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Ministry of Housing, Utilities & Urban Communities

SUPPORTING INNOVATION IN WATER & WASTEWATER IN EGYPT

GUIDELINE ON RIVERBANK FILTRATION IN EGYPT

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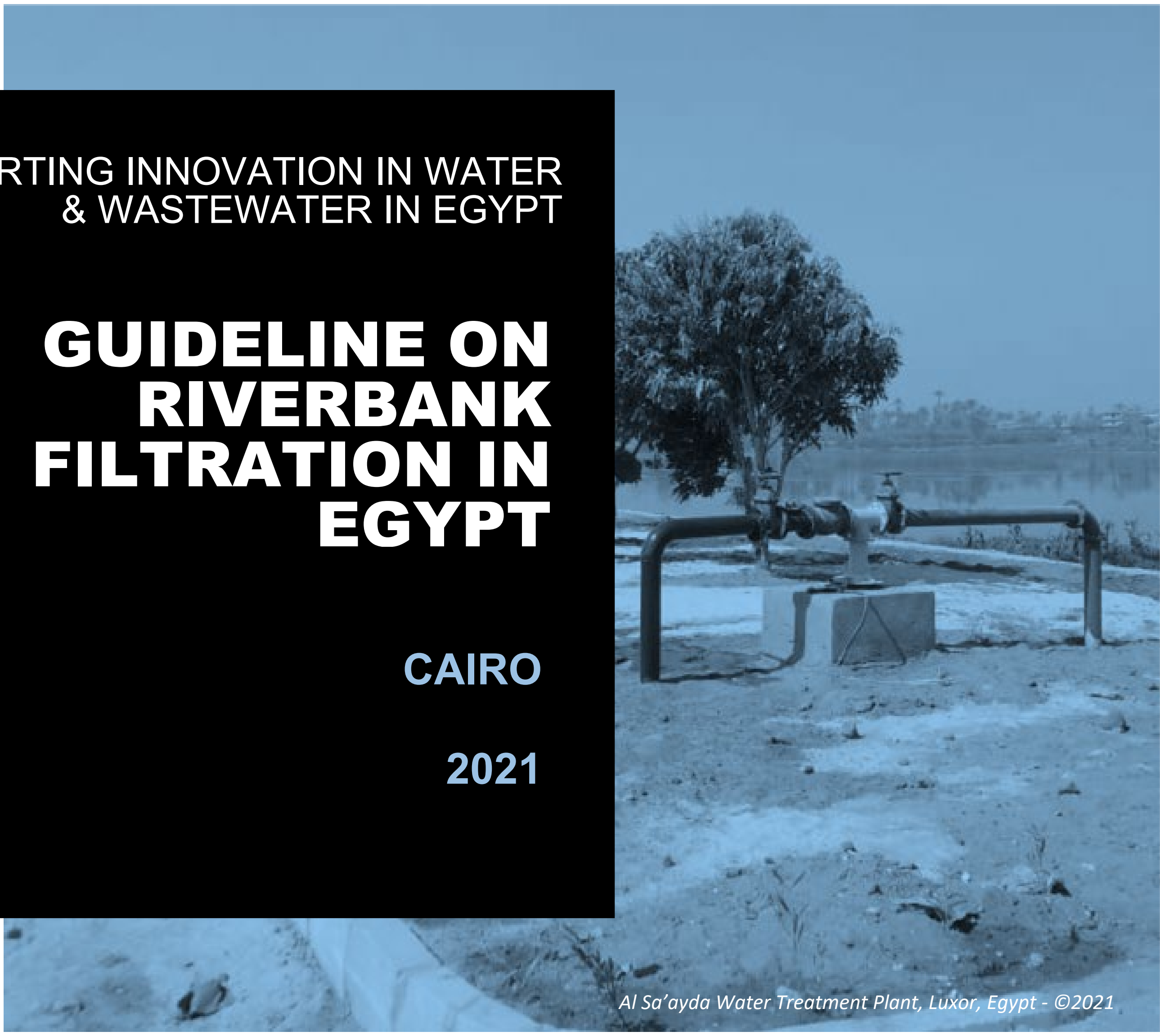

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GUIDELINE ON RIVERBANK FILTRATION IN EGYPT

PARTNERS

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Abbreviations

AOX	Adsorbable Organic Halogenes [µg/l]
AQTESOLV	AQuifer TEst SOLVer (aquifer test analysis software)
BDOC	biologically degradable organic carbon [mg/l]
BF	bank filtration
d_{10}	grain size diameter corresponding to 10% cumulative undersize particle size distribution [mm]
d_{60}	grain size diameter corresponding to 60% cumulative undersize particle size distribution [mm]
DALYs	disability adjusted life years
DBP	disinfection by-products
DOC	dissolved organic carbon [mg/l]
DW	deep groundwater well
EC	electrical conductivity [µS/cm]
GC	gas chromatography
GHB	general head boundary [m^3/s]
GIS	geographic information system
GW	groundwater
GWL	groundwater level
HCWW	Holding Company for Water and Wastewater
HPLC	high performance liquid chromatography
HTWD	University of Applied Sciences Dresden
IP	inner piezometer
IWRM	Integrated Water Resources Management
K	hydraulic conductivity [m/s]
LPCD	liters per capita per day
M	saturated thickness of aquifer [m]
MAR	managed aquifer recharge
masl	meter above sea level
mbgl	meter below ground level
MS	mass spectrometry
NOM	natural organic matter

OP	outer piezometer
OTP	organic trace pollutants
OW	observation well
PAH	polycyclic aromatic hydrocarbons
PW	production well, pumping well
Q	abstraction rate [m³/s]
RBF	riverbank filtration
s	drawdown in a well [m]
SS	suspended solids
SW	surface water
T	transmissivity [m²/s]
TDS	total dissolved solids [mg/l]
THM	trihalomethanes
TOC	total organic carbon [mg/l]
WL	water level
WSP	water safety plan
WTP	water treatment plant
WWTP	waste water treatment plant

1 Introduction to Riverbank Filtration in Egypt

1.1 Definition and purpose of RBF

Riverbank filtration (RBF) or simply bank filtration (BF, a unified term for river and lake bank / bed filtration) is a process in which the subsurface at a river, canal, reservoir or lake bank serves as a natural filter and biochemically removes potential contaminants present in the surface water (Fig. 1-1). RBF is initiated by the lowering of the groundwater (GW) table below that of an adjoining surface water (SW) table which induces SW to infiltrate through the permeable riverbed and river bank (or lake bank) into the aquifer as a result of the hydraulic gradient. The infiltration may be the direct result of an influent river under natural conditions or be induced by GW abstraction wells.

The pumped raw water is a mixture of up to three different water sources, which usually have different qualities. Main components of the raw water at most sites are the bank filtrate, meaning the infiltrating river water, and the landside groundwater, which flows into the wells from the landside. Under appropriate geohydraulic conditions, the third source is groundwater from the opposite riverside, flowing beneath the river towards the wells (Grischek & Paufler 2017).

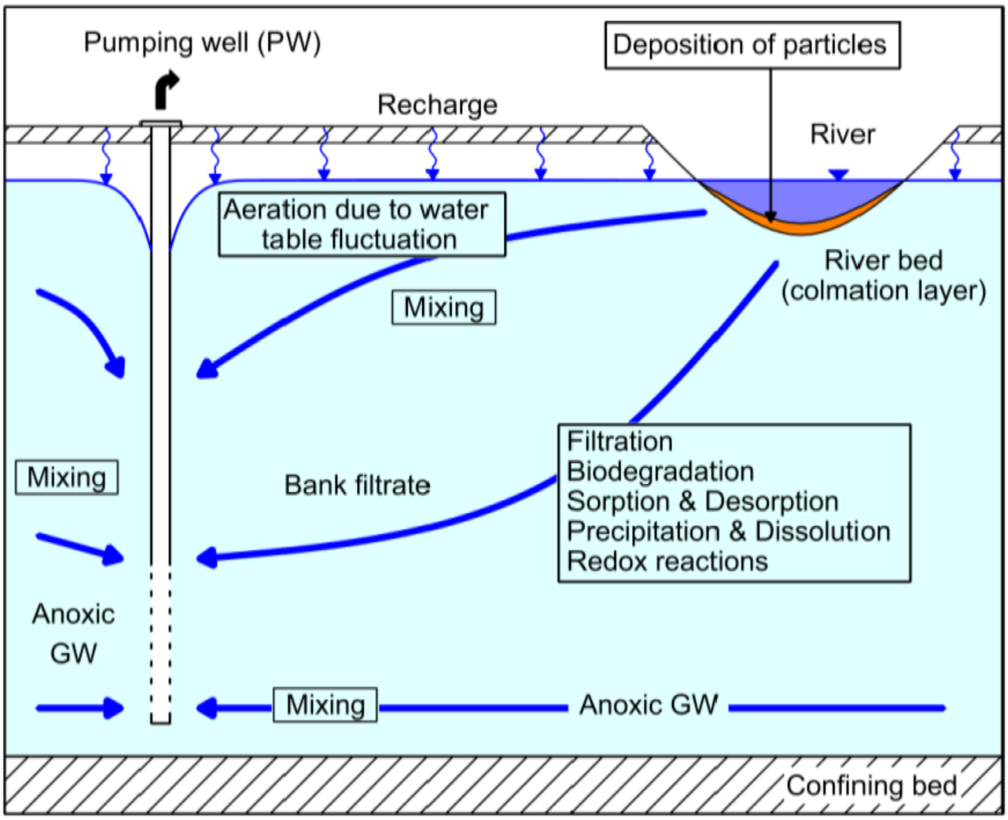

Fig. 1-1: Processes of riverbank filtration (Grischek & Paufler, 2017)

The aquifer serves as a natural filter and also biochemically attenuates potential contaminants present in the SW. Compared with direct SW abstraction, RBF with its effective natural attenuation processes removes suspended solids, particles, biodegradable compounds, bacteria, viruses and parasites; partly attenuates adsorbable compounds and equilibrates temperature changes and concentrations of dissolved constituents in the bank filtrate.

The success of RBF schemes is dependent on the microbial activity and chemical transformations that are commonly enhanced in the clogging layer within the river bed compared to those that take place in surface water or groundwater. The actual biogeochemical interactions that sustain the quality of the pumped bank filtrate depend on numerous factors including riverbed and aquifer structure and mineralogy, SW quality, particle content and composition, oxygen and nitrate concentrations in the SW, types of organic matter in the surface and ground water environments and land use in the local catchment area (Grischek et al. 2002).

For the quantitative and qualitative management of bank filtration systems, the catchment zones, infiltration zones, mixing proportions in the pumped raw water, flow paths and flow velocities of the bank filtrate need to be known. Flow conditions during bank filtration are commonly described using interpretations of water level measurements and hydrogeological modelling. An important factor is the formation of the clogging (colmation) layer within the riverbed (Fig. 1-1) that has a reduced hydraulic conductivity due to clogging from the input and precipitation of sediment particles, micro-organisms and colloids, precipitation of iron and manganese (hydr)oxides and calcium carbonate as well as gas bubbles. The hydraulic conductivity of the river bed varies with the dynamic hydrology and therefore cannot be regarded as constant.

During floods that have sufficient hydraulic transport energy, the riverbed can be reworked and the clogging layer eroded. In general, a higher portion of infiltrated water in the pumped raw water is expected due to removal of the clogging layer and the greater hydraulic gradient from the river to the wells. Severe floods can also have a detrimental effect by eroding the river bank and thus affecting the installed production wells and the GW flow regime; for example flow path lengths in the aquifer and retention times of the bank filtrate.

The beneficial attenuation processes listed subsequently result mainly from mixing, biodegradation and sorption processes within two main zones (Fig. 1-1): the biologically active clogging layer, where intensive degradation and adsorption processes occur within a short residence time; and along the main flow path between the river and abstraction borehole where degradation rates and sorption capacities are lower and mixing processes greater. Compared with SW abstraction, RBF with its natural attenuation processes has the following advantages:

- equilibration of temperature changes and concentrations of dissolved constituents in the bank filtrate;
- removal of particles, turbidity, heavy metals, biodegradable compounds, algae and cyanobacteria, pathogenic bacteria, viruses and parasites, pharmaceutical wastes and other trace organics, partial attenuation of adsorbable organic compounds, oil products and pesticides, and decline of mutagenic activity.

