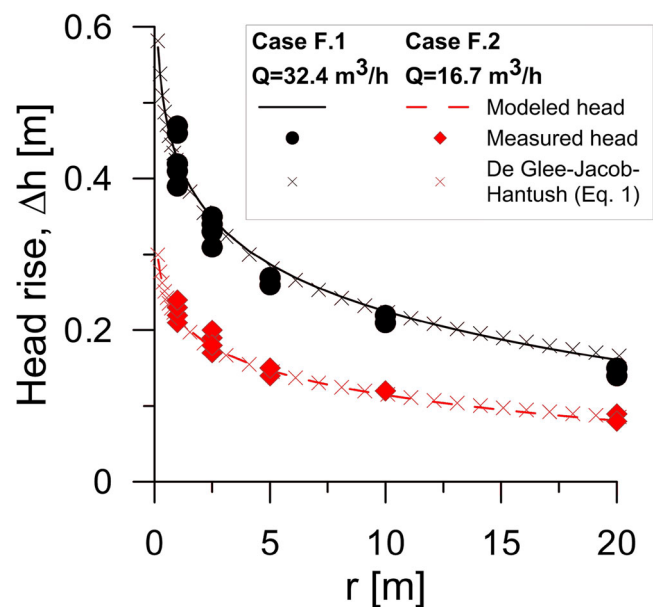
The hydraulic heads measured at the shallow, middle and deep aquifer levels for a given radial distance are approximately equal using constant recharge rates of 32.3 (case F.1) and 16.7 m<sup>3</sup>/h (case F.2) on the FPW. Hence, no vertical head gradient exists in the aquifer (Fig. 8) and the stationary flow around the FPW is strictly in the lateral direction. This means the Dupuit-Forchheimer assumption holds. For both cases, an excellent fit was obtained with the De Glee-Hantush-Jacob equation (Eq. 1), using a  $K$  of 72.7 m/day,  $c$  of 10 days, and aquifer thickness of 20 m. Despite the actual heterogeneous layering of the aquifer, the analytical approximation by considering a homogeneous semiconfined aquifer is sufficient since the hydraulic gradients over depth can be considered negligible.

Due to aquifer heterogeneity, a nonuniform screen outflow over the FPW screen length is achieved. The calculated percentage of outflow per 1-m filter is shown in Fig. 9b. Approximately 38% of the outflow occurs at the high-permeability layer at 20 mbgl that is targeted for the PPW during recharge in the FPW. In the upper portion of the aquifer the percentage of outflow is negligible.

