Electronic supplementary material - Hydrogeology Journal

Review: Horizontal, directionally drilled and radial collector wells

Georg J. Houben[1], Sarah Collins[2], Mark Bakker[3], Thomas Daffner[4], Falk Triller[5], Anvar Kacimov[6]

1 Federal Institute for Geosciences and Natural Resources (BGR), Stilleweg 2, 30655 Hannover, Germany

Tel.: +49-511-6432373 Email: georg.houben@bgr.de

2 British Geological Survey, Edinburgh, UK

3 Delft University of Technology, Delft, Netherlands

4 Umweltbüro GmbH Vogtland, Weischlitz, Germany

5 Bohrlochmessung Storkow GmbH, Storkow, Germany

6 Sultan Qaboos University, Oman

Supplement to section '*History and types of non-vertical wells*' of the main article

Khanats and kharezes

The oldest man-made horizontal subsurface installations to extract and convey groundwater are the qanats (khanats) or karizes (kharezes), which have been - and in some countries remain being – used for water supply, especially in the Near and Middle East, in Northern Africa, but also in parts of Asia and Europe. They were probably first used about 3000 years ago in Persia and Arabia (Beaumont 1973; Mays 2008; Sutton 1978; Wilkinson, 1977; Ebrahimi et al. 2021). The hydraulically active tip of the qanat intercepts groundwater at the foothills of a mountain, where the water table is higher, dipping towards the topographically lower valley zone. In some cases, several fingers protrude into the water-saturated zone from the starting point of the conveyor tunnel, making these qanats remarkably similar to modern RCWs. The tunnels can originate at the so-called "mother well", i.e. a vertical well, which can be up to 300 m deep (Ebrahimi et al. 2021). The tunnel is orthogonal to both this well and to the piezometric surfaces of a shallow unconfined aquifer. The rest of the often kilometer-long tunnel conveys the water to a lower-lying settlement in the foreland. An array of vertical shaft wells is made above the course of the qanat tunnel to dispose of the removed material. Their spacing usually varies between 30 and 150 m (Ebrahimi et al. 2021). Tunnel lengths may reach up to 100 km with 2115 shafts (Ebrahimi et al. 2021). Later, these shafts serve for air ventilation and as access for maintenance. During dry seasons, usually once in several years, when surface flow in the channel is small, laborers have to manually dredge and bail to the ground surface the sediments, which gradually accumulate at the tunnel bottom. The depth of the access shafts increases from the upper reaches of the qanat downstream, corresponding to the increase of the groundwater depth. Thus, mathematically, a qanat tunnel is an - essentially transient - line sink, the strength of which varies along its course. A "normal", "subcritical" or "supercritical" 1-D phreatic flow over a tilted bedrock, which underlies the qanat-tapped aquifer, is superposed with flow induced by this line sink such that the seepage pattern becomes essentially 3-D (Kacimov 2005, 2006; see Polubarinova-Kochina, 1955,1977 for the classification of the flows types). Combinations of vertical and horizontal wells were also used. Voudouris et al. (2019) describe a 2,000 year old vertical well shaft from Lorestan, Iran, which was equipped with a horizontal gallery dug into the direction of a river to induce bank filtration.

Covered drainage trenches

The covered drainage trenches probably developed from spring captures, where the installation of horizontal collector pipes was already employed by the Romans to improve spring yields (Tölle-Kastenbein 2012; Öziş 2015). Often several strings were connected to a central collector shaft in a fan shaped pattern, from where the water was pumped, e.g. to a higher reservoir or water tower. The trenches rarely exceeded depths of 6 to 7 m, as the dewatering and the geotechnical stabilization of the trench walls became increasingly difficult with deeper depths. Maximum recorded trench depths ranged between 9 and 13 m (Houben 2019). The total length of the strings could be substantial, often several hundreds of meters, in some cases up to several kilometers (Houben 2019). In some cases, the pipes were replaced by little tunnels, often elliptical, which allowed later inspection and cleaning. The drainage trenches were quite expensive, due to the high construction costs for dewatering and trench stabilization.

Figure S1 shows historical pictures of drainage ditches. Although most of the features shown on the left are still employed today, trenching machines are often used to install the drains (Rushton and Brassington 2013).

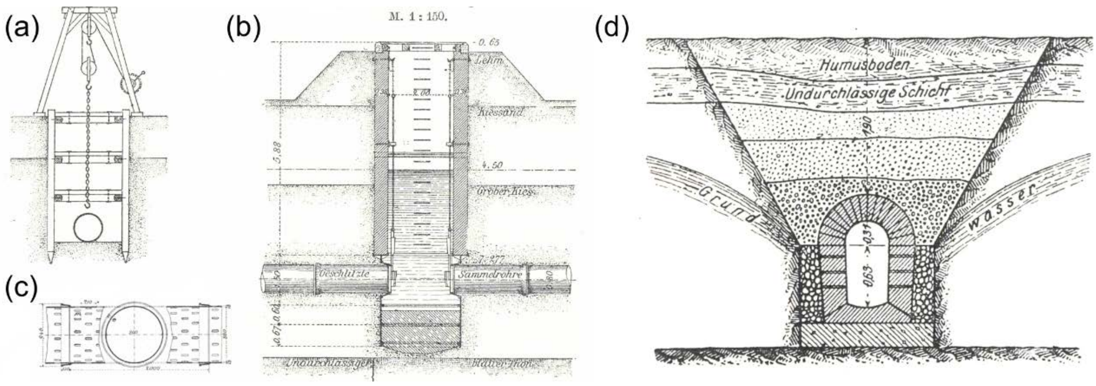

Fig. S1 Left: drainage trench built for the water supply of Hannover, Germany, 1878, with (a) trench support (timbering), (b) collector shaft, (c) perforated pipe (modified after Houben 2019). Right: (d) drainage ditch with concrete seepage tunnel, several layers of gravel pack and an impermeable cover layer (Heilmann 1927). Humusboden = top soil, undurchlässige Schicht = impermeable layer, Grundwasser = groundwater.

Horizontal drip tunnels in consolidated rocks, an ancient technique known from castles and fortifications, were put to use for public water supply in the 19th century. Again, the length could reach several kilometers. A country were this technique was intensively employed is Belgium. They were mostly installed in consolidated Paleozoic rocks, often limestone and Mesozoic chalk aquifers (Dassargues et al. 1988) but sometimes also in unconsolidated sands (Biron et al. 2014). Many of these systems are still active, e.g. the one supplying water to the major town of Liege, using 45 km of galleries in a chalk aquifer (Dassargues et al. 1988; Hallet et al. 2000; Brouyere et al. 2004). Several systems of this type were also build in Germany; e.g. for the water supply Wiesbaden which in part still relies on 11.5 km of drip tunnels in quartzite built between 1875 and 1910 (Vogel 1951; Houben 2019). So-called “Maui tunnels”, skimming tunnels tapping dike-impounded groundwater are common on the Hawaiian Islands (Stearns and MacDonald 1942). They consist of (sometimes inclined) shafts and horizontal tunnels of up to 1000 m length, skimming the water table. Due to their excellent yield some are still being used.

Slant wells

A special slant water supply well is the Mannesmann well shown in Figure S2a, introduced by the German company of that name in the 1960s (Huisman 1972). It consisted of a classical caisson, from which the laterals were drilled upwards at an angle of 10 degrees or more. Drilling started from the caisson with a small-diameter pilot drillhole, followed by the temporary casing. The screens are then installed from the surface downwards, with or without a gravel pack. Finally, the casing is pulled. The laterals terminate in little revision shafts at the surface. This allowed better accessibility for later inspection and cleaning. However, they were more expensive than classical RCW since the laterals had to be significantly longer and had to pass through the unsaturated zone, often at a shallow angle. Such wells were drilled in Germany for water supply in very low numbers in the 1960s but did not catch on. They could also be used for artificial groundwater recharge through the laterals. The authors found documentation on one example from Northern Germany. It was drilled in 1965 with a caisson depth of 30 m. It had twelve laterals, all of them inclined at an angle of around 14°. The laterals were up to 125 m long, of which almost half was unscreened und thus unproductive casing. It was converted into a regular RCW in 1991. The reverse of this concept is the Loeck well (Fig. S2b), developed by the German driller Hermann Loeck (Huisman 1972). Here, the inclined laterals are drilled downwards at an angle from a shallow shaft. The absence of a caisson is the main advantage, as it is usually the most expensive part of such a well.