Given sufficient flow path length and time and sufficient filtration capacity of the riverbed and aquifer, microbial contaminants will be removed or inactivated to levels that help protect public health. The active attenuation processes in riverbank filtered porous media, even in a sub-optimal setting or time, is likely to mitigate the public health impact (Schijven et al. 2002).

Studies at RBF sites in the United States confirmed the results from studies in Europe concerning the substantial removal of natural organic matter after subsurface passage, thus reducing the potential for the formation of disinfection by-products after chlorination (Weiss et al. 2003).

However, undesirable effects of bank filtration on water quality can include increases in hardness, ammonium, dissolved iron, manganese and arsenic concentrations.

From a sustainability point of view, RBF systems make better sense than full-scale treatment plants that directly abstract SW, since in general the energy and resource use in RBF will be lower and little to no chemical residues will be produced. RBF systems commonly require less energy to operate and to deliver a unit amount of water than conventional SW treatment systems. Furthermore, RBF has socio-economic values (Tab. 1-1). In this context, the development of new RBF systems in Egypt has increased the per-capita availability of drinking water.

Tab. 1-1: Socio-economic value of bank filtration (Ray et al. 2002)

Services and benefits	Value
Contaminant removal (pathogens, chemicals)	Reduced medical costs, longer life-span, improved productivity, capital cost reduction, cancer risk reduction & enhanced environment
Reduced maintenance	Capital cost reduction
Improved reliability (as source-water)	Drought protection
Removal of nutrients	Reduced post-treatment costs, lower regulatory scrutiny & lower monitoring costs
Enhanced community supply	Increase in per capita-availability & less time spent to access / collect water

Before investigating a potential site for RBF, the purpose or need for RBF should be clearly defined for the rural or urban communities that are to be supplied with water. This includes determining local demand for drinking water and deficits in quantity and/or quality and evaluating problems with the existing drinking water source, identifying benefits of using RBF over existing source, identifying presence of suitable surface water body and approximate location of RBF site.

In general RBF is preferred for drinking water production when groundwater resources are insufficient and/or groundwater or surface water quality require increased efforts in water treatment to reach the desired quality. Thus the water supplier will usually choose from amongst the following objectives:

(1) Abstraction of a significant portion of bank filtrate (e.g. >50 % bank filtrate in the abstracted water from the well), especially if groundwater resources are limited or groundwater quality requires specific water treatment,
(2) Removal of turbidity by mechanical filtration processes in the riverbed, especially if surface water treatment is affected by periods of high river water turbidity,

(3) Removal of pathogens (bacteria, viruses, protozoa), especially if river water is polluted by domestic waste water and high dosage of disinfectants during drinking water treatment creates problems with disinfection by-product formation and bad taste of drinking water,
(4) Removal of pollutants from shock loads, e.g. oil spills from ship accidents, to assure functionality of water treatment facilities,
(5) Removal of organic micropollutants (e.g. pesticides, pharmaceuticals) by opting for long travel times or flow path lengths of bank filtrate for maximum use of naturally occurring processes in the aquifer.

Objectives (1) to (4) have been observed to be important drivers for implementation of RBF schemes in Upper Egypt. For small scale RBF schemes, objective (5) is in contradiction to objective (1). High portions of bank filtrate together with long flow paths and/or travel times can only be achieved for large scale RBF schemes (Q >20,000 m^3/d), under favorable hydrogeological conditions or by locating wells on an island in the river.

An additional, specific objective (6) in Egypt reflects the strong increase in water demand due to population growth, time constraints and the lack of available land for new surface water treatment stations:

(6) Cost-efficient and short-term increase in drinking water production from existing surface water treatment plants along the River Nile by using the available land.

Achieving the above named objectives depends on the local geomorphological, hydrogeological and hydrogeochemical conditions at the proposed RBF site. The location along the river, distance between the well and river, travel time of bank filtrate and SW and ambient GW quality are important factors that have to be considered.

1.2 Favourable hydrological and hydrogeological conditions

The first step is for the water supply organization or the planner of a new water supply scheme to take an informed decision to opt for RBF and what the benefits of its application compared to direct surface water and / or groundwater abstraction are expected to be. This decision is coupled to the availability of suitable hydrogeological conditions because the latter must be met in order for RBF to occur. Grischek et al. (2007) reviewed numerous RBF operations and site characteristics to identify key characteristics of a potentially successful RBF and concluded that:

- the site is typically located at the mid-reaches of the river,
- the location at an inner bend of a meander is an advantage,
- flow velocity of >1 m/s and a shear stress of <5 N/m^2 helps avoid clogging of the river bed,
- the thickness of the aquifer is typically >10 m,
- the hydraulic conductivity of the aquifer ranges between 10^{-2} and 10^{-4} m/s, and
- riverbed infiltration rates <0.2 $m^3/(m^2 \cdot d)$ are to be preferred in order to minimize clogging.

These parameters should be used as indicative because RBF can be used for a wide variety of conditions. On the other hand there might be conditions such as insufficient dissolved oxygen concentration available in the river water together with a high load of biodegradable organic compounds which could limit the application of RBF technique at a particular location.

Along the River Nile, sites with the following characteristics should be prevented:

- river branches with shallow depth of water and extensive plant growth near the river bank
- locations with deposition of fine sediments and organic material near to the river bank, as observed around surface water abstraction facilities.

Such conditions indicate potential problems related to riverbed clogging, e.g. low portion of abstracted bank filtrate and anoxic conditions resulting in higher concentrations of manganese in the pumped water.

2 Planning a Riverbank Filtration Scheme

2.1 Basic site assessment

A general approach for RBF site selection is shown in Fig. 2-1. Available maps of land use, watershed, river network, terrain and geology form the basis for basic site assessment. At present, geographical information systems (GIS) and different scoring schemes are under development to support RBF site selection, e.g. in China and India. Generalization of such approaches is difficult, mainly because of different site conditions and the complexity of hydrological, hydrogeological and hydrochemical factors, different aims and economical limitations of water supply companies. Finding the optimal solution for a water supply system is not a straight-forward process and cannot be automated. Nevertheless, available hydrogeological maps, piezometric contour maps, river hydrographs and river cross-sections should be searched at different institutions.

As most RBF sites in Egypt will be located along the River Nile and objective (6) applies, stage 1 of the planning process can be kept short and easily copied to neighboring sites.

At stage 2 – Visual reconnaissance – potential space at the waterworks property and the availability of additional land to construct RBF wells outside the existing property has to be checked. Such restrictions strongly determine the site assessment procedure. During the visual reconnaissance, favorable conditions for RBF are indicated by the presence of alluvial deposits and topographically relatively level land. Other factors to take into account are the presence of any existing wells near or on the riverbank which may eventually already abstract some bank filtrate or can otherwise be used for monitoring water quality, landside wells to determine the quality of ambient groundwater, points of discharge of waste water into the rivers as a RBF site should ideally be located upstream and site-access.

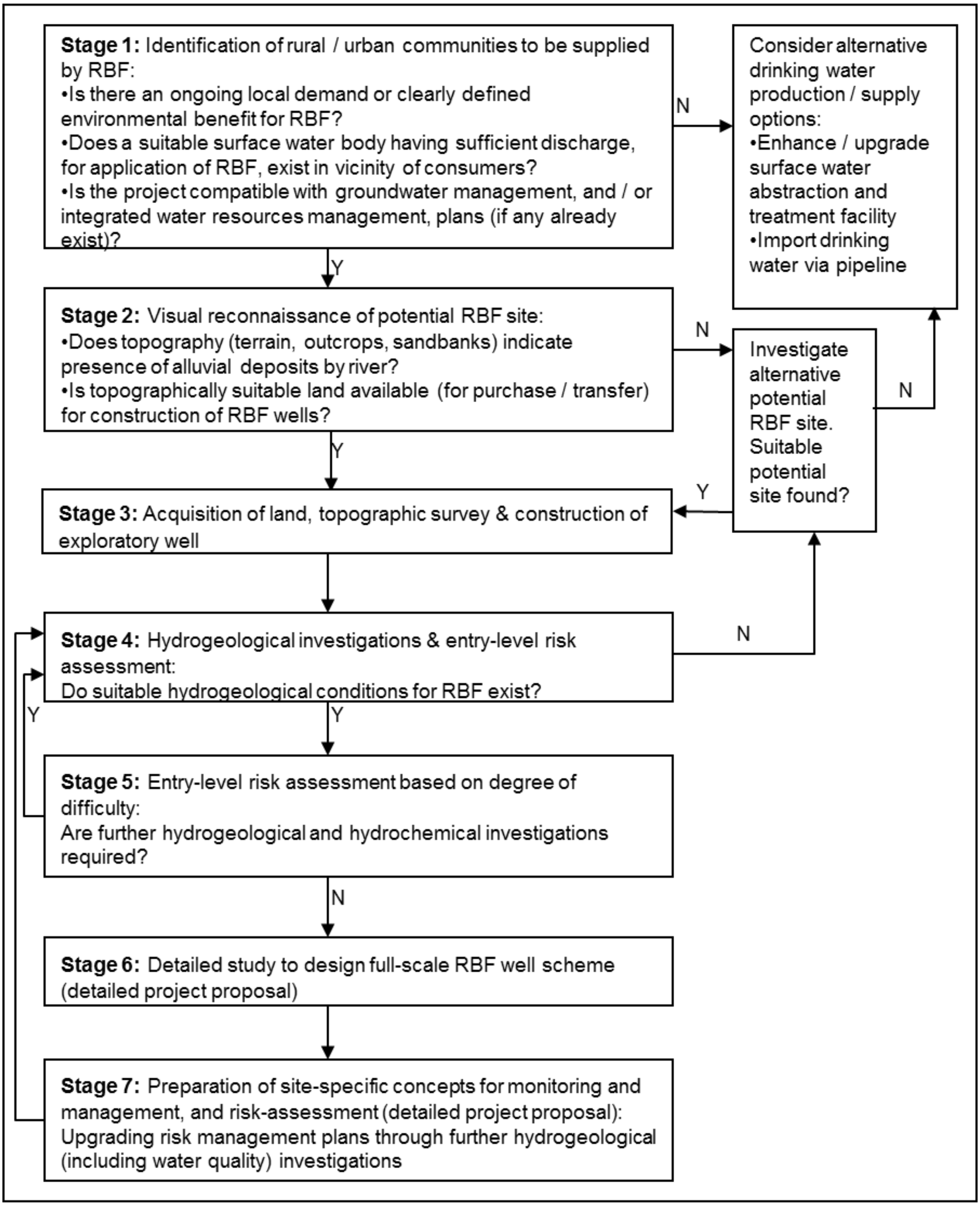

Fig. 2-1: Concept to plan RBF schemes and basic site assessment (Sandhu 2015)

Stage 3 – Topographical survey and construction of exploratory well(s) – is of major importance for RBF site selection in Egypt. Groundwater in Egypt often has high manganese and/or iron concentration, thus objective (1) is followed to get a high portion of bank filtrate to prevent post-treatment of pumped water (removal of Mn and Fe), which differs from common treatment steps (coagulation, filtration) for surface water treatment at the sites. Also, at many sites along the River Nile, a strong groundwater flow gradient towards the river is observed, often due to irrigation in the agricultural area along the river. As a result, RBF wells have to be located near to the riverbank, in some cases outside the property of the water company. But short travel times of bank filtrate limit the efficiency of attenuation processes to remove pollutants. To find the optimal location to meet more than objective (1), reliable determination of water levels of river and groundwater and of the groundwater flow gradient are preconditions. Thus, at most sites installation of at least one groundwater observation well is required to allow measurement of the groundwater level and comparison with river water level. Such installation of a GW observation well should be done preferably before constructing a (deeper) exploratory well. If the groundwater flow gradient is too high, RBF may not be feasible and drilling an exploratory well may not be required. The exploratory well should be drilled at a potential location of the RBF wells and could be even transformed into a fully equipped RBF well later.