### Artificial recharge with PPWs

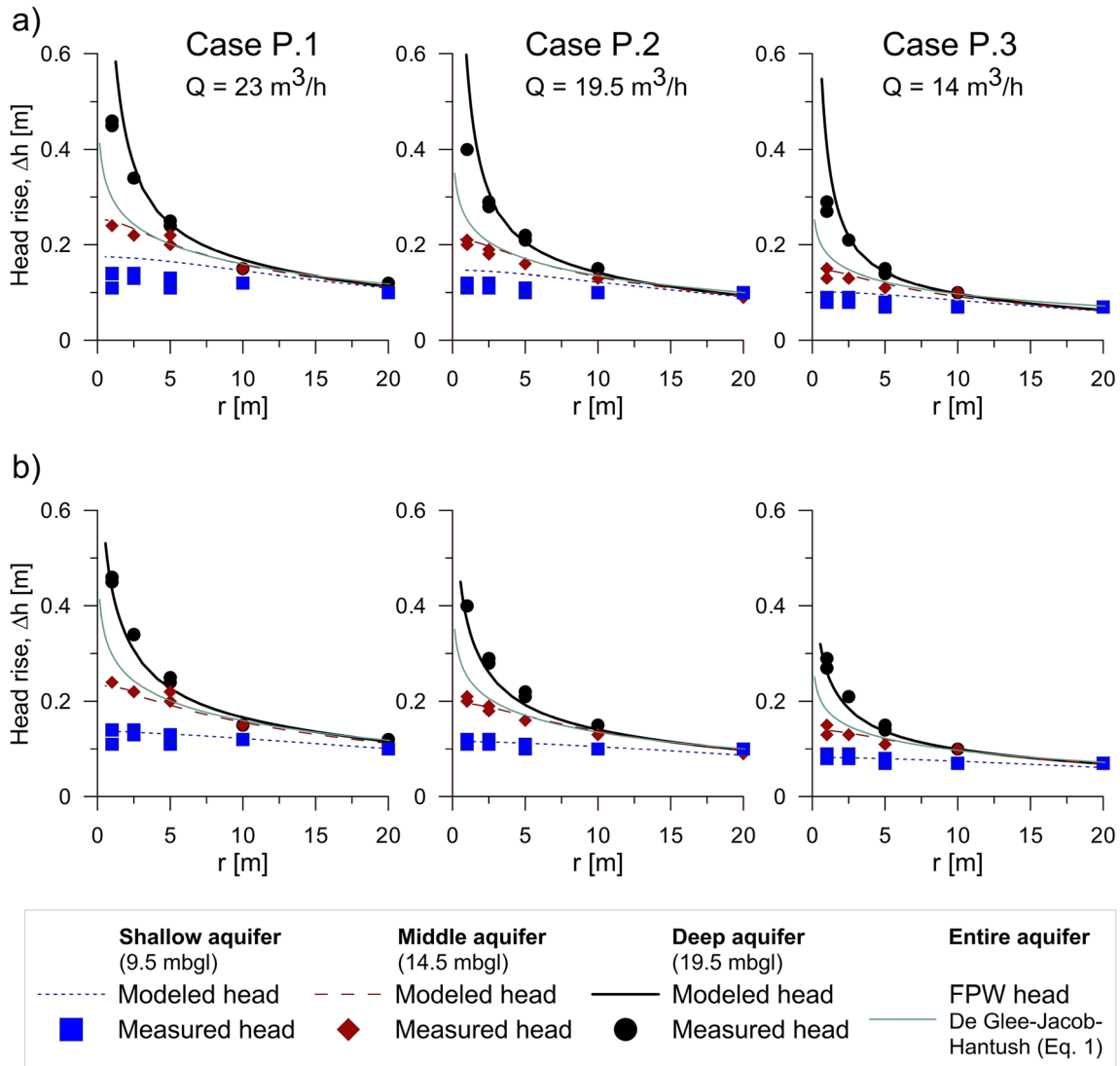
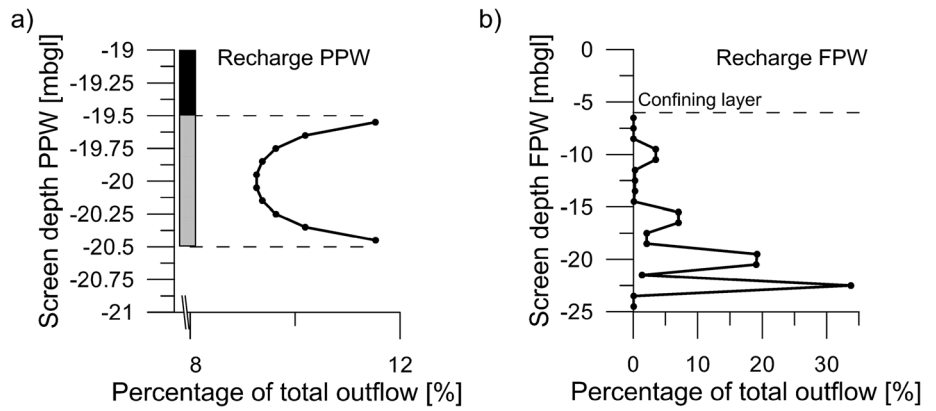
For the aquifer test on the PPW, constant recharge rates are used at approximately 23 (case P.1), 19.5 (case P.2) and 14 m<sup>3</sup>/h (case P.3; see Appendix). Due to well clogging during recharge of the pumpedoxic groundwater, a substantial well head increase occurred for case P.1, starting at 0.3 bar, up to 1.5 bar. This well head increase was smaller for case P.2 (starting at 1.2 bar, up to 1.5 bar) and case P.3 (starting at 0.35 bar, up to 0.5 bar).

For the three selected cases, a reasonable fit of the numerical result with the actual head rise in the observation wells is obtained when assuming the homogeneous, isotropic aquifer from the FPW aquifer test (Fig. 10a). However, in the near-well vicinity of the well screen the head rise is drastically



**Fig. 8** The measured head in the monitoring wells for the stationary recharge tests on the FPW with the simulation results in a heterogeneous aquifer and the fit with the De Glee-Hantush-Jacob method (Eq. 1) with  $K = 72.7$  m/day,  $c = 10$  days,  $H = 20$  m