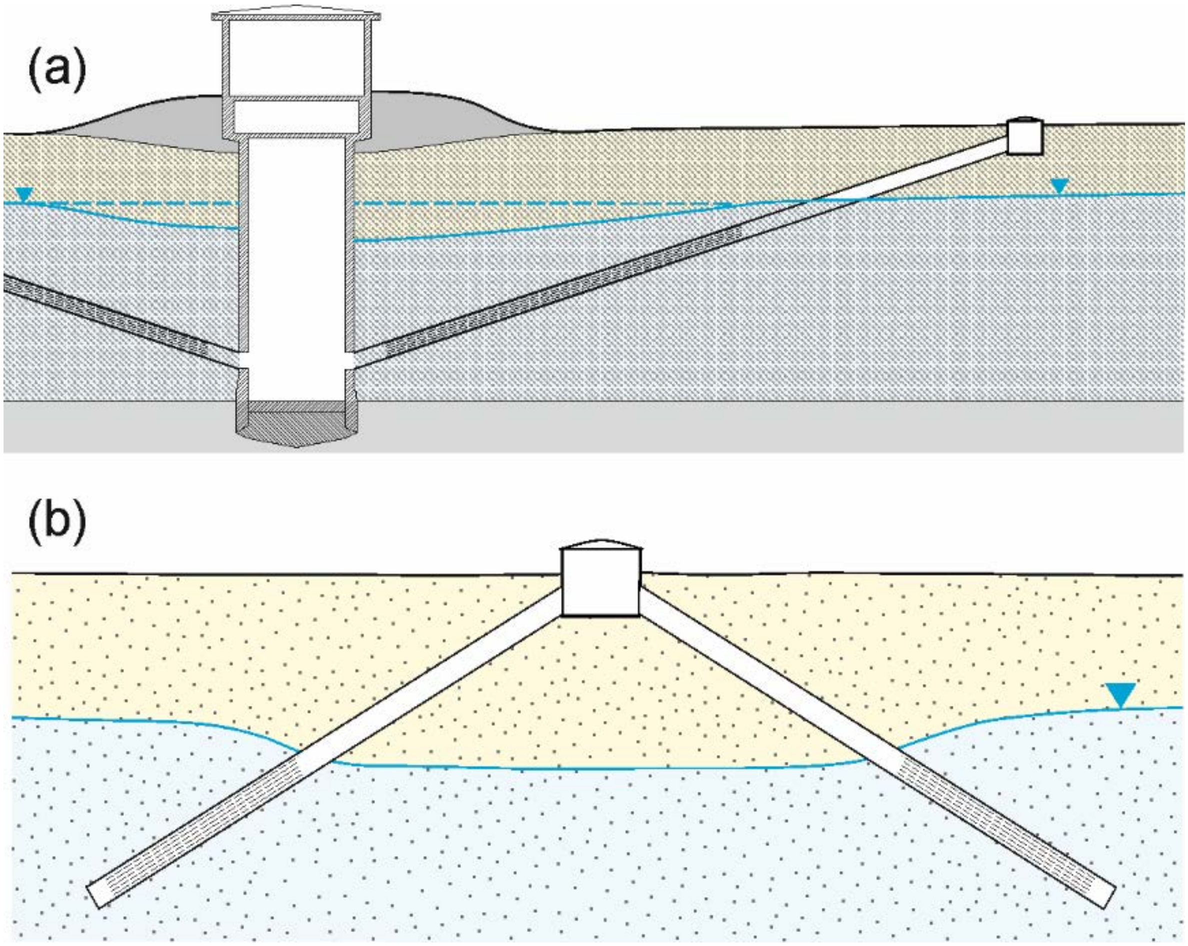

Fig. S2 Special types of slant wells: (a) Mannesmann well, (b) Loeck well.

Another special type of horizontal well is the disk well (Tellerbrunnen in German). Here, jetting spears were driven into the subsurface from a caisson. The water jetting was intended to induce a separation of the grain sizes of the aquifer material, so that a gradation of grain sizes would occur, with the coarsest fractions coming to rest close to the caisson, similar to a gravel pack (Bieske 1959). Jetting was done all around the caisson, so that – in theory – a disk of permeable material would be formed all around the caisson (considering the presence of the caisson, this zone should actually have the shape of a torus). The method, of course, only worked in aquifers with non-uniform grain size distribution. Water entered the caisson through some short laterals or perforations in the caisson itself. The disk well was supposed to close the gap between RCW and vertical wells, with lower cost than the former and higher yield than the latter. Despite the savings by avoiding long laterals, the shaft remained a costly investment. The concept was tried only briefly in Germany, including one well in Neuss, and then abandoned.

A recent development is the horizontal auger drilling (HAD). It consists of two steps: at first a thin pilot rod is pushed into the aquifer, followed by a horizontal auger of larger diameter, running in a protective casing. After pulling the auger, the installation of gravel packs and clay plugs is possible, followed by pulling of the auger casing. Since the machinery is somewhat smaller, it can be used in shafts of smaller diameters.The Abt method, a combination of vertical drilling and horizontal screens, was developed in Germany. A string of overlapping vertical boreholes are drilled and the bottom part backfilled with filter gravel adapted to the aquifer granulometry. This creates a continuous linear bed of gravel. The upper parts can be backfilled with impermeable material, as needed. Drilling diameters are usually 1,200 mm, with an 800 mm overlap, maximum depth is 20 m. The horizontal screen is then pushed into the gravel bed from a classical RCW shaft. The extra cost for the vertical drill holes is compensated by (a) the much higher yield of the lateral, due to the thick and highly permeable gravel pack, allowing much shorter laterals, (b) smaller diameters of the protective casing used for the installation of the lateral (e.g. 300 mmm for 200 mm lateral), which require less force and thus smaller hydraulic jacks, (c) the cumbersome installation of a gravel pack into the small space between lateral and protective casing can be dispensed with. Another advantage is that material obtained during drilling can be used to dimension gravel pack and screen slots.

Special types of caisson construction

In some cases, the caisson can be replaced by existing underground infrastructure. In East Germany, existing shaft wells (diameter 1.5 m) were converted into RCW by sealing the original shaft and installing laterals (Krebs et al. 1957). Initially, small screen diameters of 50-80 mm and short lateral lengths of < 6 m were tried but proved to be insufficient. Later laterals had diameters of 130 mm and longer lengths, which dramatically increased the yield. A similar type of RCW was developed for Sri

Lanka, where existing large-diameter (> 2.5 m) dug wells were converted into RCW by adding laterals (90 mm diameter) with a length of up to 30 m (Ball and Herbert 1992). These wells produced a continuous yield of 60 m^3/d, more than double the original wells. Laterals can be replaced by tunnels, resembling the drip galleries described above. One well of this type was built in the Ruhr area, Germany, during WW2 (ten Thoren et al. 1997). There, the yield of an 80 m deep shaft well (shaft diameter 1500 mm), tapping a fractured aquifer, was augmented by adding two cross-drifts (1.8 by 0.75 m) of 40 and 80 m length, respectively, secured with brickwork lining. Water entered over the entire length of the cross-drifts through two rows of perforated bricks, installed at 0.3 and 0.7 m height. In the late 1990s, six additional laterals (120 to 300 mm) of up to 51 m of length were added by horizontal drilling from the well bottom. Their yield was improved by hydraulic fracturing. The well now has a yield of 20 m^3/h and the shaft provides a storage volume of 220 m^3.

A modern but rare type of tunnel laterals is known as pozo con mina (well with mine/tunnel) in Spain (Queralt et al. 2012). The caisson is sunk through the alluvial aquifer into the lower aquitard. There, a horizontal tunnel is excavated from which several drillholes are directed upwards into the aquifer. Each of them can be opened or closed by a valve (from above). The caisson also contains perforations to improve the yield. A combination of laterals driven from collector tunnels is also possible, thus diminishing the number of caissons (Hubbs et al. 2011). A rather unusual type of caisson construction was described by Bouezmarni et al. (2014). They drilled overlapping boreholes along the perimeter of a circle and backfilled each with concrete. After removing the remaining sand from the inner circle, regular laterals were pushed into the aquifer.

Supplement to section '*Construction techniques*' of the main article

Early horizontal drilling techniques in mining

Horizontal drilling techniques have been documented from the mining sector as early as the 18th century, e.g. in the book by Leupold (1724). The images there show percussion drilling done by hand with small diameters (2" to 4", 5 to 10 cm). It was probably used to explore for ore, provide ventilation and for dewatering. Horizontally drilled wells were introduced in the dewatering of lignite strip-mines in Germany at the beginning of the 20th century (Sonntag 1914). In some mines, lignite seams and thin sand layers between them proved difficult to dewater by vertical wells. Therefore, a horizontal drill string, driven by an electrical motor, was installed underneath a carriage (Fig. S3). Drillholes were done at the sole of the seam, with a horizontal spacing of 5 to 10 m between the borings. Drilling fluids could be pumped through string and drill head to facilitate the progress. The rig ran on train tracks and was moved back and forth by a rope on a winch. Drilling length was usually 20 to 60 m, in rare cases up to 100 m. Drilling diameter was 330 or 400 mm. The boreholes were slightly inclined upwards to facilitate the outflow of water. If no obstacles, such as boulders, were encountered, progress would be between 3 and 6 m per hour. As no gravel pack could be installed, the screen was covered by a fine metal mesh. Horizontal dewatering drillholes are still being used in lignite mines in Eastern Germany, today often combined with vacuum pumping (Jolas 2003).

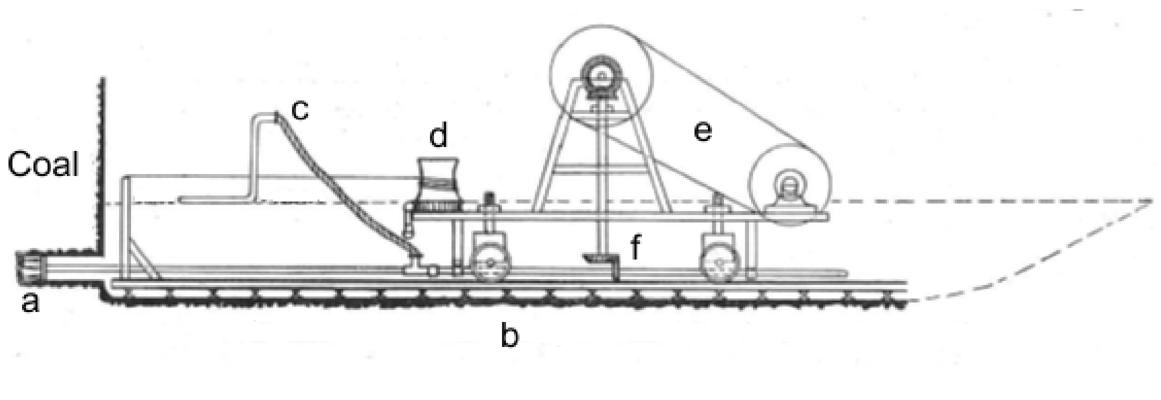

Fig. S3 Horizontal drilling technique for dewatering in lignite mining (Sonntag 1914). *a* = drill bit, attached to string, *b* = train track, *c* = drilling fluid hose, *d* = winch, *e* = motor and drive system, *f* = gear box.