Surveillance

If no geodetic information is available at a site, geodetic measurements have to be carried out, especially focusing on the location of existing and planned wells and the riverbank. Distances between the riverbank and the wells can be determined using a measuring tape. Water level measurement points on each well should be set and at each site a reference height (datum) defined to standardize water level measurements. Well elevations as well as river water levels have to be determined using a levelling instrument and staff. As commonly there are no observations wells that are not affected by the production wells, it is difficult to measure GW levels. If existing wells are not operated at night, GW levels should be measured in the early morning before pumps are switched on again. Decommissioned wells can be used to drain off a GW level and, in a second step, calculate a GW gradient. The well infrastructure should be documented. Water meter, flow rate und pressure data have to be collected for existing wells. Not all existing wells are equipped with a water meter, inhibiting discharge calculation of the wells. Long-term water level measurements are preferred and can be recorded via data loggers.

River channel geometry

An echo sounder could be used for electroacoustic measurement of river water depths. Water levels should be read with their corresponding coordinates and imported into a GIS software. A contour map should be prepared and transferred into a channel-bed profile by means of a river cross-section. The derived river channel geometry is required to prove hydraulic connection with the aquifer. It is also required for groundwater flow modeling.

Construction of an exploratory well

The exploratory well forms a basis for stage 4 – Hydrogeological investigation – and has to be drilled without drilling liquids to get sediment samples from different depths of the aquifer. In case that the confining layer is very deep, drilling could be stopped at about 50 mbgl. The determination of the thickness of the typical clay layer on top of the aquifer is of first priority. If the lower boundary of the clay layer is found at a lower level than the riverbed, there may be no hydraulic connection between the river and the upper aquifer, making RBF not feasible. Getting sediment samples from greater depths for grain size analysis and estimating hydraulic conductivities (K-values) is of second priority. Using the exploratory well for pumping tests is of third priority and only necessary, if there is a high chance of hydraulic connection between the river and the aquifer.

Stage 4 may also include application of non-invasive techniques, e.g. geophysical tools such as seismics, geoelectrics, self-potential, electromagnetics, ground penetration radar, gravity and magnetics. These can be applied to obtain information on the structural settings of the subsurface geology, hydrogeological settings and dynamic processes (Tab. 2-1; Schütze & Flechsig 2014). However, an optimal and reliable interpretation of data is more likely attainable using a multi-parameter approach by applying a combination of methods. Therefore, the criteria for selection of a specific method depends on the expected physical contrasts between the measured parameters, the spatial resolution of the method and organizational and logistical factors such as duration of the geophysical measurements, area to be investigated and cost and manpower involved. The cost, manpower and time involved for all types of non-invasive techniques are important to consider during investigations of a potential RBF site because while each method offers certain advantages, these can only be performed by experienced personnel. Furthermore the information obtained from these techniques must be supported with information from at least one invasive technique, ideally from an exploratory well.

Tab. 2-1: Geophysical tools for the exploration of potential RBF sites (Schütze & Flechsig 2014)

Method	**Parameter measured (symbol)**	**Indicator for**
seismics	velocity (v)	mineral content, porosity, density
geoelectrics	resistivity (ρ)	porosity, water content, salinity of water
self-potential	natural potentials (U)	fluid flow, temperature gradients, ion concentrations
electromagnetics	conductivity (σ), permittivity (ε)	porosity, water content, salinity of water
ground penetration radar	permittivity (ε), conductivity (σ)	porosity, water content, salinity of fluids (water)
gravity	density (ρ)	density of minerals, porosity
magnetics	magnetic permeability (κ)	content of magnetic minerals (e.g. magnetite)

2.2 Detailed site assessment

Detailed site investigation is a very important parameter in the design process. The purpose is to identify the key design parameters and to identify any potential complications that could impede the short and/or long term performance of the bank filtration schemes. Fig. 2-2 highlights major steps and parameters.

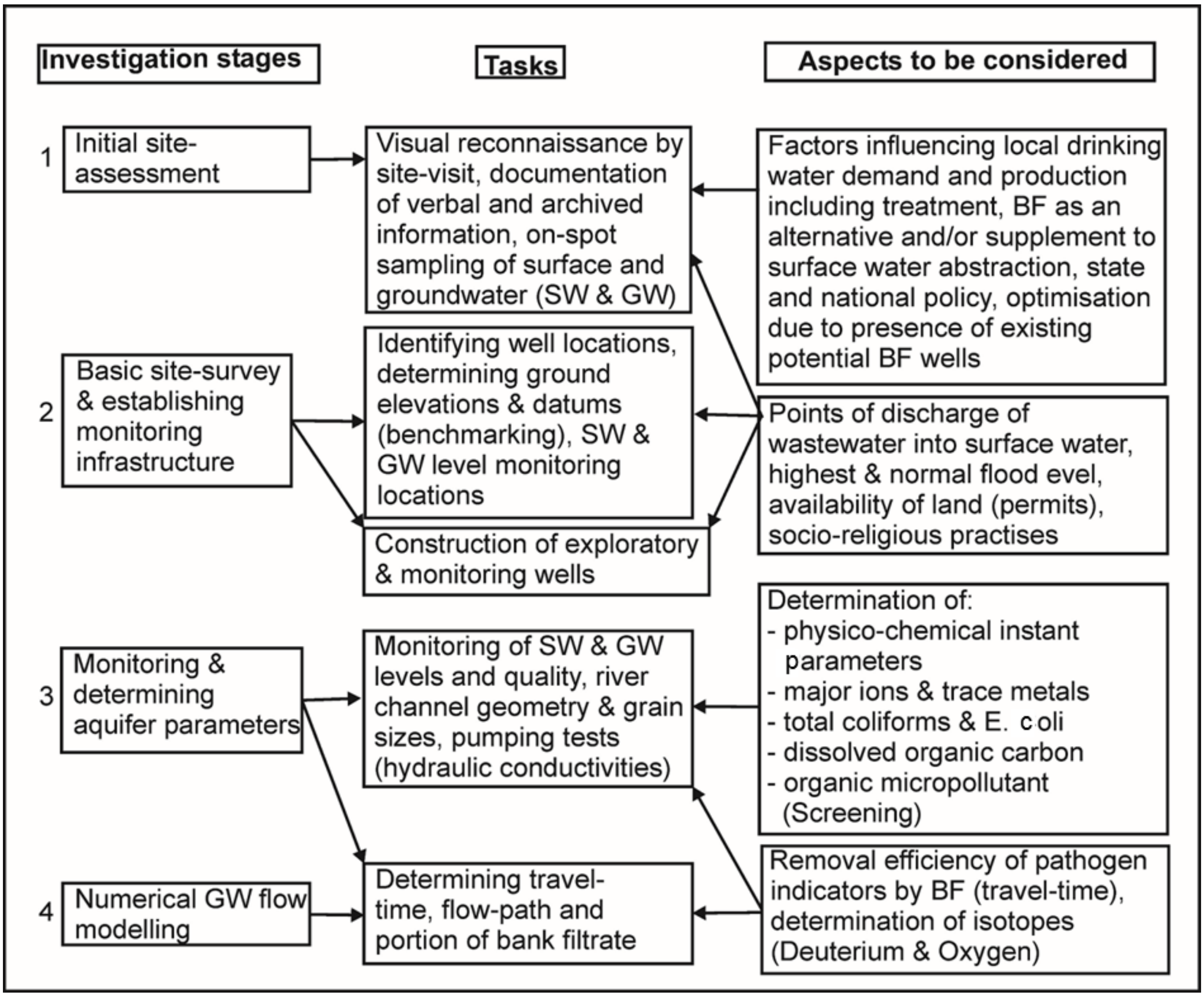

Fig. 2-2: Overview of methodology for the detailed investigation of a RBF site (Sandhu 2015)

Hydrogeological testing should be used to determine the governing properties of the aquifers, aquitards and aquicludes identified in the geological strata graph report and also include an estimation of saturated aquifer thickness (M), transmissivity (*T*), storativity (*S*) and hydraulic conductivity (*K*). Tab. 2-2 presents the various methods available to determine aquifer parameters.

Tab. 2-2: Methods to determine aquifer parameters

Category	Test	Details
On site	Well pumping test	Large mass/volume estimation
On site	Borehole	Results limited to locality of bore
Visual Assessment	Classification	Can give approximate guide

The determination of the aquifer matrix permeability is of significant importance and is also crucial for the calculation of the RBF systems safe yield. The following points should be considered before conducting any hydrogeological testing:

- The ground is likely to be heterogeneous and anisotropic, which prevents a definitive permeability value from being obtained. A conservative approach should be adopted to account for the variability in the ground conditions.
- Hydraulic conductivity may be anisotropic i.e. horizontal (K_h) is greater than the vertical conductivity (K_v).
- Hydraulic conductivity is not only dependent on the soil's description and grading, but also on fissuring and layering, which can cover an extensive area. Laboratory testing of borehole material should only be used if well pumping tests, which provide a better estimate of localized transmissivity, cannot be conducted.

During borehole drilling an estimate of the hydraulic conductivity (K) can be obtained by sieving the borehole material. Borehole material should be sampled every 1 m during drilling until final depth.

The following points contain recommendations for performing a sieve analysis based on European standards (DIN EN ISO 17892).

- A representative test sample shall be prepared by sample division.
- After division, the sample should meet the requirements of Tab. 2-3. The minimum mass of the sample therefore depends on the largest diameter of soil particles, whereby individual coarser components are not considered.

Tab. 2-3: Recommended minimum masses for screening, DIN EN ISO 17892:5.2.2.2

Largest particle diameter [mm]	Recommended minimum masses [g]
< 2.0	100
2.0	100
6.3	300
10	500
20	2000
37.5	14000
63	40000

- It is recommended to use screens of the sizes 63 mm, 20 mm, 6.3 mm, 2.0 mm, 0.63 mm, 0.2 mm and 0.063 mm, as these sizes are the limit sizes for coarse materials according to EN 14688-1. They allow the description and classification of samples.
- If more than 10% of the grains in the sample are smaller than 0.063 mm, a sedimentation test is recommended.
- For samples with coarse gravel or boulders, an initial separation on a suitable sieve may be necessary to avoid overloading the subsequent smaller sieves. A 20 mm sieve is recommended for this first separation, as this corresponds to the boundary between medium and coarse gravel. Any sieve residue which is smaller than the separation sieve (20 mm) shall be added to and mixed with the original sieve pass.

- For most soils, the samples should be shaken for at least 10 min if a mechanical sieve shaker is used. If manual sieving is used, the sieves should be shaken for at least 2 min. Sieving shall be considered complete when an additional sieving of at least 1 min does not result in a change in mass of the sieve residue of more than 1% of the mass.
- If the sum of all sieve residues differs from the total mass at the start of sieving by more than 1%, sieving shall be repeated.
- Test results should be presented in a semi-logarithmic representation. The test results may also be given in tabular form as grain size and percentage of passing through the sieve, rounded to 1%.

To estimate K, equation 2-1 after Beyer and Schweiger (1969) should be preferred to Hazen (1893) because it was developed for alluvial sediments. It accounts for the grain-size diameter corresponding to 10% cumulative undersize particle distribution and thereby takes into account the portion of finer aquifer material which affects K given in m/s.

$$K = C_B \cdot d_{10}^2 \tag{2-1}$$

where C_B is the coefficient for the natural state of compaction of the sediment after Beyer & Schweiger (1969) and d_{10} is the grain-size diameter in mm corresponding to 10% cumulative undersize particle distribution.

Calculation of K based on eq. 2-1 is valid for $2 \cdot 10^{-5}$ m/s < K < $4 \cdot 10^{-3}$ m/s, 1 < C_U < 20 and for 0.06 mm < d_{10} < 0.6 mm. Depending on the natural state of compaction (packing) of the sediments, C_B can be calculated using eq. 2-3 to 2-5 (Houben & Bluemel 2017). In this context, C_U is defined as granulometric non-uniformity coefficient which is the ratio of d_{60} to d_{10} (eq. 2-2).

Non-uniformity coefficient: $C_U = d_{60}/d_{10}$ (2-2)

Loosely compacted sediment: $C_B = 14797 \cdot C_U^{-0.147}$ (2-3)

Medium compacted sediment: $C_B = 11864 \cdot C_U^{-0.200}$ (2-4)

Densely compacted sediment: $C_B = 10041 \cdot C_U^{-0.232}$ (2-5)

Unless K has not been obtained from interpretation of pumping tests conducted on other wells nearby, the K derived from the grain-size distribution analyses can be used to obtain an initial estimate for Q_A and Q_F. Else pumping test data analysis provides the most representative K. Depending on the boundary conditions prevailing at the site, an appropriate method can be used to determine K from pumping test data. As these methods are well known, published in standard literature and are available in the internet or as numerical tools such as AQTESOLV, these are not discussed further.