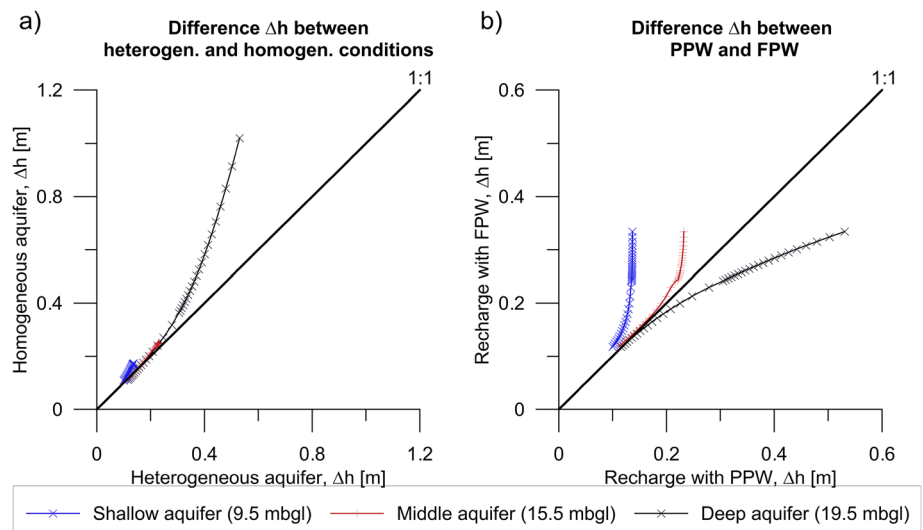
**Fig. 9** Vertical distribution of the numerical calculated screen flow rate (percentage) based on aquifer tests **a** Case P.1 with the PPW and **b** Case F.1 with the FPW



**Fig. 10** The measured hydraulic head rise,  $\Delta h$ , in the observation wells, as well as the modeled hydraulic head rise,  $\Delta h$ , at the indicated shallow, middle and deep monitoring depths (see Fig. 3) in an **a** homogeneous

aquifer, and **b** heterogeneous aquifer. Also the head rise while assuming recharge by a FPW with an equivalent  $Q$  is plotted for cases P.1–3 using the De Glee-Hantush-Jacob method

**Fig. 11** **a** The difference in head rise ( $\Delta h$ ) over the radial distance range of 0.5–20 m between artificial recharge with a PPW in the heterogeneous layered target aquifer (Case P.1) and an equivalent homogeneous aquifer considering different aquifer depths. **b** The difference in head rise ( $\Delta h$ ) over the radial distance range of 0.5–20 m between artificial recharge with a PPW (Case P.1) and a FPW in the target aquifer considering different aquifer depths



overestimated. Figure 11a shows that the maximum calculated head rise at a radial distance of 0.5 m from the well and at infiltration level (deep aquifer depth, 19.5 mbgl) is 1.1 m for the homogeneous aquifer conditions, while considering the actual aquifer heterogeneity the head rise is only 0.53 m. By accounting for the actual heterogeneity of the aquifer with the vertical variability in hydraulic conductivity as indicated in Fig. 3, an excellent fit with the observed field data is obtained.

In Fig. 11b, the strong deviation in head increase ( $\Delta h$ ) between the tested PPW and FPW at shallow aquifer level is shown. Comparing the monitored head rise at shallow aquifer depth (9.5 mbgl) with the head rise around a FPW at an equivalent recharge rate of 23 m<sup>3</sup>/h, the hydraulic head is considerably lower for case P.1 (Fig. 10b). The head rise is a factor 0.58 lower at 1-m radial distance from the well 9.5 mbgl, while this is still a factor 0.8 and 0.91 at 5- and 10-m radial distance.

Note that for a realistic dewatering project design, multiple artificial recharge wells are required to discharge the pumped water from the dewatered area at high volumetric recharge rates. By the principle of superposition, the reduced head rise with the use of multiple PPWs instead of multiple FPWs will be much higher.

The outflow at PPW is simulated with the constant heads given in Table 4, resulting in the desired bulk recharge rates of 23, 19.5 and 14 m<sup>3</sup>/h. The geometry of the PPW results in diverged outflow from both screen ends, resulting in higher outflow rates at the top and bottom of the well screen (Fig. 9a). The percentage of total outflow over a 0.1-m well screen section at the top and bottom is 11%, while this is 9.2% in the middle of the well screen. Simulating artificial recharge with the assumption of a uniform well-face flux boundary, as well as with uniform head ( $h_{PPW}$ ), shows negligible differences in predicted head rise outside a radius of 0.5 m from the well. Differences in head rise are only simulated within a radius of

0.5 m from the well screen, resulting in relative differences up to 2% in the calculated head rise for case P.1.