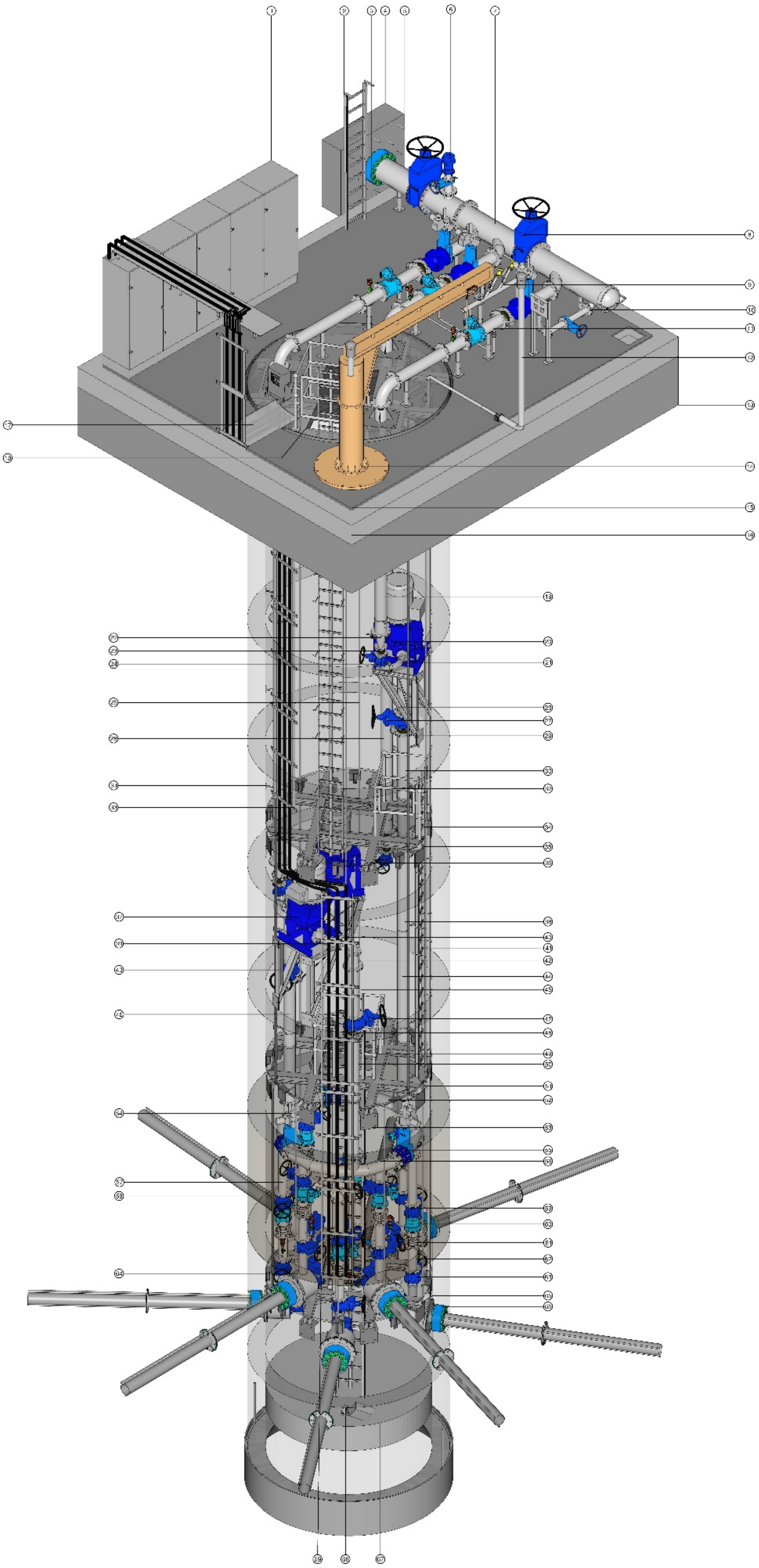

Figure S4: Detailed technical drawing of a radial collector well (Umweltbüro Vogtland, Daffner). (*1*) electric cabinet, *(2*) ladders, *(3)* sealing for pipes, *(4)* concrete counter bearings, *(5)* concrete fixing anchors, *(6)* air release valve, *(7)* collector pipe, *(8)* gate valve, flanged, *(9)* stainless steel sensor housing (pH, conductivity, temperature, oxygen), *(10)* connection fittings for firefighting, *(11)* magnetic inductive water meter and pressure probe, *(12)* air fan with extractor line and hood, *(13)* concrete foundation, *(14)* column crane, *(15)* floor cover, *(16)* concrete floor, *(17)* electrical cable cover, *(18)* hatch, *(19)* riser pipe, *(20)* centrifugal pumps, *(21)* pressure connection, *(23)* reducer, *(24)* rubber expansion joint, *(25)* gate valve, flanged, *(26)* supply line for acid (rehabilitation), *(27)* racket for pump, *(29*) gate valve , flanged, *(30)* riser pipe, *(31)* electropotential connection, *(32)* railing, *(33)* electrical cable ducts, *(34)* piezometer pipe, *(35)* platform support, *(36)* steel beam, *(37)* centrifugal pumps, *(38)* riser pipe, *(39)* pump inlet, *(40)* electrical supply cable for pump, *(41)* ladder, *(42)* reducer, *(43)* pump bracket, *(44, 45, 46)* riser pipes, *(47)* gate valve, flanged, *(48)* rubber expansion joint, *(49)* platform, *(50)* riser pipe, *(51)* platform, *(52, 53)* platform support, *(54)* Knife gate valve, *(55)* fitting, extension piece, *(56)* ring pipe (collector), *(57)* supply pipe from lateral to ring pipe, *(58)* caisson with pre-fabricated segments and laterals, *(59)* gate valve for lateral, *(60)* magnetic inductive water meter for lateral, *(61)* fitting and extension piece, *(62)* supply pipe from lateral to ring pipe, *(63)* gate valve for lateral, *(64)* gate valve housing, *(65)* piezometer pipe, *(66)* seal for caisson/lateral interface, *(67)* caisson floor, reinforced concrete, *(68)* caisson sump and pump, *(69)* gate valve with inspection window (plexiglas pane).

Statistical data on RCWs

The total number of RCWs worldwide is difficult to assess. Germany has at least 200 RCWs (Daffner et al. 2019a,b), many along the river Rhine, supplying the large towns and chemical factories and steel mills located along it. The city of Düsseldorf, which installed the first RCW in Germany in 1951, operates 18 RCWs to obtain bank filtrate for water supply. Not all German RCWs are located near large rivers. The public water supply company of Hannover operates 16 RCWs, extracting groundwater from a shallow unconfined aquifer. Austria has 204 registered RCWs (Klambauer 2017), of which 65% are used for public and the remainder for industrial water supply. Switzerland operates around 200 (Conrad 2010), Serbia 99 for Belgrade alone (Dimkić et al. 2011), Hungary 217 for Budapest alone (Dillon 2002; Nagy-Kovács et al. 2018), South Korea more than 100 (Hong et al. 2016; Lee et al. 2012; Hang-Tak et al. 2020) and the US around 250 (Hunt 2003). These numbers would add up to roughly 1,100 RCWs. Others have been documented from Canada (Ophori and Farvolden 1985; Ameli and Craig 2017), Great Britain (Jones and Singleton 2000; Rushton and Brassington 2013), France (Vibert 1953; Bassompierre and Soyer 1959; Spiridonoff 1964; Archembault et al. 2003), Belgium (Fehlmann and Fehlmann 1959; Bouezmarni et al. 2014), Italy (Citrini 1951, 1953; Spiridonoff 1964), Spain (Spiridonoff 1964), the Netherlands (Pluijmackers et al. 2005), Denmark (Hinsby, pers. comm.), Czech Republic (Knezek and Kubala 1994), Poland, Russia (Razumov 1974), Lithuania (Tarshish 1992), Botswana (Davies et al. 1995), Zimbabwe (Kitching 1991), Sri Lanka (Ball and Herbert 1992), India (Prakash and Raman 1987, Gurudnaha Rao and Gupta 1999; Kumar and Mehrotra 2009; Banerjee 2012), Malaysia (Mohamed and Rushton 2006; Ismail et al. 2013) and China (Appiah-Adjei et al. 2012, 2013). Although the total number of RCWs is negligible compared to that of vertical wells, they can be very important on a local scale, especially where a high water demand has to be met locally, as one RCW can easily replace several vertical wells.