Pumping tests

To determine hydraulic properties of the aquifer, pumping tests should be conducted. Due to the necessity of water supply at many sites, it may be not possible to switch off all pumps at a site in advance to (to re-establish natural flow condition) and during the pumping test. Furthermore, at most sites observation wells are missing. In such case, already existing pumping wells can be used for water level monitoring. Drawdown and recovery should be monitored and recorded with automatic pressure loggers. For control purposes additional manual measurements have to be conducted with an electric contact meter. All data should be evaluated in a table processing program and diagrams of the water levels created to evaluate the water level changes when neighboring wells are shut down or put back into operation. In addition, pumping test analysis tools, such as AQTESOLV, can be used for curve matching to estimate hydraulic properties. Drawdown data can be used to determine the transmissivity T [m^2/s]. The hydraulic conductivity K [m/s] then can be obtained from eq. 2-6.

$$K = \frac{T}{M} \tag{2-6}$$

with M = saturated aquifer thickness [m].

Well location

It is advised to design well galleries with more than two wells parallel to the riverbank as the portion of bank filtrate increases with the number of wells. This has the advantage that an intermittent failure of one of the wells does not much influence the flow conditions and thus does not change the redox conditions. The favorable hydraulic conductivity in the River Nile valley aquifer enables wells to be placed at a very small spacing of 10 to 30 m relative to each other (Bartak et al. 2015). Modeling results show that despite a distance between the wells of almost 50 m, the portion of the bank filtrate hardly changes. Instead, the flow times of the central wells of a well gallery even accelerate. The shorter the distance from the wells to the river the higher is the gradient from the river to the wells. A travel distance of around 10 m from the River Nile to the pumping wells was found to be sufficient to remove bacteria (Wahaab et al. 2019). Modeling and field test results show that there is a considerable influence of the groundwater flow gradient on the water quality of the RBF well. Thus, it is highly recommended to determine the value reliably. Sites with a high gradient should be designed with wells close to the bank or/and inside a meander bend.

Groundwater flow modeling using MODFLOW or any other software is recommended to identify optimum well location and to determine travel times and potential portions of bank filtrate in pumped water depending on pumping rates. Based on experiences from modeling studies in Egypt (e.g. Bartak et al. 20, Paufler et al. 20), a table of required input parameters has been prepared (Tab. 2-4). Whereas some parameters (e.g. effective porosity) could be estimated, water levels have to be determined on-site. If all input data are provided to any modeler, the set-up of a groundwater flow model will be easy and allow to simulate different scenarios and to run sensitivity analyses for selected parameters. A template (Excel spreadsheet) has been made available to HCWW to support preparation of input data sheets.

Tab. 2-4: Required input data for groundwater flow modeling

Input Parameter	Unit
Name of the Site	**-**
Date of exploration	**-**
Width of area available	m
Length of area available	m
Thickness of the top layer	m
Thickness of the aquifer	m
K-value of the aquifer	m/s
Effective porosity of the aquifer	-
Width of the river	m
Depth of the river	m
Ground surface level	m asl
Landside groundwater level	m asl
at a distance of … m from the riverbank	m
Groundwater slope	‰
River water level	m asl
Description of the riverbed material	-
Planned abstraction volume of all wells per day	m^3
Number of filter screen sections available	-
Length of 1 filter screen section	m

3 Construction and testing of a RBF scheme

3.1 Well design

The type of the well depends on the amount of water to be abstracted (well discharge), site-specific geology and planned travel time of the bank filtrate to the well. For small scale RBF schemes, vertical wells are preferred. Horizontal collector wells are feasible for large scale RBF schemes with high abstraction rates or aquifers of low thickness. Experiences from existing RBF schemes in Upper Egypt prove that vertical wells can deliver the required amounts at the given hydrogeological conditions. Horizontal collector wells are very expensive and require specialized drilling companies which are not available within Egypt. Installation of horizontal drain pipes along the riverbank by digging and connection to a collector well is also not feasible at most sites because of steep river bank morphology, limitations to get access with heavy machines and limited availability of land. Large-diameter (~ 10 m) caisson wells are used for some RBF systems designed to meet high water demands in areas with shallow groundwater tables (≤3 m below ground level) having medium to coarse alluvium containing cobbles and boulders which make installation of vertical wells difficult. As large boulders have not been found during drilling works along the River Nile and the waterworks properties are mainly located more than 3 m above the lowest river water level, there is no need for construction of such wells. Horizontal collector wells and horizontal drain pipes would be special constructions at only few sites in Egypt in the future and require detailed design studies. In Egypt, construction of vertical wells is economical and fast compared to the other well types. Thus, well design will be only discussed here for vertical wells.

To design a well, information on the grain-size diameter of the aquifer and riverbed material, the hydraulic conductivity (K), aquifer capacity (Q_A), well yield (Q_F), groundwater level at rest above aquifer base (H), water level in the well at steady-state drawdown conditions (h), radius of influence or cone of depression (R) and borehole radius (r_0) are mainly required. Once the hydraulic conductivity is known the aquifer capacity and well yield can be calculated.

Aquifer capacity and well yield

Aquifer capacity and well yield depend on the aquifer conditions, thus unconfined and confined aquifers have to be distinguished (Fig. 3-1).

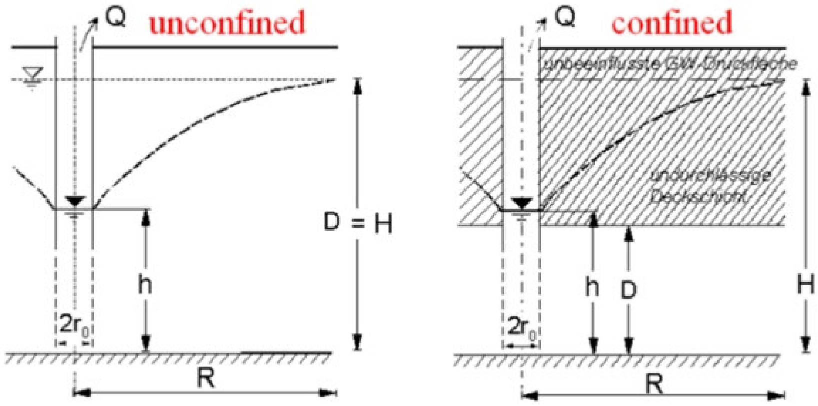

Fig. 3-1: Effect of pumping from an unconfined and confined aquifer

The maximum extractable safe well yield (discharge) from an aquifer is calculated after the hydraulic conductivity has been determined. By plotting the steady state flow rate towards an abstraction well or the aquifer capacity (Q_A) [L^3/T] by eq. 3-1 and 3-3 and the yield of the abstraction well (Q_F) [L^3/T] given by eq. 3-2 and 3-4, versus the water level in the well, the maximum extractable yield (discharge) for a well is determined at the intersection of Q_A and Q_F. Q_F should be greater than Q_A to account for well aging during the course of operation of the well.

Unconfined aquifer

$$Q_A = \frac{\pi \cdot K \cdot (H^2 - h^2)}{\ln R - \ln r_0} \quad (3\text{-}1)$$

$$Q_F = \frac{2}{15} \cdot \pi \cdot r_0 \cdot h \cdot \sqrt{K} \quad (3\text{-}2)$$

Confined aquifer

$$Q_A = \frac{\pi \cdot K \cdot (H^2 - h^2)}{\ln R - \ln r_0} \quad (3\text{-}3)$$

$$Q_F = \frac{2}{15} \cdot \pi \cdot r_0 \cdot h \cdot \sqrt{K} \quad (3\text{-}4)$$

where K is the hydraulic conductivity of the aquifer [L/T], H is the pre-test rest water level measured from the aquifer base (before pumping) [L], h is the steady-state water level after a constant drawdown is obtained (during pumping) [L], R is the radius of influence (cone of depression or drawdown) of the well at a constant drawdown [L], and r_0 is the radius of the well-bore [L].

This approach is documented in many textbooks, neglecting the practical aspect of placing the pump not within the filter screen. As the pump has to be placed at least 1 m below the lowest water level during pumping and >1 m above the filter screen, the determination of Q_A and Q_F has to be adjusted according to Fig. 3-2.

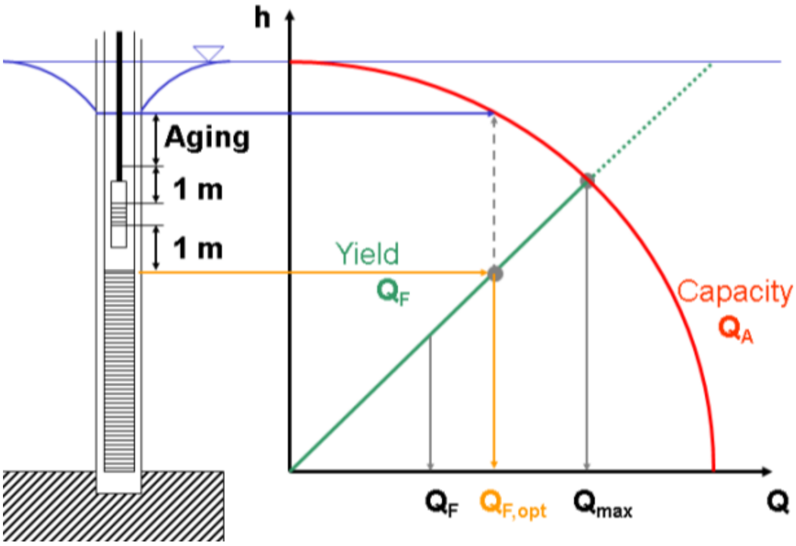

Fig. 3-2: Aquifer capacity and well yield if the pump is placed in the upper part of the well (Grischek 2010)

However in some cases when there is an uncertainty in the hydraulic conductivity, it is beneficial to consider different scenarios ("worst" and "best" cases) ranging from lower to higher hydraulic conductivities. To illustrate this, an example is provided for the design of an RBF well (Fig. 3-3). The calculations took into account field data and are based on surface and groundwater levels measured during low flow conditions.

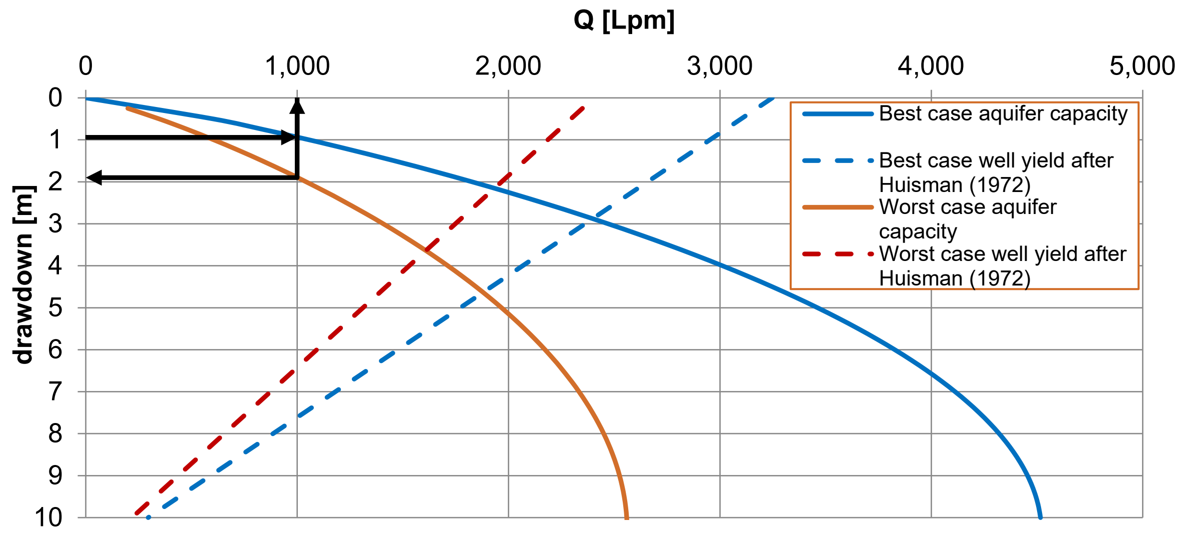

Fig. 3-3: Calculated well yield vs. well capacity (yield incl. 75% safety)

The “worst case” assumed $K = 4 \cdot 10^{-3}$ m/s, whereas the “best case” is based on $K = 7.3 \cdot 10^{-3}$ m/s. Both K lie within the range of literature values for medium gravel. Well capacity was calculated according to Huisman (1972), wherein the maximum well borehole entry velocity of Sichardt (1928) is reduced by 50% to account for well aging. The theoretical optimum well discharge (intersection of Q_A and Q_F) is in the range of 1,580 to 2,360 l/min. However, to take into account for strong well aging due to precipitation of Fe- and Mn-(hydr)oxides as a result of mixing of bank filtrate with landside groundwater with higher Fe- and Mn-concentration (commonly the case in Egypt), it is recommended to use state-of-the-art filter screens, ensure a proper gravel pack and not to exceed a maximum discharge of 1,000 l/min (60 m^3/h).