## Discussion

### The use of aquifer heterogeneity for efficient recharge PPWs

The present study shows that the use of recharge PPWs allows one to take advantage of the vertical variability in hydraulic conductivity in the aquifer. By accounting for the more and less permeable strata in the target aquifer, one could select a suitable, more permeable, layer at depth for artificial recharge to improve the efficiency of the recharge PPW and the efficiency of dewatering systems that involve reinjection of dewatered groundwater. At the field site, the selection of the sandy gravel layer with a hydraulic conductivity of 300 m/day at 20 mbgl provided recharge rates up to 23 m<sup>3</sup>/h at a PPW with a well screen of only 1 m length. Considering a homogeneous aquifer instead of the actual heterogeneous layering of the aquifer resulted in a drastic over-estimation of the required well head by a factor of 3.1. This means that the required well head  $\Delta h_{PPW}$  is 4.66 m instead of 1.51 m (see Table 4). Hence, the results of cases 11–14 show that wrongly assuming a heterogeneous layered aquifer as an equivalent homogeneous anisotropic aquifer could result in a major efficiency loss of the recharge PPWs, as well as the abstraction PPWs.

### Hydrogeological characterization and the recharge PPW depth

Usually, dewatering companies have to rely on a limited amount of hydrogeological information on the dewatering

site. Comprehensive field investigation would drastically increase the project costs for small to medium scale dewatering projects and hence, in reality, dewatering companies often have to rely on cone penetration test data (e.g. Robertson 1990), online available soil profiles and borehole logs to obtain information about soil heterogeneity. This could already provide useful information about potential high-permeability layers in the aquifer, as well as low-permeability zones. Prior to planning the dewatering design, analysis of such information is necessary to predict the optimal artificial recharge depths for PPWs.

In the Netherlands and Germany, dewatering companies use quick and cheap straight-flush rotary drilling with clean water as drilling fluid for well completion of naturally developed artificial recharge PPWs—so-called fast high-volume infiltration (FHVI) wells—in high-permeability strata in the aquifer (Van Lopik 2020). Skilled drilling operators are provided with a rough hydrogeological picture of the shallow subsurface and indicate high-permeability layers in an aquifer with their straight-flush drilling rig. During straight flush rotary drilling with clean water, large quantities of the flushed water pumped down the drill-pipe are infiltrating into the surrounding soil (Powers et al. 2007). Drilling through high-permeability layers in the aquifer causes significant loss of flushed water volume into the surrounding soil and reduces the velocity of the flush in the borehole annulus. A large sudden reduction in debris flush outflow at the surface is a good indication of a high-permeability layer in the aquifer and can be used by the drilling operator to place a naturally developed PPW at the desired depth in the aquifer at a minimum of well completion and development costs (Van Lopik 2020). A small aquifer test could provide useful information about the maximum volumetric recharge rate of the well and the hydraulic conductivity at recharge depth. Using straight-flush rotary drilling, multiple naturally developed PPWs can be installed within a very short period, while the completion and development of a conventional FPW with reversed-circulation rotary drilling is time consuming.

### Planning of dewatering design with multiple recharge PPWs

The scenarios consider a simplified well scheme of one abstraction PPW and one recharge PPW. However, in practice, multiple pumping wells are required to obtain the desired drawdown at the excavation site. This can be done by small filters and vacuum pumping in the top of the aquifer for smaller pumping rates, or by deep wells with larger well diameters and screen lengths for deep excavation sites and larger pumping rates (Powers et al. 2007). The number of required recharge wells is determined by the size of the dewatering project (volumetric abstraction rate), the aquifer characteristics, and the recharge well type. For example, a larger amount

of cheap, quickly installed naturally developed recharge PPWs with limited screen length is required due to lower well efficiencies compared with installation of proper completed recharge FPWs with filter packs to discharge the required dewatered groundwater volume.