Daffner et al. (2019a,b) collected data from 350 RCWs, mostly from Germany (204) and from other European countries (46 wells). For 266 of them, technical information was available that allowed a statistical evaluation: 92% had a “wet shaft”, only 8% a dry shaft. Shaft diameters varied between 2 and 5 m. They accounted for a total of 1,770 laterals (average 6.7 per well) with a total length of 63.4 km (average lateral length 35.8 m). Two thirds of all RCWs had caisson depths of no more than 20 m; depths exceeding 30 m are rare (Fig. S5a). Slightly more than two thirds of all laterals fall in the length range between 20 and 50 m, only 20% are longer (Fig. S5b). It should be noted that some of the shorter lengths may be unintentional, as laterals sometimes encounter an obstacle during construction and then have to be abandoned at the position where the problem occurred. The screen diameters are significantly smaller than those commonly used for vertical wells (Fig. S5c). Only the very few bigger ones (> 300 mm) come close. Around 60% have diameters of 200 and 250 mm.

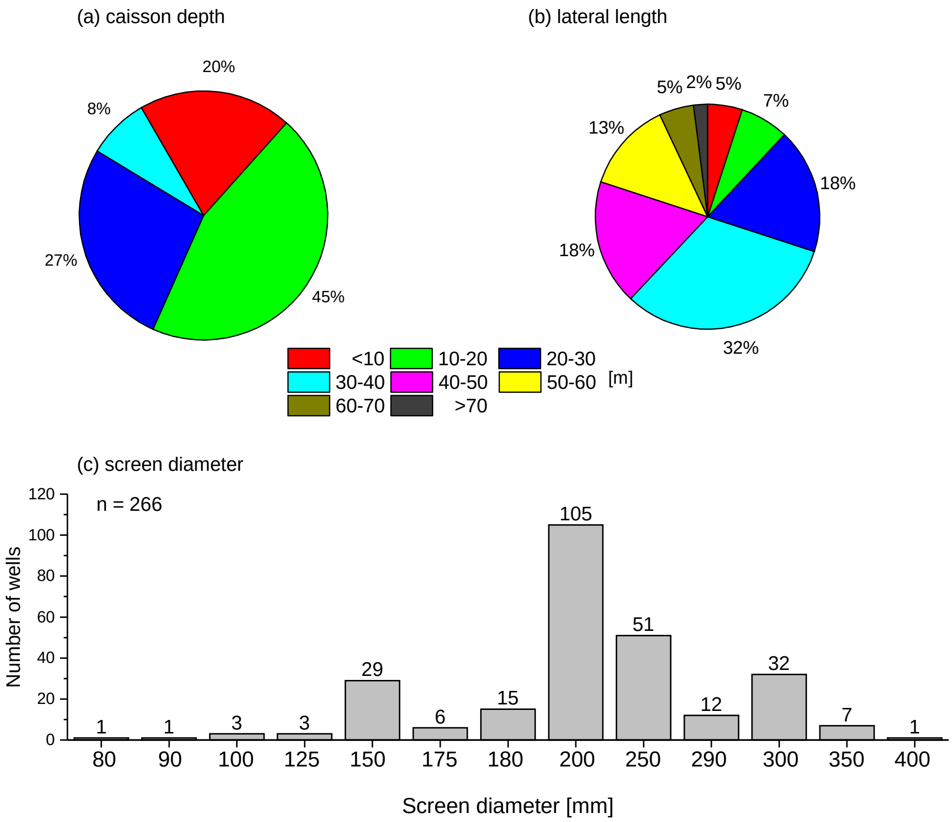

Figure S5: Statistical evaluation of technical parameters of RCWs, mostly from Germany: (a) caisson depth, (b) lateral length, (c) screen diameter (after data by Daffner et al. 2019a,b).

An analysis from Austria gave a similar picture: screen diameters vary between 120 and 600 mm, but 200 mm is the most common (Klambauer 2017). Caisson depths are usually between 10 and 20 m, although there is one well with a depth of 73 m. The number of laterals usually does not exceed 12 but there is one RCW that has 22 laterals installed in two levels.

One of the largest RCWs worldwide supplies water to the eastern parts of Warsaw, Poland (Kollis 1961). The caisson, which has a diameter of 11 m, was placed right into the river Vistula. It is equipped with 15 laterals, with a total length of 1472 m (98 m average), emplaced 4 to 8 m below the riverbed.

From the caisson, water is pumped through a tunnel of 300 m length to the treatment plant. The extraction rate ranges between 90,000 and 120,000 m^3/d (3750 to 5000 m^3/h). It is probably the only RCW to have a publicly known nickname, “Gruba Kaśka” (Fat Katie), and its own Wikipedia page (in Polish and German).

Supplement to section '*Modelling groundwater flow to horizontal wells*' of the main article

Electro-analog models

Before the advent of numerical models, electro-analog models were a common tool to study groundwater flow. The method utilizes the equivalence of the Darcy law for groundwater flow in porous media to the Ohm law for the flow of electricity. Several such electro-analog models were developed for horizontal wells (Kordas 1961; Nemecek 1961; Milojevic 1961, 1963; Glybov et al., cited in: Forkasiewicz and Vandenbeusch 1976; Debrine 1970; Förch 1973; Strzozda 1975; Müller et al. 2009). The aquifer is mostly emulated by an electrolytic tank, a typical and still used experimental set-up to study electrostatic flow fields. As an alternative, electrically conductive paper was also used to obtain 2D steady-state flow fields (Strzozda 1975; Müller et al. 2009). The electrolytic tank consists of a water tank filled with conductive water (tap water or dilute salt solutions), its bottom being a plate electrode. Applying a voltage to metal electrodes installed in the tank induces a current across the electrolyte for which the equipotentials – and thus the flow paths perpendicular to them – can be measured. For shallow tanks, a 2D field is obtained, with deeper tanks, a 3D field. For an RCW model, the electrodes play the role of the laterals, which are installed at side one of the sidewalls of the tank. Debrine (1970) applied six potential differences, ranging between 2 and 12 volts for each geometrical set-up. While Debrine (1970) studied one lateral, Kordas (1961) used three and Milojevic (1961) eight. By varying the rod length, the influence of partial penetration can be tested (Debrine 1970). The influence of the elevation of the lateral above the tank base can also be studied. Debrine (1970) compared the results to the analytical model developed by Hantush and Papadopulos (1962) in order to test their assumption of uniform flux over the lateral length and found a good agreement. Similar to the sandtank models, the results by Glybov et al., cited in: Forkasiewicz and Vandenbeusch (1976), Kordas (1961) and Milojevic (1961, 1963) were used to derive semi-empirical equations predicting the overall yield of the well and the yield distribution of and along the individual lateral. These were then upscaled to field dimensions.

Physical sand tank models

Falcke (1952) used a 1:20 scale semi-cylindrical model of 200 cm radius, with 1 m height and 7 m^3 volume. In the standard case, it was equipped with four laterals of 122 cm length each and 1 cm diameter (brass mesh). The high open screen area of 45% allowed a desanding of the vicinity of the lateral. One lateral was directly aligned against the outer glass wall, allowing flow paths around it to be visualized, using small potassium permanganate crystals buried into the sediment. The hydraulic conductivity of the sand (grain size 0 to 3 mm) was 8×10^{-4} m/s. Water levels were fixed at the outer rim and the shaft. The aquifer was unconfined, therefore readings of water levels were negatively affected by capillary effects which are unavoidable at this small scale. Mean flow velocities in the sandtank were found to be 0.03 cm/s, which is significantly higher than in nature in most cases. Based on the controlled boundary conditions, measured heads and visualized flow paths, graphical flow nets were constructed for each set-up. He summarized the findings from his physical model experiments into a dimensionally inconsistent empirical relationship for the field scale (linear *Q-s* relation only), relating the yield of a well to its drawdown and a constant *C*, the latter comprising the hydraulic conductivity of the aquifer and a factor describing the geometry of the system. He was able to compare the results obtained from using this equation to *Q-s* plots from three actual wells, with some success.

$$Q = C \cdot s = (\beta \cdot K^{0.33} \cdot tg\alpha) \cdot s \qquad \text{(S1)}$$

with

Q = yield of well [m^3/s]

s = drawdown at shaft [m]

K = hydraulic conductivity [m/s]

C = constant

α = factor obtained from nomogram in Falcke (1952), based on diameter, number and length of laterals, as well as aquifer thickness

β = 0.30-0.34

Kotowski (1985) transferred his model findings to the field scale by the empirical equation

$$Q = 2.58 \cdot K \cdot n_\mathrm{l}^{0.390} \cdot L_\mathrm{l}^{0.621} \cdot z^{-0.075} \cdot s_\mathrm{p}^{0.940} \quad \text{(S2)}$$

with

Q = yield of well [m³/s]

K = hydraulic conductivity of aquifer [m/s]

s_p = drawdown at shaft ($1.0 \leq s_\mathrm{p} \leq 5.0$) [m]

n_l = number of laterals ($4 \leq n \leq 12$)

z = elevation of lateral ($3.0 \leq z \leq 6.0$) [m]

L_l = length of screen of lateral ($12.5 \leq L_\mathrm{l} \leq 62.5$) [m]

at H_w = 9.0 m above aquifer base, r_l = 0.124 m, L_bc = 3.0 m, $K = 2.05\times10^{-3}$ m/s

with

L_bc = length of blind casing [m]

For his second physical model series, Kotowski (1985, 1988) derived the empirical equation for the actual scale of wells as

$$Q = 11.94 \cdot K \cdot n_\mathrm{l}^{0.248} \cdot L_\mathrm{l}^{0.417} \cdot z^{-0.110} \cdot s_\mathrm{p}^{0.929} \quad \text{(S3)}$$

with

s_p = $2.0 \leq s_\mathrm{p} \leq 5.0$ m and otherwise the same constraints as above.