Design of vertical well filter media and well filter screens

This section is based primarily on standard guidelines and literature used in Germany (DVGW 2001, 2005, DIN 2014). However national guidelines and literature should also be considered where applicable. Vertical wells should use a central screen surrounded by a filter pack. The shaft is dug or drilled to a much greater diameter than that of the screen, say 550 mm for a 300 mm screen, and a temporary liner such as a large steel casing pipe is inserted to stop the surrounding borehole material from collapsing. The screen is put in place, the filter material is inserted in the gap and the liner is removed. This technique enables a good thickness of filter material but is more suited to shallower wells of up to 25 m depth. Following this, the well is usually surged to pull small sand through the screen; this process is called well development.

The filter sand or gravel pack must prevent sand and other soil or suspended particles from being washed into the well. The filter pack should also ensure a high water permeability in the entrance area and its vicinity because the highest hydraulic gradients occur here. Therefore a filter pack has to guarantee both the development of the well and a sand-free abstraction of water. For this the grain-size of the filter pack has to be carefully calculated.

The filter media should be natural sand or gravel having almost spherical grains with a smooth surface. Artificially made fragments (e.g. grit) are not recommended. The filter media should consist of pure quartz (approx. 96% SiO_2 by weight), with the content of clay, lime, mica, feldspar and other components not exceeding 4% total and organic material not exceeding 0.5%. The over- and under-size composition should be <10%. Good filter material usually has a C_U of 1.5 or lower (eq. 2-2).

The density of packing of the grains of the filter gravel is important concerning the passing of small grains through the interspaces between larger grains. With the theoretically loosest packing, the diameter of the large grains d_1 is 2.41 times greater than the largest diameter d_2 for a round grain between them (Fig. 3-4 left). With the theoretically most compact packing density, d_1 is 6.46 times greater than the largest passage d_2 (Fig. 3-4, right). In practice, the real density of packing ranges between the theoretically loosest and densest packing.

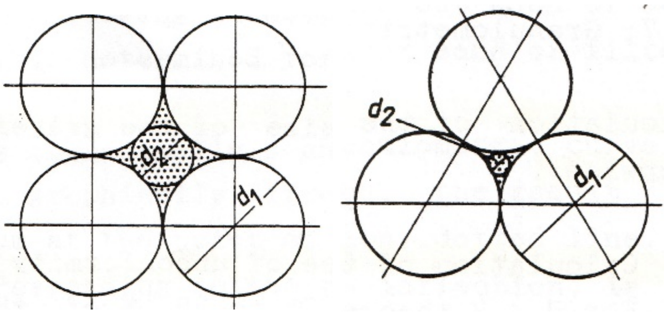

Fig. 3-4: Loose packing density (left) and compact packing density (right) (Balke 1999)

Therefore, the ratio of the largest passage to the grain size, called the screening factor, lies between these two limits and is around 4 to 5. For the example of the granulometric curve provided in Fig. 3-5, the screening factor of 4.5 is given. In practice the aquifer material does not have a uniform grain size, the degree of non-uniformity C_U is of great importance (eq. 2-2; Fig. 3-5).

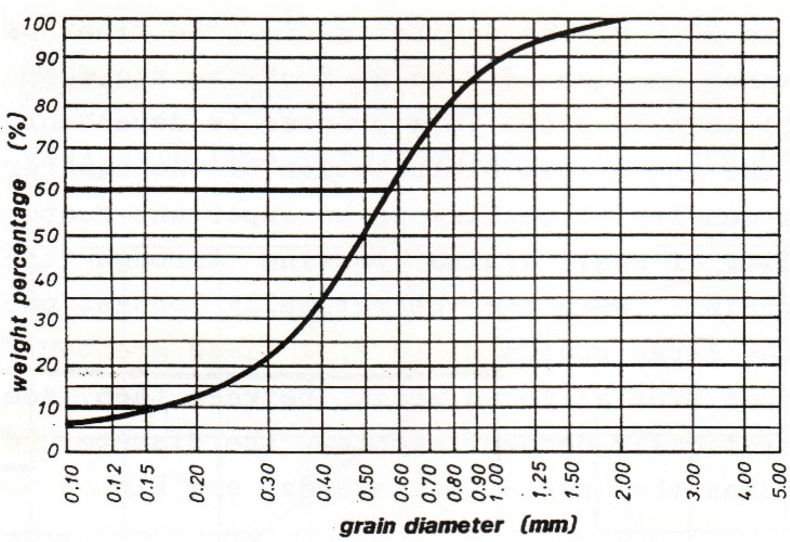

Fig. 3-5: Granulometric (grain size distribution) curve showing grain size diameters corresponding to 10% and 60% passing (Balke 1999)

Determination of grain size of filter pack

For C_U < 5, the grain size of the gravel pack is calculated as follows:

1. For C_U < 3, theoretically 75–85% of the rock in place are supposed to go through the interspaces of the filter grain; for 3 < C_U < 5, the theoretical removal is 85–95%.
2. Calculate the "removal grain-size" or relevant grain size from the granulometric curve (example of Fig. 3-5). Look up the 75–85% and 85–95% intercepts on the Y-axis of Fig. 3-5 and corresponding grain size diameter (removal grain size or relevant grain size) on the X-axis.
3. Multiply the "removal grain size" by the screening factor 4.5. The result is the grain size of the gravel pack.
4. It should be checked if the planned abstraction rate of the well can cause buoyancy of the gravel pack. If so, the grain size for a second gravel pack, which is placed above the first one, has to be calculated.

For C_U > 5, the portion with the coarsest grain size of the borehole material (sample) has to be removed and thereafter another granulometric curve has to be evaluated by sieving. This procedure must be repeated until $C_U \leq 5$.

Another method, that of Nahrgang & Schweizer (1982), can also be used to calculate the grain size of the filter gravel pack (eq. 3-5). Therein, the grain size that theoretically allows an out-wash of 50% of the aquifer material (D_{50}) is determined for the filter sand or filter gravel.

$$D_{50} = d_g \times F_g \qquad (3\text{-}5)$$

Where, D_{50} is the grain size of the filter sand or gravel in mm, d_g is the relevant grain size of the sediment in mm and F_g is the screening factor.

To determine d_g, the normally s-shaped granulometric curve is graphically differentiated (Fig. 3-6). The result is a curve with a maximum at the point of inflection (Fig. 3-6, dashed / broken line). The grain diameter corresponding to the point of inflection, is the relevant grain size of the sediment (d_g), which is often around D_{50} in magnitude. In case of mixed grain sediment no point of inflection may occur and in such cases d_g = d_{30}. If there are two points of inflection, it is recommended to select the corresponding grain size of the smaller value.

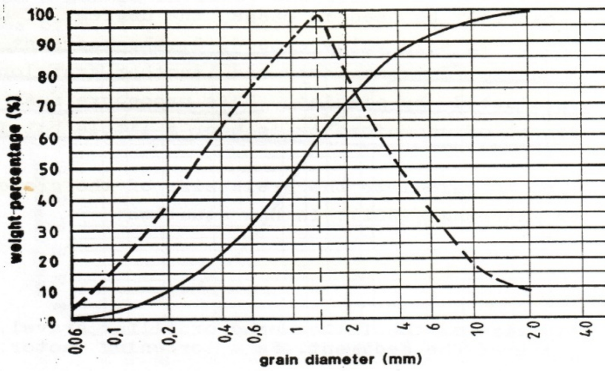

Fig. 3-6: Example of a granulometric curve for the determination of d_g using the methodology of Balke (1999) and Nahrgang & Schweizer (1982)

The screening factor F_g can be calculated, as given by eq. 3-6 and 3-7, for

1 < C_U < 5, F_g = 6 + C_U (3-6)

$C_U \geq 5$, F_g = 11 (3-7)

The basis for this is also the granulometric curve of the aquifer sediment. Several calculations to determine the grain size of the filter pack using both methods, have shown to give comparable results (Balke 1999). In practice, sand and/or gravel with a diameter between 0.71 mm and 16 mm is mainly used for the filter pack.

Well filter screen design

If the calculations for the aquifer capacity Q_A and well yield Q_F imply that the aquifer should be exploited as deep as possible, the well screen (filter section) should cover all water-bearing layers, with casing section covering layers of low permeability (silt, clay) or at the location (depth) of the pump (in case it is installed approx. at mid-depth). Furthermore, even at maximum drawdown during pumping (including during dry season), the water level in the well should not reach the perforations of the screen. Hence the upper edge of the screen has to be installed at least about 1 m or more meters below the deepest expected water level. The length of the screen (l_s) of a well can be calculated by eq. 3-8.

$$l_s = d_b - (d_l + 1\text{ m above pump} + l_p + 1\text{ m blind pipe below pump entrance} + l_{sump}) \qquad (3\text{-}8)$$

where, d_b is the depth of the well bore [L], d_l is the greatest depth of the water level [L], l_p is the length of the pump [L] if not placed in the sump pipe, l_{sump} is the length of the sump pipe [L], if not placed in the confining layer.

The diameter of the screen is determined by the diameter of the pump and the additional equipment, as well as by the permissible entrance velocity of the groundwater flowing into the well. In case that the pump is placed within the sump pipe, then all the pipes above the sump pipe must be wide enough to allow the pump and additional devices to be lowered without endangering the casing and the screens.

The velocity of groundwater entering a well (entrance velocity) has an important influence on the life-span of the well, especially the screens. Normally, groundwater flow in an aquifer is laminar. But when approaching a well, the flow conditions in the aquifer or in the filter pack around the screens change to turbulent, if the water exceeds a certain flow velocity. This change is disadvantageous because turbulence favors the influx of fine particles and iron hydroxide deposition, both of which result in well-clogging. Therefore the screens must have a sufficient entrance area to avoid a turbulent inflow of water. The entrance area varies with the diameter of the screen pipe and width of the openings.

The flow conditions in the pore spaces can be determined with the help of the Reynolds number (3-9)

$$Re = \frac{v_f \times d_{wk}}{v} \qquad (3\text{-}9)$$

where, Re is the Reynolds number [-], v_f is the filter velocity in m/s, d_{wk} is the effective grain size diameter in m, ? is the kinematic viscosity for water in m^2/s.

Turbulence occurs when a certain filter velocity is exceeded. For sand and gravel, the limiting Re value for laminar flow is 10, turbulent flow is 300 and a transition between R_e 10 and 300 (Balke 1999). However, it is controversial whether the transition from laminar to turbulent flow is at Re = 10, or below (approx. 6 to 2) due to influences of grain roughness (Balke 1999).

The mean filter velocity (v_f) at the borehole well can be calculated by the empirical formula after *Sichardt* (1928) based on the hydraulic conductivity of the aquifer (K) as in eq. 3-10.

$$v_f = \sqrt{\frac{K}{15}} \qquad (3\text{-}10)$$

The washing-out of fine sediment from the aquifer into the borehole is generally expected for $C_U > 6$ and at critical filter velocities ($v_{crit.}$) of 0.002–0.003 m/s (Balke et al. 2000, DVGW 2005). Therefore the groundwater velocity should not exceed 0.002–0.003 m/s at the time of leaving the filter media and entering the well screen. The permissible effective filter velocity in the filter pack and the permissible flow rate through a screen (DN 600) is given in Tab. 3-1.