Powers et al. (2007) show that for many dewatering problems the excavation with a set of pumping wells can be simplified to one large single pumping well. The same can be done for the set of recharge wells. This assumption has validity for scenarios of circular dewatering systems of closely spaced pumping wells, as well as for a similar circular set of recharge wells. For such systems, the required distance between the centre of the excavation site with the set of pumping wells and the set of recharge wells can be estimated based on the modeled cases (1–14) presented in this study. However, many dewatering systems are more complex and this assumption does not hold, such as for dewatering scenarios with widely spaced pumping and/or recharge wells, larger excavation sites, and the use of multiple arrays of recharge wells at different locations near the excavation site. In the design stage of the construction dewatering project, dewatering companies could provide reasonable predictions of the drawdown and the minimum required distance between the dewatering wells and the artificial recharge PPWs by accounting for slightly higher discharges and drawdowns, as well as a realistic upper and lower bound for the possible aquifer anisotropy based on the available hydrogeological information. Note that optimal drawdown/head rise predictions for PPWs is more complex than for FPW and requires 3D modeling while considering each individual partially penetrating recharge and pumping well in the dewatering scheme, since each well changes the performance of surrounding wells.

### Impact of background groundwater flow

Cases 1–14 only account for situations without background flow. In reality, the groundwater flow could have a significant impact on the dewatering design. Especially at large hydraulic gradients, the impact on the well design can be significant. For such scenarios, lower pumping rates at the downstream side of the excavation are required than at the upstream side to obtain the desired drawdown. Ideally, artificial recharge wells are installed downstream from the extraction site to reduce pumping costs (Powers et al. 2007; Cashman and Preene 2013). The negative influence of the hydraulic head rise due to recharge at the downstream side is smaller than for a scenario with no background flow, allowing for installation of artificial recharge wells closer to the dewatering site. Hence, the difference between the allowed above-ground minimum distance from the excavation site while using a PPW or FPW is smaller than for the analyzed scenarios with no background flow. However, a simple estimation of the actual drawdown at a given hydraulic gradient for the modeled scenarios in the



present study can be obtained by using the principle of superposition.

### Additional head loss in PPWs

The use of small well-screen lengths has the advantage of targeting the hydraulic impact at desired depth, as well as limit the drilling cost by placement of longer filters. However, PPWs that only screen a small portion of the aquifer have the disadvantage that this results in high additional well losses with respect to an equivalent FPWs. The present study shows that the required well head for artificial recharge with a PPW in a layer of relatively high permeability could be drastically reduced (see Table 5). Moreover, the use of PPWs that are screened at larger depths enable higher well heads, since infiltration pressures of recharge wells are restricted to the maximum-permissible pressure to avoid soil fracturing at a given soil depth (Olsthoorn 1982). Therefore, much higher pressures on the influx pipelines towards the PPWs can be retained for dewatering systems, than when using FPWs. This might prevent depressurization of air in both the pipelines and recharge PPW during above-ground groundwater transport and reduce potential risk of air clogging of the well.

The increase in pumping costs to enable optimal well heads and sufficient recharge rates for the PPWs need to be lower than the reduction of pumping costs by selection of the optimal well design that provides the smallest hydraulic impact at shallow aquifer depth at the excavation site with recharge PPWs. In practice, dewatering companies in the Netherlands and Germany are using recharge PPWs with limited screen length to optimize their dewatering design. Ideally, well heads up to 5 m are used to recharge groundwater in the recharge PPWs (screen lengths up to 2 m). Note, that smaller well screens of PPWs are more sensitive to clogging due to a smaller well-screen surface area compared to FPWs. This could cause potential increase in well head of the recharge well and reduced recharge rates over time (e.g. Olsthoorn 1982; Rinck-Pfeiffer et al. 2000; Houben et al. 2018), which was observed for the recharge PPW tested in the present study (case P.1–3).

In order to reduce the additional well head associated with partial penetration and well clogging, PPWs with longer screens can be used. For example, if the optimal screen depth of the PPW in a high-permeability layer is determined, one could choose a PPW with a longer screen with its top screened in the high-permeability layer. The results show that the controlling effect on the drawdown at excavation level of a PPW screened at 20 or 25 mbgl is more or less similar (see Fig. 5). Therefore, the use of a PPW with a filter length of 5 m between these two depths will result in a similar controlling effect on the drawdown at excavation level, while the risks of well-clogging are reduced.

### Using PPWs for artificial recharge to prevent soil settlement

The present study shows that using PPWs ( $\Delta z = 10$  m) in the upper part of the target aquifer obtains lower drawdowns than a FPW (Fig. 5). In many cases, the groundwater drawdown due to pumping in aquifers could cause soil settlement (Phien-vej et al. 1998; Pujades et al. 2014; Zhang et al. 2017). For such dewatering systems, obtaining sufficient drawdown near the excavation site is not the major concern for optimization of the dewatering scheme. In order to reduce soil settlement near the excavation site, curtain sealings and aquifer recharge are suggested to reduce the drawdown in the vicinity of the excavation site (Zhang et al. 2017). The recharge wells are screened over a large portion of the target aquifer to obtain the required drawdown and control of the settlement effects. Combining flow-barrier curtain sealings with a well set-up of artificial recharge PPWs targeting shallower aquifer depths, instead of using FPWs, could avoid the risks of settlement while improving the dewatering design drastically.