Ersatzradius method

In order to calculate the ersatzradius, Schneebeli (1966) arrived at

$$r_{\mathrm{w}} = \frac{L_{\mathrm{l}}+r_{\mathrm{c}}}{\sqrt[n]{4}} \qquad \text{(S4)}$$

This leads to values for the equivalent radius, at $n_l = 6$ of around 0.8, which rise to 0.9 for more than 12 laterals. Deducting typical caisson radii will make the result similar to the equations shown in the main text. Glybov et al., cited in Forkasiewicz and Vandenbeusch (1976), used an electro-analog model to arrive at

$$r_{\mathrm{w}} = [(0.2 + 0.54\log n_{\mathrm{l}}) \cdot L_{\mathrm{l}} - 0.2b] \cdot \frac{(1.06+6.34r_{\mathrm{d}})}{\log d_{\mathrm{l}}} \qquad \text{(S5)}$$

with

r_{d} = nominal radius of lateral

d_{l} = nominal diameter of lateral

n_{l} = number of laterals

L_{l} = length of lateral

b = thickness of confined aquifer

Analytical and semi-analytical models

Thiem (1870) developed his analytical model when studying options for the water supply of Winterthur, Switzerland. It is a simplified adaption of the Dupuit-Thiem model for a fully penetrating vertical well in a homogeneous aquifer located on an island with a circular constant-head boundary. For the drain, he considered flow in a rectangular aquifer bounded at one side by a constant head boundary (river) and the drain at the other side with a lower head (Fig. S6a). The model calculates the water table between these heads in a vertical cross-sectional 2D view. The region to the right of drain, which could provide background groundwater flow, is ignored. Although the model considers a finite length of the drain, the semi-radial flow at the two end of the drains is also ignored. As such the model is only valid for a cross section vertical to the drain and of limited practical use.

$$H^2 - h^2 = \frac{2 \cdot Q}{l \cdot K} \cdot E \qquad \text{(S6)}$$

where

E = distance between river bank and drain

l = length of drain (in y direction)

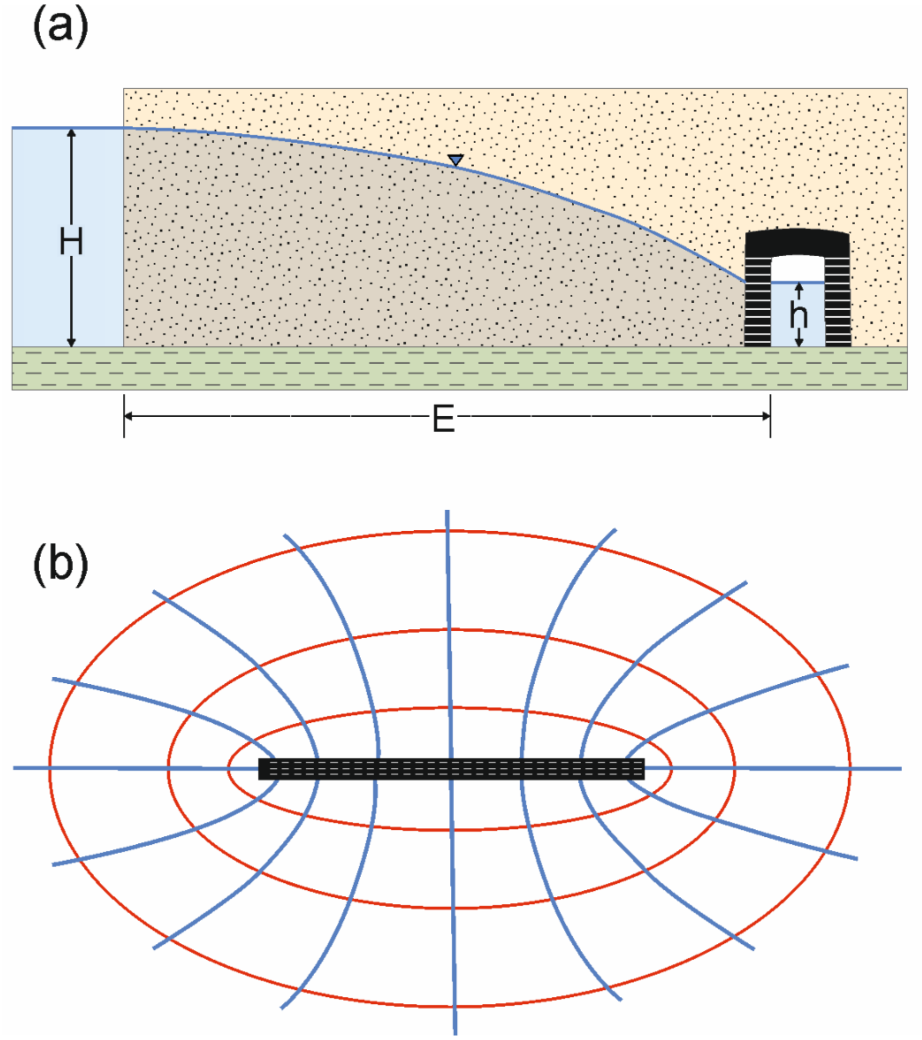

Fig. S6 Geometry for the analytical models by (a) Thiem (1870) for bank filtration from a river (left) to a drain (does not consider flow from the right-hand side of the drain) and (b) Forchheimer (1886), plan view, red = equipotentials, blue = flow paths. *H* = water level in river, *h* = water level in drain, both at steady state, *E* = distance between river and drain.

The first model that allowed the full drawdown field around a horizontal drain in flow to be addressed was developed by Philipp Forchheimer (Forchheimer 1886). He considered steady state flow in a 2D plan view aquifer without regional background flow (Fig. S6b). The drain of length f_d is infinitesimally thin but covers the entire thickness of the aquifer. The equipotentials around the drain are considered to be ellipsoids, which can be described by complex numbers. The focal points of the ellipsoids are the tips of the drain at ± ½f_d (Fig. S6b). Assuming that, at larger distances, the drain can be approximated as a point sink ($f_d \rightarrow 0$), the heads at a distance r from this well can be obtained by

$$H^2 - h^2 = \frac{Q}{\pi \cdot K} \cdot \ln \frac{r_1 + r_2 + \sqrt{(r_1 + r_2)^2 - f_D^2}}{4r} \qquad \text{(S7)}$$

with

r_1, r_2 = distance from the focal points to a point xy

r = distance from well (center of drain)

Measuring the heads h_1 and h_2 at two points at distances y_1 and y_2, both located on a line going through the middle of the drain and perpendicular to it, allows calculating the hydraulic conductivity of the aquifer after

$$K = \frac{Q}{\pi \cdot (h_1^2 - h_2^2)} \cdot \ln \frac{\sqrt{f_D^2 + 4y_1^2} + 2y_1}{\sqrt{f_D^2 + 4y_2^2} + 2y_2} \qquad \text{(S8)}$$

The same can be done for the *x* direction, that is a line in the extension of the drain, with two heads h_1 and h_2 measured at two distances x_1 and x_2.

Ellipses and hyperboles, which have the same focal points, intersect at right angles. The flow paths towards a drain can thus be described as hyperbolas with focal points at both tips of the drain. The ellipses and hyperboles shown in Figure S6b thus form a flow net. The oblique asymptotes of all hyperbolas pass through the middle of the drain and will meet the hyperbolas in infinity. Since flow towards the drain at infinity is uniformly distributed, the same amount of water flows between two branches of a hyperbolae, which asymptotes enclose the same angle. The yield of one meter of drain (flow from both sides) is then

$$q = \frac{Q}{\pi \cdot \sqrt{\frac{1}{4} \cdot f_{\mathrm{D}}^2 - \xi^2}} \qquad \text{(S9)}$$

with

ξ = distance from middle of drain

This equation predicts that the inflow is non-uniform and lowest in the middle of the drain. It will increase strongly towards the tips, theoretically reaching infinity. Forchheimer (1886) warned that high flow velocities at the tips might induce the suffusion of sand there. This model was thus the first to correctly predict the uneven inflow distribution of horizontal screens. Finally, Forchheimer (1886) also expanded his treatment of drains by considering the drawdown field around several drains emanating from one common point of origin, thus a regular, star-shaped arrangement of drains, similar to a modern RCW. He concluded that to produce the same flow rate of water, such an arrangement would always require a greater total drain length compared to a single drain. For additional drains, the ratio of additionally required length is

Number of drains	3	4	5	6
Additional length required	1.2	14	1.6	1.8

Schneebeli (1966) followed an approach similar to Forchheimer (1886), based on the complex analysis of ellipsoids and hyperbolas, however, without mentioning Forchheimer. It is therefore unclear whether Schneebeli was aware of the older study. Strack (1989) extended the solution of Forchheimer (1886) by adding a uniform cross-flow using conformal mapping, and presented an example for a RCW with six laterals in otherwise uniform flow, where the heads are constant along all laterals. Another analytical model of HWs was developed by Polubarinova-Kochina (1955), who studied the yield Q of a number n of horizontal drains of length l with uniform influx distribution, situated in the same plane of a quasi-infinite aquifer in a fan-like manner, with elliptical equipotentials around the drain.