Tab. 3-1: Grain size, effective velocity and flow rate for a screen diameter of 600 mm, Reynolds number 10 and water temperature 9.5 °C (Bieske & Wandt 1977)

Grain size of filter pack [mm]	Permissible filter velocity [m/s]	Permissible flow rate per 1 m flow length [m³/h]
3	0.0044	29.78
4	0.0033	22.46
6	0.0022	14.89
8	0.0016	11.23
12	0.0011	7.44
16	0.00083	5.62
25	0.00053	3.59
35	0.00038	2.57

Higher filter velocities of 0.03 m/s are given by other authors (e.g. Smith 1963, Driscoll 1986, Tholen 1997), with the US Environmental Protection Agency recommending filter velocities in a similar range of magnitude of up to 0.01 to 0.03 m/s (EPA 1975). Although the differences in filter velocities appear high, these are based upon different boundary surfaces and borehole diameters.

3.2 Well construction

A drilled well, situated in unconsolidated sediments, consist of a sump pipe, screen pipes and casings (Fig. 3-7). The screen pipes should be surrounded by a filter pack, whose grain diameter is chosen according to the granulometric conditions of the sediments of the aquifer as shown in chapter 3.1. At the surface, the well must be protected by a sealing out of clay or concrete.

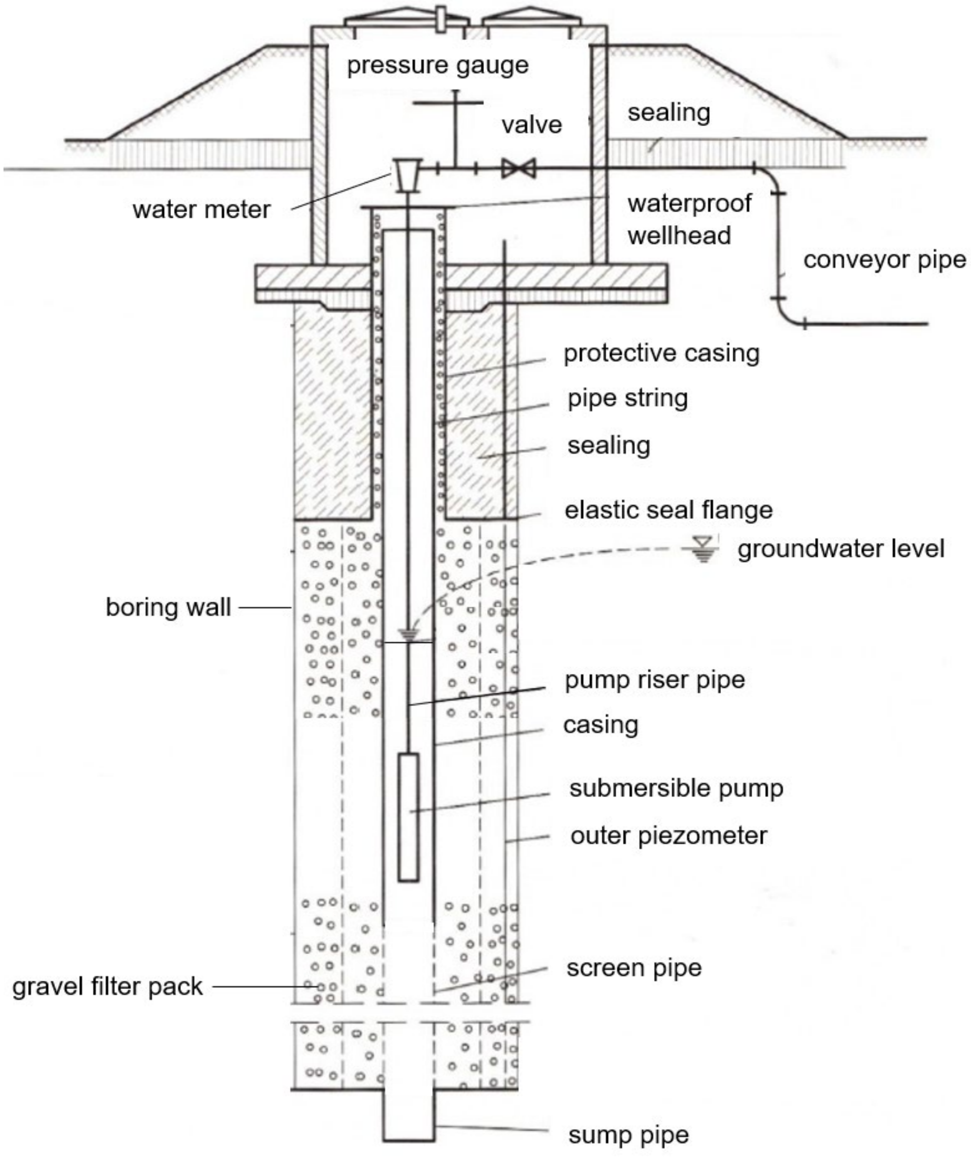

Fig. 3-7: Basic scheme of a vertical well

The following sections on well construction and development are extracted from Balke (1999).

The casing consists of pipes without perforations, installed between and above screens in positions where no water is caught. They are made of metals or plastic. Casing pipes of metal, commonly steel or stainless steel, are usually produced with diameters from 80 to 1200 mm, a wall thickness from 3 to 10 mm, and lengths from 1 to 6 m. For practical application, a length of 5 m is advantageous, since they can be handled well and, compared with the use of shorter pipes, need less connections, thereby reducing costs.

There are different possibilities to connect the pipes with each other. Thread connections are often used, as they can be handled easily and quickly. Very heavy pipes and those with a large diameter are joined with flange connections. Also welding of pipes is possible. Additional information on the properties of casing such as the weight per meter, the critical external pressure, the maximal installation weight, etc. can be received from the manufacturers. The main advantage of a metal casing is its strength, the main disadvantage is the risk of corrosion. Advantages of casing pipes made of plastics, e.g. polyvinylchloride (PVC), are the light weight and the resistance against corrosion. The installation of plastic casings is limited by temperature and external pressure. Pipes made of plastics can be used at temperatures up to approx. 40 °C, chlorine-treated PVC pipes up to approx. 60 °C. Plastic pipes commonly have diameters from 35 to 600 mm, wall thicknesses of 3 to 24 mm, and lengths from 1 to 6 m. To connect the pipes only thread connections are used with different kinds of thread types. At RBF sites in Egypt, plastic pipes are feasible but should be protected from sunlight at the well head.

A sump pipe is a pipe without perforations and closed at its lower end by a plate. With metal pipes, the bottom often is welded or fixed by a thread or a blind flange. Plastic pipes are closed by a threaded bottom plate. In any case, the connection must be tight to avoid the influx of fine particles during the pumping, therefore the installation of a rubber seal, if possible, is recommended.

Screen pipes are pipes with perforations. The desirable features of well screens are:

- to prevent sand entrance after the development of the well,
- to have screen openings which do not plug.
- to guarantee a favourable entrance velocity of the water,
- to have little filter resistance,
- to have regular distributed screen openings,
- to resist pressure and tensile forces,
- to be corrosion resistant,
- to hinder incrustations.

A large open area lowers the entrance velocity of the water flowing into the well. That reduces the risk of fine grain elutriation and incrustation. The critical velocity is described in chapter 3.1 and should not be exceeded. Plastic screens are produced with slots parallel or rectangular to the axis of the pipe. The slot size varies between 0.2 mm and 4 mm, depending on the grain size of the filter gravel pack.

Installation of casing and screens

After pulling up the drilling string it has to be checked by plumbing, whether the boring has the planned depth. If necessary, the bottom of the borehole must be cleaned from sedimented material (e.g. with the help of a valve auger). To guarantee a sufficient fall velocity of the finer particles through the water column when placing the filter pack, the drilling mud has to be diluted as much as possible without endangering the borehole wall. If the particles are not able to sink down in a viscous drilling mud, the annular space can be plugged in its upper part and high pressure on the screens is developed which can cause a collapse. This problem can also occur in flush drillings without mud additives, if clay layers contribute to the viscosity of the flushing.

According to the design, the material and the weight of the casings and screens the installation has to be practised in hanging or standing position. Pipes are built in hanging position if the pipe joints can bear great tensile forces, otherwise they are installed standing on a strong bottom.

Procedure for hanging installation

The sump pipe is the first part to be installed, finally situated at the lowest position of the well. For installing the sump pipe into the borehole, the upper end of the pipe is connected with a hoisting device either by a thread or a flange. The hoisting device is then pulled up through a steel cable fastened to the derrick. The connection between the hoisting device and the cable is made by using a shackle. In this manner, the sump pipe is pulled up, positioned vertically above the borehole, let down cautiously and held at its upper end by a pipe clamp. There are special wooden clamps faced with leather and steel clamps for heavy casings. The opening of the clamp has to have exactly the same outer diameter as the pipe string in order to be built in. To bring the pipe under control, both halves of the clamp are placed below the cam at the end of the pipe and are fixed with screws. The clamp must be designed to bear the tensile forces via its small support width. The opening of a steel clamp should not have sharp edges in order to prevent pipe damages, especially at plastic coated pipes. To enable the lowering of the sump pipe into the borehole, it has to be filled with water to avoid buoyancy effects when dipping into the drilling mud. During the lowering of the pipe, the clamp is open. To hold the lowered sump pipe, the clamp is closed again. Now, the first screen is prepared. It has to be checked for damages, especially of the threads and the plastic coating. The screen is pulled up by using the hoisting device, connected with the pipe which hangs in the borehole fixed by a clamp and then after the opening of the clamp, the whole string is lowered into the borehole. This procedure is repeated until all screens and casings are installed.

At any well with a filter pack, it has to be ensured that the screens to be installed are placed exactly centric in the borehole to guarantee that the filter pack has the same thickness at every point. Therefore, centralizers are used to guide the pipe string. They consist of two pipe clamps with three or four bow shaped rips. At the screens, the centralizers are fixed directly below the pipe joints. For short pipes it is mostly sufficient to have centralizers every 5 m, for long metal pipes every 10 m. If the water level gauge (see chapter 3.5) of the well is not installed at the same time as the well string the centralizers have to be in the same position so that the gauge string can be pushed past the centralizers.

After the installation of the casing and screens into the open borehole the filter pack is dropped into the annular space tween the borehole wall and the screen wall. Regarding this operation, it has to be taken into consideration that the fall period of the material to the planned depth depends on the grain size. Sands and fine gravels need more time than coarser material. Placing sand and gravel at the same time leads to undesirable separation and as a result the filter pack will show bedding. The separation of various grain sizes increases with the well depth, the height of the water column, small grain diameters, a wider range between finest and coarsest grains, smaller annular space between the borehole wall and the pipes and a greater quantity of placed gravels. In every case, the occurrence of graded bedding has to be avoided. A drop pipe of plastic should be used to avoid plugging in the annular space. Another way of installing the filter package is by using tissue baskets, which are filled with filter material and fixed to the screens directly before letting them down into the borehole.

At fine-grained aquifers the filter pack can be placed in a double-layer. In this way the finer grain is located in the outer part and the coarser one close to the screen. The grain size of the coarser layer has to be calculated by multiplying the diameter of the smaller grain size by the screen factor 4.5. The thickness of the circular filter layers is given in Tab. 3-2.

Tab. 3-2: Minimum thickness of filter layers around well screen

Filter grain size (mm)	Thickness of filter pack (mm)
0.25 – 2.0	50
2.0 – 8.0	80
8.0 – 31.5	100

If an unconsolidated aquifer consists of various layers the filter pack can be suited to the finest grain or it can be placed in a vertical differentiation, according to the grain sizes of the various sediments.

Sealing

To protect from surface water intrusion a sealing must be installed. The process of filling the space between the well casing and the side of the drilled hole is called grouting. The properties of clay to absorb water and to swell up considerably, make it especially suitable as seal material. Bentonite is a clay that expands by about 13 times when it is mixed with water. For well construction, raw clay, air-dry lump clay, air-dried bricks and firm-dried clay pellets can be used. When mixed with cement the bentonite-cement slurry creates a seal that is able to absorb slight movement of the casing without cracking. The slurry is also more resistant than plain bentonite to washouts if high water content zones must be sealed.

Cement by itself tends to shrink as it cures and may pull away from the sides. Shrinkage is especially likely if the cement is mixed with too much water. Also cement heats up significantly when it cures and so cement grout is not recommended for plastic well casing.