### Conclusions

Based on the results of experimental recharge tests on a PPW and a FPW, as well as numerical results of the scenario modeling on a simplified pumping-recharge dewatering scheme, the following is concluded:

- 1 The design of construction dewatering systems combined with artificial recharge for discharge of dewatered groundwater can be drastically improved by targeting recharge in relatively high-permeability layers in the aquifer with PPWs. To date, most dewatering designs consider recharge wells that screen large parts of the aquifer thickness without taking into account the vertical variability in hydraulic conductivity of the aquifer. The modeled scenarios show that PPWs with a 1-m filter screen at depth (15 m below the confining unit in a 20-m-thick aquifer) can be installed much closer to the dewatering site than conventional recharge FPW. For example, to obtain a required drawdown of 2.5 m within a radius of 10 m from the abstraction site and a shallow groundwater level for combined abstraction and recharge at a rate of  $Q = 250$  m<sup>3</sup>/h, PPWs can be installed at 20 m distance, while FPWs need to be installed at a distance of 41 m. Consequently, the above-ground distance between the target excavation site and the recharge site can be significantly reduced, while the repumping of recharged water at the excavation site, as well as the overall costs of the dewatering, can be minimized.
- 2 Screening PPWs at relatively high-permeability layers in the subsurface allows one to infiltrate large quantities of

water at relatively small well head. The use of equivalent homogeneous anisotropic aquifer conditions predicts erroneously high well head and near-well hydraulic gradients for the tested recharge PPWs. In the field test of the present study, recharge rates up to  $25 \text{ m}^3/\text{h}$  were obtained in a heterogeneous semi-confined aquifer. A gravel layer with a high hydraulic conductivity of  $300 \text{ m/day}$  was used as the target layer for recharge in the aquifer.

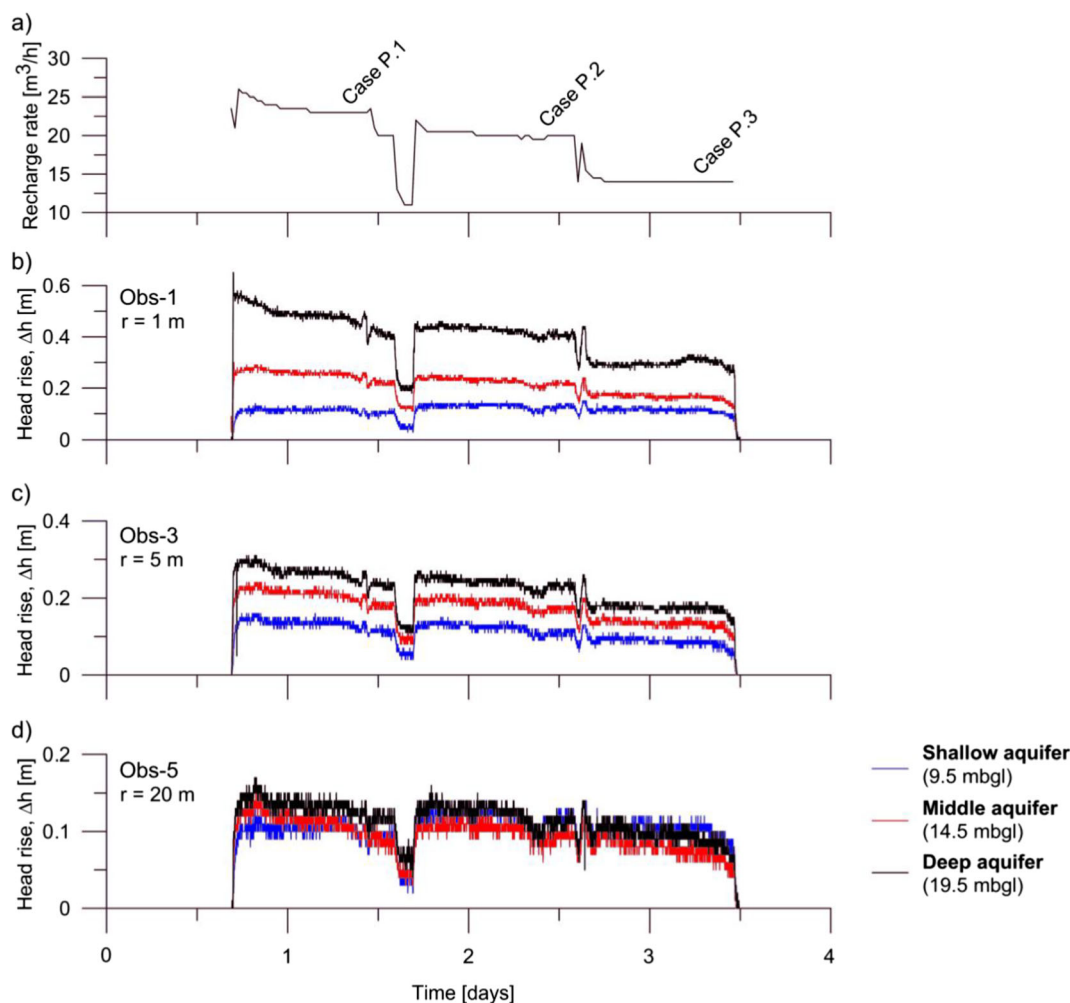
- 3 The experimental data and the scenario modeling show the importance of proper hydrogeological characterization of the soil. Relatively high-permeability layers in the subsurface can be used to infiltrate large quantities of water with PPWs. Moreover, predicting the hydraulic head in the