The most frequently cited analytical model of Hantush and Papadopoulos (1962) assumes a uniform flux along the length of the lateral. Further assumptions are a limited drawdown ($s < 0.25 \cdot H_0$), a small percentage of water release due to aquifer compaction and that the caisson radius is significantly smaller than the lateral length ($r_c << L_l$). The drawdown for the i^{th} of a group of i laterals after a long time of pumping (quasi steady state, valid for $t > 2.5 \cdot b^2/v'$ and $t > 5 \cdot (r^2+L_i^2)$) is then

$$s_i = \frac{Q_i/L_i}{4 \cdot \pi \cdot K \cdot b} \cdot \begin{Bmatrix} \alpha \cdot W\left(\frac{\alpha^2+\beta^2}{4 \cdot v' \cdot t}\right) \cdot -\delta \cdot W\left(\frac{\delta^2+\beta^2}{4 \cdot v' \cdot t}\right) + 2L_i - 2\beta\left(\tan^{-1}\frac{\alpha}{\beta} - \tan^{-1}\frac{\delta}{\beta}\right) \\ + \frac{4 \cdot b}{\pi} \cdot \int_{n=1}^{\infty} \frac{1}{n} \cdot \left[L\left(\frac{n \cdot \pi \cdot \alpha}{b}\right), \left(\frac{n \cdot \pi \cdot \beta}{b}\right) - L\left(\frac{n \cdot \pi \cdot \delta}{b}\right), \left(\frac{n \cdot \pi \cdot \beta}{b}\right)\right] \cdot \cos\frac{n \cdot \pi \cdot z}{b} . \cos\frac{n \cdot \pi \cdot z_i}{b} \end{Bmatrix} \quad \text{(S10)}$$

with

$$\alpha = r \cdot \cos(\theta - \theta_i) - r_c$$

$$\beta = r \cdot \sin(\theta - \theta_i)$$

$$\delta = r \cdot \cos(\theta - \theta_i) - l'$$

$$r = \sqrt{x^2 + y^2}$$

$$l' = r_c + L_i$$

$$v' = \frac{K \cdot b}{S_y}$$

$$L(u, \pm w) = -L(-u, \pm w) = \int_0^u K_0 \cdot \left(\sqrt{w^2 + y^2}\right) dy$$

$$w = 2 \cdot n \cdot b + z_i - z$$

$$u = \frac{r^2 \cdot S}{4 \cdot K \cdot b \cdot t}$$

and

K = hydraulic conductivity [L/T]

Q_i = pumping rate of i^{th} lateral [L^3/T]

L_i = length of i^{th} lateral [L]

b = thickness of confined aquifer or initial water-saturated thickness of water-table aquifer [L]

S = specific yield, effective porosity

r_c = radius of caisson [L]

N = number of laterals

n = 1, 2, 3, 4, … ∞ (integer counter)

r, z, Θ = cylindrical coordinates (z positive downwards)

r_i, z_i, Θ_i = cylindrical coordinates of i^{th} lateral

x, y, z = rectangular coordinates

t = time since start of pumping [T]

$K_0(u)$ = zero-order modified Bessel function of the second kind

$W(u)$ = well function = $Ei(-u)$, the exponential integral, which can be approximated by a Taylor series (Theis 1935)

With $z = 0$, the approximate drawdown of the water table is obtained. At a radius from the caisson, where $r \geq (r_c + L_i + b)$, the bracketed term in Equation S10 approaches zero. If $r > 5 \cdot (r_c + L_i)$, the drawdown can be described by the Theis (1935) equation. Hantush and Papadopoulos (1962) also provide solutions for short times of pumpage.

$$s = \frac{Q}{4 \cdot \pi \cdot K \cdot b} \cdot W(u) \qquad \text{(S11)}$$

One of the first transient analytical models is the one described by Ferris (1962). Similar to Forchheimer (1886), the lateral is a fully penetrating and infinitesimally thin (no storage) vertical channel, placed in a homogenous aquifer of infinite extent. The flow rate, however, is uniform along the length of the channel, thus ignoring the distortion of the flow field at the tips of the channel (Kawecki 2000).

$$s = \frac{Q_L}{T_x} \cdot \sqrt{\frac{T_x \cdot t}{S}} \cdot \mathrm{ierfc}\left(x \cdot \sqrt{S/T_x \cdot t}\right) \qquad \text{(S12)}$$

with

Q_L	=	discharge rate per unit of channel length (constant)
T_x	=	transmissivity in x-direction
S	=	storage coefficient
x	=	distance from channel
t	=	time
ierfc	=	first repeated integral of the error function

Based on Borisov (1964), Beljin and Losonsky (1992) developed a steady state model for HWs. The model assumes a uniform distribution of flux over the length of the screen. They also adapted their model to include effects of aquifer anisotropy, off-centre location of the well and the presence of a skin layer.

The analytical model of Gringarten et al. (1974), which describes the transient flow towards a single highly permeable fracture, has also been adapted for HWs, since a lateral functions in a similar way as a fracture (e.g. Joshi 1991; Beljin and Losonsky 1992; Giese et al. 2019). The fracture width is replaced by the diameter of the well screen, which is assigned an infinite hydraulic conductivity.

Another transient model for both drawdown and recovery of a RCW lateral was derived by Odeh and Babu (1990). They distinguish between four phases, early radial flow, early linear flow, late pseudo-radial and finally late linear, although – depending on the circumstances - some phases may be too short to be distinguishable. Their model is also able to address aquifer anisotropy and skin effects.

Williams (2013) adapted the Cooper and Jacob (1946) equation for transient flow to a fully penetrating vertical well for horizontal and slant wells. Cooper and Jacob (1946) had shortened the asymptotic series expansion of the well function by Theis (1935) by truncating after the ln *u* term. It should be noted that this is valid only for small distances and large times. This yields

$$s = \frac{2.3 \cdot Q}{4 \cdot \pi \cdot K \cdot b} \cdot \log \frac{2.25 \cdot K \cdot b \cdot t}{r^2 \cdot S} \qquad \text{(S13)}$$

In order to transpose this to a horizontal or slant well, Williams (2013) used a number of i point sinks, with intensities Q_i each, along the vertical projection of the well screen and distributed the total discharge *Q* over these sinks (horizontal and vertical wells are just special cases of a slant well, angle = 0° and 90°, respectively). The drawdown obtained is the one that would be measured in a fully penetrating observation well. The approach yields a 2D drawdown field, while the Hantush and Papadopoulos (1962) model allows calculation of drawdown for any plane of the aquifer thickness.

$$s = \frac{2.3 \cdot Q}{4 \cdot \pi \cdot K \cdot b} \cdot \log\left(\frac{2.25 \cdot K \cdot b \cdot t}{S}\right) - (2/n_s) \cdot \log(RP_1 \times RP_2 \times RP_3 \times \ldots \mathrm{RP_{ns}}) \qquad \text{(S14)}$$

with

n_s = number of point sinks along the vertical projection of the well screen

RP_x = distance from (arbitrary) observation point to point sink x [L]

The Williams (2013) method uses the same assumptions and simplifications of the Cooper and Jacob (1946) model, including uniform inflow along the screen. Williams (2013) was able to obtain a good fit to the more explicit Hantush and Papadopoulos (1962) model using 10 to 20 line sinks for his example well (screen length 150 m). If wanted, the flow rate to the individual point sinks can be modified to emulate an uneven inflow distribution.

As the model by Hantush and Papadopulos (1962) is often considered to be some kind of gold standard, several authors have used it as a reference case to compare their newly developed models against (e.g. Huang et al. 2012). Here, we compare the 3D models of Zhan et al. (2001) and Joshi (1991) against Hantush and Papadopulos (1962) for a confined aquifer (z = height of laterals); the 3D model of Zhan and Zlotnik (2002) against Hantush and Papadopulos (1962) for an unconfined aquifer (z = height of laterals); and the 2D Williams (2013) model against Hantush and Papadopulos (1962) for average drawdown in a confined aquifer (Fig. 18). The models of Zhan and Zlotnik (2002), Zhan et al. (2001) and Williams (2013) produce almost identical drawdown distributions to that of Hantush and Papadopulos (1962). The solution of Joshi (1991), which is derived from the oil literature (e.g. Gringarten and Ramey 1973; Daviau et al. 1988) and uses source function and Green's function methods, produces slightly less drawdown than those of Hantush and Papadopulos (1962) and Zhan et al (2001), which are based on a Laplace transform approach.