Drill cuttings may not be used as a backfill as they do not pack together well enough to provide a good sealing. Typically grouting involves pumping cement and/or bentonite into the annular space starting at the bottom of the casing and filling up to the surface to avoid trapping of air and water. If the annular space between the boring wall and the casing is of sufficient size, the grout can be placed through a small grout pipe, lowered outside the casing. The casing is closed with a drillable plug at its lower end to prevent the inflow of grout. The casing has to be filled with water in order to avoid the risk of the slurry causing the casing to float upward. The grout can sink down due to gravity flow or it can be pumped to guarantee a rapid placement. The grout pipe can be left in place or can be removed gradually during the procedure of grout placement. However, the lower end of the grout pipe must always be positioned within the slurry. Another method to seal the upper section of a well sufficiently is to use a protective casing, which is provided with an elastic seal flange at its lower end. The annular space between the boring wall and the protective casing can be filled up with grout or clay.

Position of the pump

In any case, it has to be taken into consideration that the entrance nozzle of a suction pump or the suction area of a submersible pump must not be positioned inside a screen pipe, since the strong suction around the entrance area will affect the screen, the filter layer behind it and possibly the adjacent sediments of the aquifer as well. There are three possibilities for installing pumps: above, below or amidst the screens inside of casing pipes. In the case of an aquifer with great thickness, a good permeability and a small drawdown during discharge, the pump can be placed above the uppermost screen. This is the common position for submersible pumps in RBF wells in Egypt.

For aquifers of small thickness and/or deep drawdowns, the pump can be located within an extended sump pipe. In this case, it has to be guaranteed that the motor of the pump will be sufficiently cooled. Therefore, a pipe has to be installed surrounding the pump and forcing the water flow around the motor towards the entrance area. If the length of the screen pipe is sufficient, a blank casing can be installed in the middle of the screen string, where the pump can be situated.

3.3 Well development

After the construction of a well has been finished, the well must be developed; that means it is treated by high pumping rates in order to rise the porosity and the permeability of the aquifer, to stabilize the sediment around the well and to remove pluggings at the borehole wall or in the sediments adjacent caused by the drilling process. After this procedure, the well shall deliver "technically sand-free" water during continuous operation. An insufficient development of a well prevents to achieve the well capacity which could be obtained according to the hydrogeological conditions and the well design.

If there is an outwash of fine-grained particles during the running of a well, severe damages on the pump and other parts of the well by mechanical abrasion may occur. Deposits in the piping cause a reduction of the flow-section and plugging of nozzles and screens. Flushing out of material from underground is responsible for subsidence in the area around the well or even of the complete breakdown of the well.

One can distinguish between simple development and intensive development. At the simple development the whole screen string of the well is treated together. The development is started with a high pumping rate. To avoid bridging of sand grains at the screens, which may occur at a steady pumping, the development is carried out pulsating or intermittent. In this mode, the flow direction of the water is changed 6 to 10 times per hour, e.g. after 5 minutes of pumping the pump is switched out for another 3 minutes. To enable the water to flow back out of the pipe string, all valves have to be opened and the non-return valve of the used submersible pump has to be removed. Immediately after the stop of the pump, the flow direction of the water reverses, the rising pipe runs completely empty and a portion of the water returns back into the aquifer.

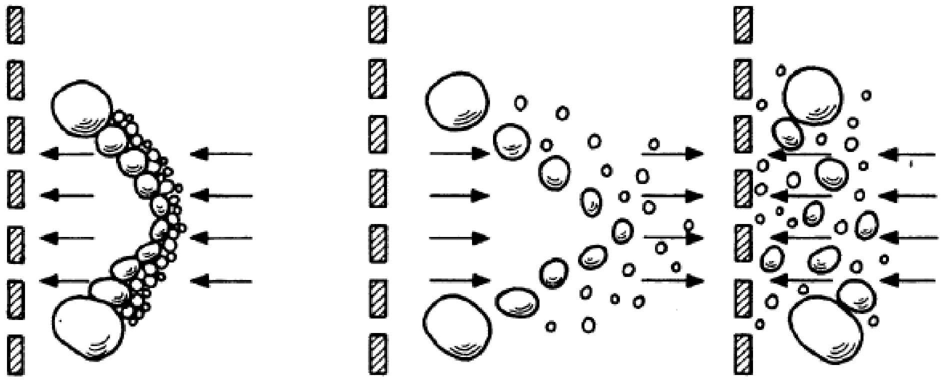

Fig. 3-8: Bridge formation and destruction of the bridges by reverse flow
(Langguth & Voigt 2004)

An intensive development has to be carried out if it is to be expected that there are plugged areas in the borehole wall and in the sediments adjacent, especially if drilling mud has been used. It is also employed, if there are thin layers of silt or fine sand within the aquifer which cannot be supplied with an adapted layer of filter sand and a special screen because of its small thickness. Using an intensive development, the screen pipe string of the well is developed in separate sections through surging with a special device consisting of a submersible pump with sealing members above and below. In this manner, the area of influence is restricted which allows a rather strong treating. The distance between the two sealing members ranges mostly from 2.5 to 5 m, depending on the aspired intensity of the development and the length of the pump. The developing takes place in steps from the lower to the upper part of the screen string. The single development stages have to overlap 50 cm.

The discharge rate during the development should charge the developed section at least five times more than the discharge at the expected continuous well yield. The minimum discharge for the development is calculated as in eq. 3-11.

$$d_d = \frac{d_c}{l_s} \cdot l_m \cdot 5 \tag{3-11}$$

where, d_d is the discharge at development [L/T], d_c is the planned continuous discharge [L/T], l_s is the length of screen string [m], l_m is the distance of sealing members [m].

To determine the sand content of the pumped groundwater it is necessary to measure both, the discharge and the sand content. The development of a single stage is considered to be successful, when the sand content is diminished to about 0.1 to 0.5 g/m³. Measurements of the sand content during the development indicate the success of the procedure. The capacity characteristic of a drilled well improves after a sufficient development. If needed, the development can be repeated.

According to Cashman et al. (2012), the best way to determine the sand content is to take a sample of water in a clean white plastic tub (of the sort used to hold soil samples) and any sand will be clearly visible in the bottom of the tub. A specialist sediment sampling container called an “Imhoff cone” can also be used to check for sediment in the water, but this device is intended for much higher sediment loads. Development is usually discontinued when the well no longer yields sand or fine particles when pumped by the airlift. Based on experience, wells in granular aquifers can take between 6 and 12 hours to be effectively developed. If a well still yields high amounts of sand after more than 2–3 days, the well is unlikely to improve, and a replacement well (possibly with a different filter medium) should be considered.

After the development the sediment deposits in the sump pipe are bailed with a sand pump or an air lift pump (mammoth pump). For deep wells the sump pipe has to be calculated large enough to take all the deposited material.

3.4 Well Yield Test

Different types of pumping tests are undertaken with the most common being the step-drawdown (variable discharge) and constant discharge tests. Step-drawdown tests measure the well efficiency and the well performance (well yield). The step-drawdown test is performed to determine the yield of the production well (PW) and the residual sand content in order to ascertain a "technically sand-free" water lifting. It can also be performed to determine the well discharge for a subsequent constant discharge test to measure aquifer characteristics (transmissivity, hydraulic conductivity) and help to identify the nature of the aquifer and its boundaries. It is also recommended to carry out a television logging and to store the log on video for later comparisons.

All types of pumping tests involve controlled pumping from the PW and monitoring of the discharge flow rate from the PW and the drawdown in observation wells (OWs) at varying radial distances away (Fig. 3-9). Detailed procedures for step-drawdown and pumping tests are described in international standards (e.g. European Standard EN ISO 22282-4, 2012; British Standard BS ISO 14686 2003) and in standard scientific and professional literature (e.g. Cashman et al. 2012; Kruseman & de Ridder 1973, 1990). In general, the pumping tests consist of the phases in Tab. 3-3, whereby the focus is on the step-drawdown (well yield) test in this guideline (Tab. 3-3, phase 3).

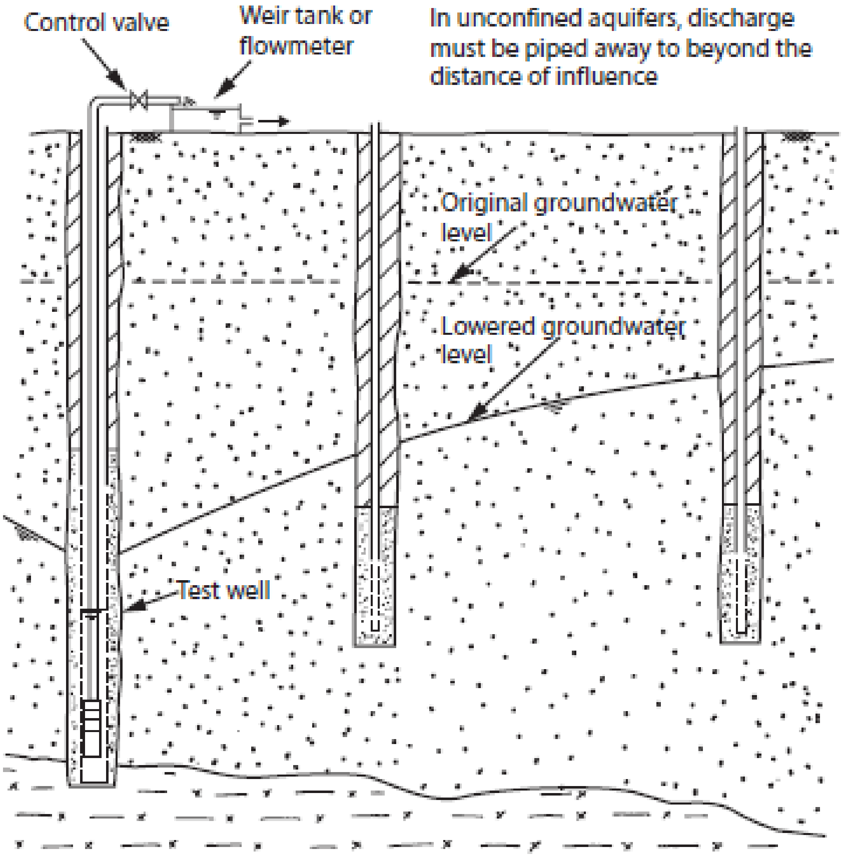

Fig. 3-9: Components of a pumping test (Cashman et al. 2012)

Tab. 3-3: Phases of pumping tests (after Cashman et al. 2012; EN ISO 22282-4, 2012)

Phase	**Description**
1. Pre-pumping monitoring	Monitoring natural GW levels in OWs for few days to weeks before commencing pumping to determine natural/artificial variations in GW
2. Equipment test	Short period of pumping (15–120 min) to check pump, flowmeters and dataloggers and to check for leaks in the discharge pipework
3. Step-drawdown test	- Measures well efficiency and well performance (well yield) - Period of continuous pumping typically 4–8 hours duration, during which the flow rate is increased in a series of steps. This is used to determine the well discharge for the constant rate pumping test (phase 4 below) - Each step is of equal duration (normally 60 or 100 min) and the test should have four or five steps at roughly equal intervals of flow rate - Water levels are normally allowed to recover for at least 12 h before the constant rate pumping phase can be started.
4. Constant rate pumping phase	- Considered to be the main part of the test and involves pumping the well at a constant flow rate (chosen following the step–drawdown test) - The constant rate test normally lasts 72 hours or even longer
5. Recovery phase	Water levels in OW and test well are monitored (1-4 days) as they recover after pumping stops.

Important considerations

- The principal parameters to be measured during a test are water levels in the PW and OWs, and the discharge flow rate from the PW.
- Water level should also be measured inside the PW. This will require the installation of a dip tube in the well so that the dip meter tape does not become entwined around the pump riser pipe due to the swirl of water near the pump intake.
- If the site is subject to natural GWL fluctuation (e.g. tidal effects), drawdowns significantly greater than the background variations should be achieved during pumping.
- It is essential that pumping is absolutely continuous during the initial hours of pumping. If pumping is interrupted, such as a pump or power failure, it is essential that the test is suspended until GWLs return to equilibrium, or those levels recorded before the commencement of any pumping. Only then should pumping recommence. Once the test pumping has been going on for 24 h or more, occasional short interruptions of a few minutes only are permissible (for example, for daily checks of generator oil levels or fuel refill).
- OWs must be installed deeper than the expected drawdown. Depending on the permeability of the aquifer, the distance from the test PW to the nearest OW should be in the range of 2–3 m (in a low to moderate permeability aquifer) or to 5–10 m (in a high-permeability aquifer) from the PW. More distant observation wells should be located within the anticipated distance of influence of the test well.