aquifer required 3D modeling, while considering aquifer heterogeneity.

**Acknowledgements** The authors wish to thank Sjaak van Laanen and Thijs de Vet from the dewatering company P.J. de Vet and Zonen for constructing the experimental fieldwork set-up. We are grateful to Jürgen Knops from the engineering company Gell and Partner GhB for providing geological data and the geotechnical report of the fieldwork site. Moreover, the authors wish to thank Thomas Sweijen for his critical comments on prior drafts of this report and two anonymous reviewers for their constructive feedback.

**Funding information** This work was supported by foundations STW (Foundation for Technical Sciences) and O2DIT (Foundation for Research and Development of Sustainable Infiltration Techniques).

## Appendix 1: volumetric recharge rate and head rise during PPW recharge test



**Fig. 12** a The recharge rate over time and the head rise ( $\Delta h$ ) in observation wells b Obs-1, c Obs-3, d Obs-5. The used values for stationary groundwater flow modelling for cases P.1–3 are labelled

## References

- Barker JA, Herbert R (1992) A simple theory for estimating well losses: with application to test wells in Bangladesh. *Appl Hydrogeol* 0:20–31
- Bear J (2007) *Hydraulics of groundwater*. Dover, New York
- Cashman PM, Preene M (2013) *Groundwater lowering in construction: a practical guide to dewatering*, 2nd edn. Taylor and Francis, Boca Raton, FL
- De Glee GJ (1930) *Over grondwaterstromingen bij wateronttrekking door middel van putten* [About groundwater flows in case of water abstraction by wells]. PhD Thesis, Technische Hooogeschool, Delft, The Netherlands
- Fang Q, Tai Q, Zhang D, Wong LNY (2016) Ground surface settlements due to construction of closely-spaced twin tunnels with different geometric arrangements. *Tun Undergr Space Technol* 51:44–151
- Forth R (2004) *Groundwater and geotechnical aspects of deep excavations in Hong Kong*. *Eng Geol* 72:253–260
- Galeati G, Gambolati G (1988) Optimal dewatering schemes in the foundation design of an electronuclear plant. *Water Resour Res* 24(4): 541–552
- Hantush MS, Jacob CE (1955) Non-steady radial flow in an infinite leaky aquifer. *Transact Am Geophys Union* 36(4):95–100
- Harbaugh AW, Banta ER, Hill MC, McDonald MG (2000) MODFLOW-2000, the US geological survey modular ground-water model. User guide to modularization concepts and the ground-water flow process. US Geol Surv Open-File Rep 00-92
- Houben GJ (2015) Review: Hydraulics of water wells—flow laws and influence of geometry. *Hydrogeol J* 23(8):1633–1657
- Houben GJ, Hauschild S (2011) Numerical modeling of the near-field hydraulics of water wells. *Ground Water* 49(4):570–575
- Houben GJ, Wachenhausen J, Guevara Morel CR (2018) Effects of ageing on the hydraulics of water wells and the influence of non-Darcy flow. *Hydrogeol J* 26(4):1285–1294
- Jin Y, Holzbecher E, Sauter M (2016) Dual-screened vertical circulation well for groundwater lowering in unconfined aquifers. *Ground Water* 54(1):15–22
- Jurado A, De Gaspari F, Vilarrasa V, Bolster D, Sánchez-Vila X, Fernandez-Garcia D, Tartakovsky D (2012) Probabilistic analysis of groundwater-related risks at subsurface excavation sites. *Eng Geol* 125:35–44
- Kasenow M (2010) *Applied ground-water hydrology and well hydraulics*, 3rd edn. Water Resources Publ., Highlands Ranch, CO