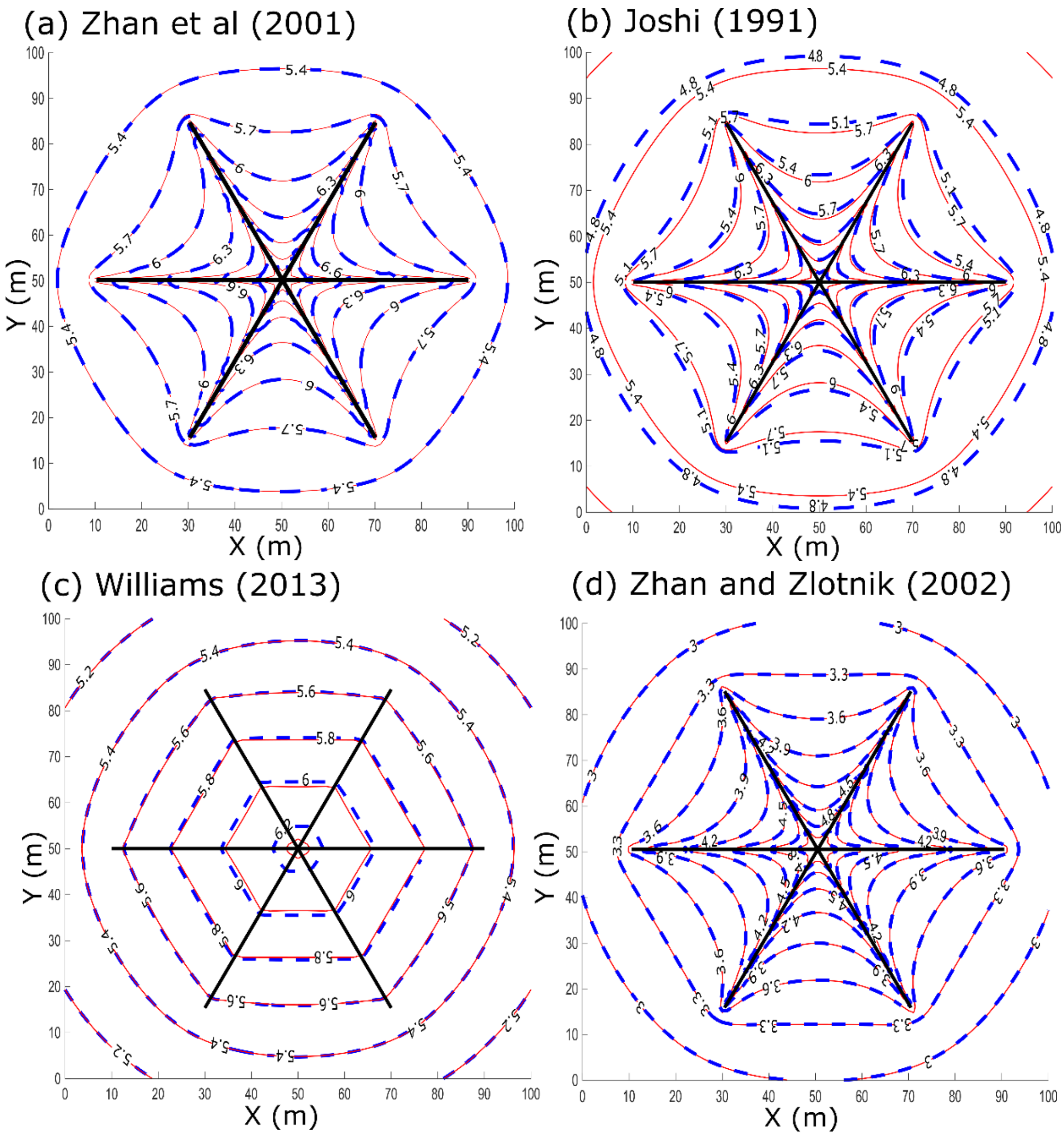

Fig. S7. Comparison of selected analytical and semi-analytical models against the Hantush and Papadopulos (1962) model (red lines in all images): (a) Zhan et al. (2001) for a confined aquifer, (b) Joshi (1991) for a confined aquifer, (c) Williams (2013) for average drawdown in a confined 2D aquifer; and (d) Zhan and Zlotnik (2002) for an unconfined aquifer. For the 3D models (a,b,d), the drawdown at the height of the laterals is plotted.

Tab. S1 Overview of analytical models and their boundary conditions

Reference	Aquifer type	Collector well	Boundary condition			Transient or steady state	Anisotropy	Well bore storage	Slant well	Skin effect
			Infinite aquifer	Near stream	Under riverbed					
Polubarinova-Kochina (1955)	U, C	×	×		×	S	I			
Hantush and Papadopulos (1962)	C, U	×	×	×	×	T	I		×	
Joshi (1988)	C		×			S	I			
Strack (1989)	U	×	×			S	I		×	
Joshi (1991)	C		×			T	A		×	
Tarshish (1992)	U	×			×	S	I		×	
Kawecki (2000)	C, U		×			T,S	A			×
Zhan et al (2001)	C	×	×			T	A		×	x
Park and Zhan (2002)	L	×	×			T	A	×	×	×
Zhan and Zlotnik (2002)	U	×	×			T	A		×	

Zhan and Park (2003)	L	×			×	T	A	×	×	×
Hunt (2005)	L	×	×			T	A		×	
Samani et al (2006)	U	×	×			T	A		×	
Sun and Zhan (2006)	L	×			×	T	A	×	×	
Dou (2008)	C		×			S	A	×		×
Tsou et al (2010)	C	×		×		T	A			×
Huang et al (2011, 2012, 2016)	U	×		×		T	A		×	
Batu (2012)	C	×	×			T	A		×	
Williams (2013)	C, U, L	×	×			T	I		×	
Iktisanov (2020)	C	H	x			S	I,A		x	x

C = confined, U = unconfined, L = leaky, T = transient, S = steady state, A = anisotropic, I = isotropic

Economic considerations

One of the most commonly used arguments against HW and especially RCW is the higher construction costs, which are a result of the expensive caisson and the special machinery employed for placement of the laterals. In many countries the choice of drilling companies specialized in RCW is limited. The prices given in the following represent the price level at the time of publication. They were not adjusted to the price levels of today. Values in West-German Deutschmark were converted to Euro (€).

For sites in the US, Spiridonoff (1964) quotes a range of actual construction costs for bank filtration RCW systems between 0.15 to 1.2 Mio. US-$, depending on the design capacity, ranging from 315 to 7,900 m^3/h (2 to 50 mgd). He concluded that using RCW for bank filtration would be cheaper by up to an order of magnitude compared to that of direct treatment of river water, which requires the construction of treatment and settlement ponds and the constant application of flocculants and other chemicals.

The costs of an RCW can easily reach that of several vertical wells. However, through its higher yield, it also replaces several vertical wells. Only a few studies have directly compared the costs for construction and long-term operation of vertical versus radial collector wells. Blasche (1987) used the total screen area as the main parameter for a comparison of RCW to vertical wells. For a RCW with a caisson depth of 22 m, a caisson diameter of 2.5 m and five laterals of 300 mm diameter and a total length of 165 m, he arrived at a screen area of 146 m². For such a RCW, equipped with a plastic-coated screen material, he estimated construction costs of 220,000 €. A stainless steel screen would raise the cost to 228,000 € (prices given at 1987 level, not corrected). To reach an equivalent screen area, he estimated that six vertical wells of 25 m depth would be needed, each of 1,300 mm drilling and 600 mm screen diameter. The construction costs, for the same two screen material options given above, would amount to 232,000 to 298,000 €. In this case, the RCW would be on par with the vertical wells regarding the construction costs. It should be noted that the diameters for the vertical wells Blasche (1987) suggests are quite high.

A very detailed construction cost analysis, comparing two options, a single RCW versus several vertical wells, was published by Hüper (1991). He considered the following factors:

(1) Exploration: costs for hydrogeological site exploration (test drilling, geophysics etc.) are probably in the same range for both options
(2) Construction costs of well: these will almost always be higher for the RCW, parity can only be reached if it can replace several vertical wells
(3) Land acquisition: while an RCW needs one larger parcel of land, each vertical well needs its own plot, especially when a protection zone around it is required. Compensations or rents might also be necessary for the pipelines. In the case of public water supply from public land this cost factor might cease to apply.
(4) Additional costs (e.g. access roads, fences, well head): vertical wells require individual access roads, fencing and well heads for each of them. The well head of a RCW is much bigger and more expensive than that of a single well but needs only one access road and fence.
(5) Pumps: each vertical well requires its own submersible pump (exception: siphon systems), while in a RCW one or more larger pumps can be installed centrally in the caisson
(6) Monitoring and process control (e.g. switchboards, pressure transducers etc.): the same as in (5) applies
(7) Pipelines: for a RCW there is need for only one pipeline towards the water treatment plant. For vertical wells, in order to avoid excessive drawdown due to overlapping cones of depression, they need be placed at a certain distance to its next neighbor, incurring additional costs for laying longer pipelines, connections and valves.
(8) Power supply: the same as in (3) applies to the power supply via electrical cables.

Hüper (1991) made model calculations for two examples, comparing one RCW to five and eight vertical wells, respectively, in shallow aquifers (Tab. S2). The number of vertical wells was, again, obtained by finding similar screen areas for both RCW and vertical wells. The distance between the vertical wells was set to 50-80 m for the calculation of the pipeline length.

For a RCW in Laren, the Netherlands, with five laterals of around 80 m length (Preussag type), Pluijmackers et al. (2014) give a nominal pumping rate of 560 m^3/h, which is estimated to replace six to eight vertical wells. This RCW has a caisson of 17 m depth (laterals at 14 m depth) and 2.8 m diameter, Pluijmackers et al. (2014) quote an investment value of 719.000 €. It should be noted that it is one of the few with the more expensive dry shaft.

Tab. S2: Two examples of construction costs for RCW versus vertical wells (data by Hüper 1991). Costs do not include taxes.

	Example 1 (sandy aquifer)		Example 2 (gravelly aquifer)	
	5 vertical wells	1 RCW	8 vertical wells	1 RCW
Exploration	30,000	30,000	30,000	30,000
Well construction	216,000	277,00	311,000	263,000
Land acquisition	40,000	31,250	90,000	31,250
Additional cost	49,000	29,500	110,200	29,500
Pumps	57,500	35,000	80,000	38,000
Control	25,000	11,500	44,000	14,000
Pipelines	40,500	17,5000	91,000	25,000
Energy supply	9,000	3,300	38,550	4,200
SUM	467,000	435,050	794,750	434,950

The operational costs of a RCW are lower than those for a vertical well, due to

- lower drawdown, thus saving electricity for pumping
- smaller number of pumps and installations to maintain
- better pump efficiency
- less mixing of aerobic and anaerobic water, leading to
    - less well clogging and thus longer rehabilitation intervals
    - less frequent flushing of iron removal filters and replacement of filter material
- shorter pipeline length (lower losses, less cleaning)
- lower analysis costs, one sample instead of six to eight

Hüper (1991) estimated that a RCW replacing five vertical wells leads to three meter less drawdown. Additionally, the single 300 m^3/h pump used in a RCW caisson has a better efficiency than five submersible pumps in vertical wells. Together, these effects can lead to a saving of up to 6,000 €/a for electricity. Pluijmackers et al. (2014) arrive at similar number. They calculated an electricity cost of 6,300 €/a for the Laren RCW, compared to 10,300 €/a for vertical wells.

Pluijmackers et (2014) estimate that the RCW has a rehabilitation interval ranging between 3 to 10 years for the different laterals, compared to every 1-2 years for the previously used vertical wells. It should be noted that the rehabilitation of a RCW is significantly more expensive than that of a vertical well. Hüper (1991) estimated rehabilitation costs of up to 50,000 € for an RCW, compared to 7,500 to 25,000 € for one vertical well. Taking together all cost savings and deducting depreciation and capital costs of 54,000 €/a, Pluijmackers et al. (2014) arrived at a total annual saving of 25,600 € for the Laren RCW, compared to vertical wells.

Regarding flexibility, vertical wells have some advantages over RCW. It is relatively easy to close down a vertical well of a well field for a rehabilitation or to add another well in case of increasing demand, while any rehabilitation or reconstruction measure in an RCW requires careful planning and execution through specialized companies.

The studies by Blasche (1987), Hüper (1991) and Pluijmackers et al. (2014) all agree that a RCW can replace five to eight vertical wells and that considering both the total construction and operational costs, RCW can be an interesting option also from an economic point of view. A word of caution is advised: all these of the mentioned authors are or were employed by companies operating or building RCW.

Ageing, rehabilitation and reconstruction of horizontal wells

Horizontal wells generally suffer from the same ageing problems as verticals wells (Fig. S8), including corrosion, mineral precipitation, biofilm formation and particle clogging (Houben 2003). The mild steel of the older Ranney wells makes them prone to corrosion. The formation of iron oxide incrustations (Fig. S8a), one of the most common ageing processes in vertical wells, is an effect of the reactions between waters of different quality coming from different depths, which are mixed over the screen length (Houben 2003). Horizontal wells pump water preferentially from a certain depth interval around the laterals, thus causing less mixing and incrustation. It is therefore often assumed that RCW age more slowly than vertical wells and require less frequent rehabilitations (Hüper 1991; Houben and Treskatis 2007). Nevertheless, incrustations have been documented from RCW as well (Spiridonoff 1964; Houben and Treskatis 2007; Dimkic´ et al. 2011, 2012). The large screen slots of Ranney wells and the incomplete development or lack of a gravel pack, often lead to the intake and accumulation of sand in the laterals (Fig. S8b), which decreases their weak hydraulic performance even more. Božović et al. (2020) found that their RCW in Belgrade suffered from both particle colmation and iron oxide incrustation. The latter was strongly correlated to the flow velocities at the lateral, with higher velocities leading to stronger incrustations.

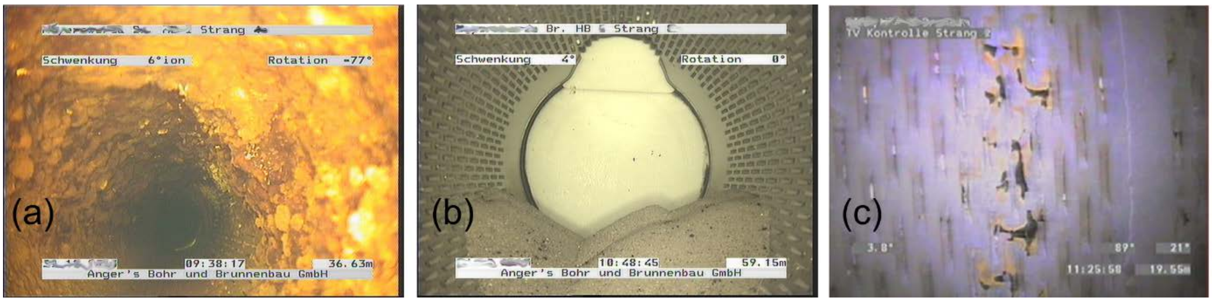

Fig. S8 Camera inspection of RCW laterals showing different ageing problems: (a) incrustations (iron oxide), (b) sand accumulation, (c) corroded screen (Photos: Daffner).

In order to identify and remove particulate, biological and mineral clogging deposits from laterals, the same inspection and rehabilitation tools as used for vertical wells are available. However, they have to be adapted to install them horizontally, often using concepts from pipeline inspection. Cameras are therefore put on wheels, allowing them to be driven along the lateral. The usually smaller diameter of laterals compared to vertical wells has to be considered as well, as it limits the choice of methods. In RCW with wet shafts, the caisson has to be pumped dry before inspection or rehabilitation, which is a tedious and expensive measure. Alternatively, specialist divers are often used to run in cameras and rehabilitation equipment. These issues explain why the rehabilitation of a RCW or HDD is much more expensive than that of a vertical well. Usually the whole RCW has to be put out of action during inspection or rehabilitation.

Mechanical rehabilitations of laterals usually employ high-pressure hydraulic or impulse techniques. For the former, packer chambers extracting water at high flow rates or counter-rotating nozzles ejecting water at high pressure are often used. The nozzles face slightly backwards, which moves the system forward and at the same time facilitates the transport of loosened material towards the shaft. Chemical rehabilitations of RCW have also been performed, e.g. with acids (Spiridonoff 1964). Distributing the chemicals over the entire length of the long lateral is often a challenge. A recent development therefore was to install a continuous feeder line for acids with the lateral during construction. This allows easy and repeated acid applications without the need to insert equipment into the lateral, which might become stuck.

A prevention method against the build-up of iron oxide incrustations in laterals is the patented anodic polarization of the (metal) lateral surface (Houben and Treskatis 2007). This creates a repulsive force between the screen surface and the iron oxide particles, which are then both positively charged. Titanium electrodes are installed at the tip of the lateral and exposed to a direct current. The method has shown promising results in laterals of several RCWs in Germany (Daffner et al. 2019b).

If rehabilitation of the laterals does not lead to the desired results, the well needs to reconstructed. The most common reconstruction technique for RCW is the installation of new screens (Moses and Riegert 2004). If no extra portholes from the time of construction are available, new ones are drilled through the caisson wall (Fig. S9), often at a different, usually higher level above the base. The old laterals are usually backfilled with impermeable material. Redrilling and pulling the old laterals is not an option due to their length and the cost involved. The installation of new laterals cannot be repeated indefinitely, as too many portholes weaken the caisson and going upwards with new laterals incurs the danger of them running dry during pumping.

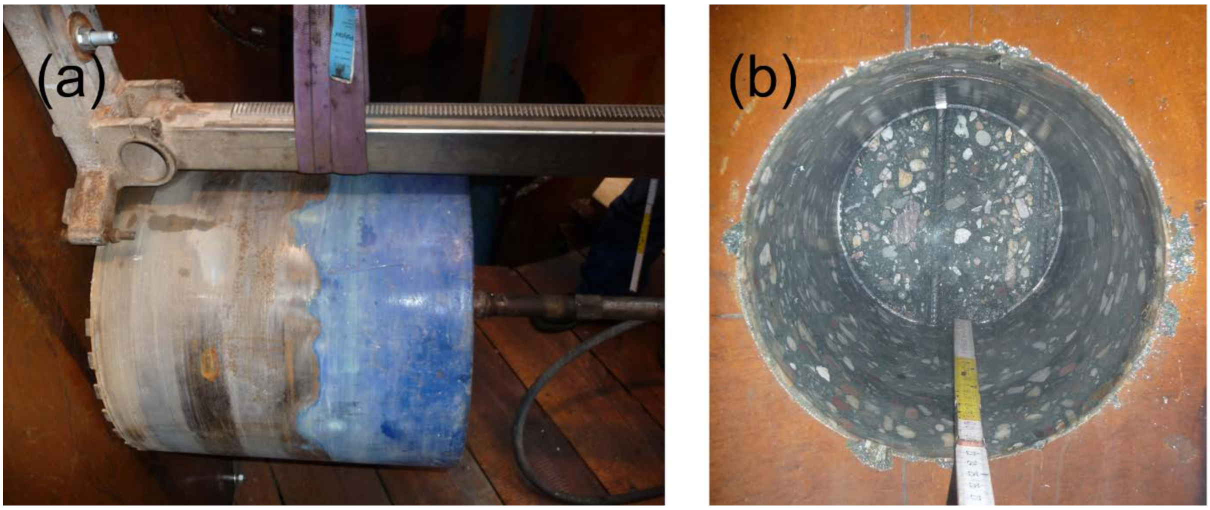

Fig. S9 Drilling of new portholes in existing caisson: (a) drilling tool, (b) view of unfinished drillhole showing concrete (Photos: Daffner).

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