- Olsthoom TN (1982) *Verstopping van persputten* [The clogging of recharge wells]. Keuringsdienst voor Waterleidingartikelen KIWA N.V., Rijswijk, The Netherlands
- Phien-wej N, Giao PH, Nutalaya P (1998) Field experiment of artificial recharge through a well with reference to land subsidence control. *Eng Geol* 50:187–201
- Powers P, Corwin A, Schmall P, Kaeck W (2007) *Construction dewatering and groundwater control*, 3rd edn. Wiley, Hoboken, NJ
- Preene M, Brassington R (2003) Potential groundwater impacts from civil-engineering works. *Water Environ J* 17(1):59–64
- Pujades E, Vázquez-Suñé E, Carrera J, Jurado A (2014) Dewatering of a deep excavation undertaken in a layered soil. *Eng Geol* 178:15–27
- Pujades E, De Simone S, Carrera J, Vázquez-Suñé E, Jurado A (2017) Settlements around pumping wells: analysis of influential factors and a simple calculation procedure. *J Hydrol* 548:25–236
- Rinck-Pfeiffer S, Ragusa S, Sztajnbock P, Vandeveld T (2000) Interrelationships between biological, chemical, and physical processes as an analog to clogging in aquifer storage and recovery (ASR) wells. *Water Res* 34(7):2110–2118
- Robertson PK (1990) Soil classification using the cone penetration test. *Can Geotech J* 27:151–158
- Roy D, Robinson KE (2009) Surface settlements at a soft soil site due to bedrock dewatering. *Eng Geol* 107(3):109–117
- Ruud NC, Kabala ZJ (1997) Response of a partially penetrating well in a heterogeneous aquifer: integrated well-face flux vs. uniform well face flux boundary conditions. *J Hydrol* 194:76–94
- Tan Y, Wang D (2013) Characteristics of a large-scale deep foundation pit excavated by the central-island technique in Shanghai soft clay. II: top-down construction of the peripheral rectangular pit. *J Geotechn Geoenviron Eng* 139(11):1894–1910
- Thiem A (1870) *Die Ergiebigkeit artesischer Bohrlöcher, Schachtbrunnen und Filtergalerien* [The yield of artesian boreholes, shaft wells and filter galleries]. *J Gasbeleucht Wasserversorg* 14: 450–467
- Tügel F, Houben GJ, Graf T (2016) How appropriate is the Thiem equation for describing groundwater flow to actual wells? *Hydrogeol J* 24(8):2093–2101
- Van Lopik JH (2020) *Design of recharge and abstraction well systems in heterogeneous aquifers: modeling and experimental studies*. PhD Thesis, Utrecht University, Utrecht, The Netherlands
- Wang J, Feng B, Guo T, Wu L, Lou R, Zhou Z (2013) Using partial penetrating wells and curtains to lower the water level of confined aquifer of gravel. *Eng Geol* 161:16–25
- Wang SK, Wee YP, Ofori G (2002) DSSDSS: a decision support system for dewatering systems selection. *Build Environ* 37(6):625–645
- Wu Y-Y, Shen JS, Cheng W-C, Hino T (2017) Semi-analytical solution to pumping test data with barrier, wellbore storage, and partial penetration effects. *Eng Geol* 226:44–51
- Zeng C-F, Zheng G, Xue X-L, Mei G-X (2019) Combined recharge: a method to prevent ground settlement induced by redevelopment of recharge wells. *J Hydrol* 568:1–11
- Zhang Y-G, Wang J-H, Cheng J-J, Li M-G (2017) Numerical study on the responses of groundwater and strata to pumping and recharge in a deep confined aquifer. *J Hydrol* 548:342–352
- Zheng G, Cao JR, Cheng XS, Ha D, Wang FJ (2018) Experimental study on the artificial recharge of semiconfined aquifers involved in deep excavation engineering. *J Hydrol* 557:868–877

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