Water level and pump discharge measurements

All readings need to be referenced to the time elapsed since the start of pumping (or recovery) for that test phase. The frequency of monitoring must be matched to the rate of change of the measured parameter. During a test, the flow rate should remain approximately constant, but the water levels will fall rapidly immediately after the start of pumping, and will fall slower as pumping continues. Measurements of water levels must be collected very frequently at the start of the test, and become less frequent as time passes. Unless otherwise warranted, e.g. due to the objective of the test, the schedule in Tab. 3-4 should be used for water level readings.

Tab. 3-4: Time-intervals for measuring water levels in production and observation wells (EN ISO 22282-4, 2012)

Elapsed time since start of pumping	Time-interval
0 – 5 minutes	≤ 30 seconds
5 – 15 minutes	≤ 1 minute
15 – 30 minutes	≤ 5 minutes
30 – 60 minutes	≤ 10 minutes
1 – 4 hours	≤ 1 hour

It is important to ensure that sufficient readings are taken to allow the rate of change of water levels to be clearly identified. Shorter time-intervals may be necessary in case the GWL in the PW and OWs changes rapidly. However, for OWs further away from the PW or if the OWs penetrate the upper or lower boundary layers of the aquifer, then short intervals are not necessary in the first few minutes of pumping. In case the well yield test lasts longer than 4 hours, then the time-intervals in Tab. 3-5 can be used.

Tab. 3-5: Time-intervals for measuring water levels during long duration well yield/ pumping tests (Kruseman & de Ridder, 1973)

Elapsed time since start of pumping	Time-interval
5 hours – 48 hours	1 hour
48 hours – 6 days	3 times/day
6 days till end of pumping	1 time/day

Water levels can be measured by manual dipping with a dip meter/ water-level tape. However, at the start of the test, readings need to be taken very frequently. If more than a few OWs are to be monitored manually, several people may be needed to take the frequent readings at the start of the test, creating potential problems of coordination between personnel. There is also the problem of actually finding enough capable or qualified people. This can lead to poor recording of the precise timing of measurements taken during the first few minutes of pumping, leading to difficulties in subsequent analysis. If manually taking water level readings with a limited number of observers, it is best to concentrate initial monitoring on points nearest the test well. As time passes and the period between readings increases, more observation wells can be included in the monitoring. The use of electronic data logging equipment, linked to pressure transducers in the OWs, helps overcome such staffing problems. Once programmed to take readings at the appropriate interval, data loggers reduce the personnel requirements and produce the test measurements in electronic form, allowing rapid analysis using spreadsheet programs.

The pump discharge can be measured using:
- a tank or gauge box fitted with a V-notch or rectangular weir
- or installing an integrating flowmeter into the discharge pipeline.

If flowmeters are used in preference to weir tanks, it is good practice to include a settlement tank in the discharge line so that it can be visually checked for suspended solids in the discharge water. In general, once adjusted at the start of the test phase, the flow rate should not vary significantly. Immediately following commencement of pumping, flow rate does not need to be monitored as frequently as water levels for the first hour or so of pumping. The discharge outlet should be well away from the test area so that return seepage does not affect the drawdown levels.

Determining the well discharge for the step-drawdown/pumping test

According to EN ISO 22282-4 (2012), the maximum well discharge Q_d can be estimated with one or more of the following:
- based on the objective of the pumping test and the existing local conditions
- and/or with the help of the theoretical estimation of the well yield (eq. 3-12).

The theoretical estimated maximum well discharge Q_d is a result of the following criteria (EN ISO 22282-4, 2012):
- The entrance velocity v of the water into the well filter screen (eq. 3-12)

$$v = \frac{Q_d}{\pi D L} \quad (3\text{-}12)$$

should be ≤ 0.01 m/s, where D is the casing diameter and L is the length of the filter screen.

- The height of the water column above the pump-entry should be more than 0.5 m.
- The drawdown for a discharge of $0.2 \cdot Q_d$ can be exactly measured with the available equipment.

Alternatively, the maximum well discharge can also be estimated from the equipment test (Tab. 3-3, phase 2; Cashman et al., 2012).

Execution of step-drawdown test

The following general information should be recorded before commencing the test:

- Elevation of ground surface at the PW and OWs,
- Elevation of reference datum or point from which water levels are read-off/ measured in each well. The top of the well casing in OWs and the top of the water level pipe in the PW is often used as a datum.
- Depth of the well screen in PW and the depth of response zones in all OWs.
- Distances from the centre of the PW to all OWs.

The well is pumped in four consecutive steps, or an absolute minimum of three steps (EN ISO 22282-4, 2012; Cashman et al. 2012). The flow rate in each step is constant, with the rate for each step greater than the last. The pumping duration of the first step is normally 2 hours and 1 hour for each of the remaining steps. The selected discharge for each pumping step Q_i is a function of the maximum well discharge Q_d (3-13)

$$Q_i = \frac{0.8 \cdot Q_d}{5-i} \quad (3\text{-}13)$$

where, i is the pumping step 1 to 4.

Important aspects during the test are (EN ISO 22282-4, 2012; Cashman et al. 2012):

- For each pumping step, the water level in the PW is to be measured at least every 5 minutes in the first 30 minutes and thereafter every 10 minutes.
- The time when the pump is started should be recorded.
- The control valve of the pump in the PW should be adjusted to achieve the desired flow rate Q_i as quickly as possible after the start of the step in the step–drawdown test. Once the flow rate has been set, the valve should not be further adjusted as this will affect the drawdown and complicate analysis of results.
- At the start of each step, readings in PW and OWs are to be taken simultaneously.

Recovery phase

The recovery phase begins immediately when the pumping in the last step stops. Monitoring of water levels in PW and OWs should be continued at analogous intervals to the pumping phase until full recovery of water level is approached. During the initial recovery period, readings should be at frequent intervals similar to the start of pumping. As the rate of recovery slows down, the intervals between readings may be extended progressively. The readings can be stopped, when the change in water level from at least 3 consecutive readings at an interval of at least 1 hour is not more than 1 cm (EN ISO 22282-4, 2012).

Data and test evaluation

According to Chapman et al. (2012), all water level and flow rate data gathered during the test should be plotted in graphical form while the test is in progress. Even if this is done on-site in rough form on a graph pad, it will help identify any anomalies or inconsistencies due to occasional human error. The most useful method of plotting data is to use the Cooper–Jacob straight line method, but other methods such as those described in EN ISO 22282-4 (2012) can also be used. The latter is especially useful to determine the pumping rate for the pumping test to determine aquifer characteristics (Tab. 3-3, phase 4). In the Cooper–Jacob straight line method, drawdown *s* is plotted on the vertical axis (linear scale) against elapsed time *t* on the horizontal logarithmic scale (Fig. 3-10).

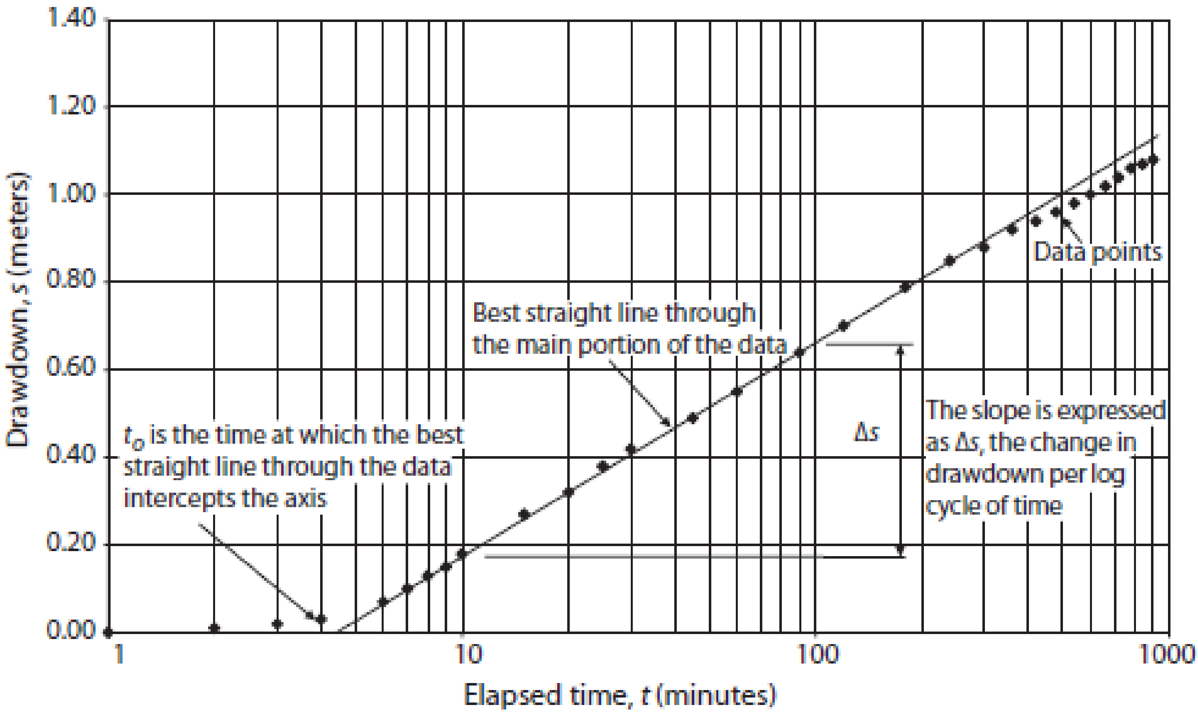

Fig. 3-10: Example of the Cooper-Jacob straight-line method of plotting data (Cashman et al. 2012)

3.5 Well head protection measures

During construction of wells and maintenance of pumps, all material (filter gravel, bentonite, borehole material) and parts/components which will be used for well construction and refilling of borehole have to be stored in a clean place (e.g. on plastic sheets) and covered to be protected from microbiological and other contaminants, especially faeces of human and animals.

The well base and all openings at the well head have to be inspected regularly and covered/closed/sealed watertight to prevent any entrance of pollutants or animals/insects. Especially openings for cables and the inner and outer piezometers should be covered by removable and watertight seals or caps.

Well heads have to be protected from flooding by placing at an elevated level compared to surrounding ground surface.

No chemicals should be introduced into the well or piezometers. In case of microbiologically contaminated wells only disinfection liquids should be used (e.g. hypochlorite solution) but no bleaching powder.

Any observation of strong deposition of fines or plant growth at the river bank or river construction works (digging etc.) should be documented.

3.6 Monitoring wells and devices

Groundwater level gauges, also called water monitoring pipes, piezometers, observation wells, monitoring wells, etc. facilitate the possibility to measure the groundwater level. In addition, they can be used to investigate groundwater temperatures and to take groundwater samples. Piezometers are also installed within the gravel pack of wells in order to obtain information about the entrance resistance of the well screen and the filter layer around.

Like drilled wells, monitoring wells consist of casing, screens and a sump pipe. Commonly, the screen pipe has a length of 2 m and is surrounded by filter sand of 1 – 2 mm grain size. In aquifers of great thickness, two or more screens can be installed at various depths.

Monitoring wells are commonly made of plastic pipes and plastic screens. In shallow aquifers consisting of sand, gauge pipes with a 2 inches diameter can be rammed in (up to about 10 m depth), whereas deeper aquifers require drilling operations. Pipes and screens of monitoring wells are connected by threads. There are several kinds of centralizers and sump pipes offered. Monitoring wells must be equipped with an appropriate top which can be locked up with a key to prevent contamination.

The diameter of monitoring wells depends on the planned use. If only the water level shall be measured with the help of an electric light probe / dipper, a diameter of 2 inches is sufficient, even for great depths. Water level recorders often require a well diameter of 4 inches. Water sampling in monitoring wells requires pipe diameters according to the diameter of the pump, commonly 4 inches.

The uppermost part of the annular space between the pipe of the monitoring well and the borehole wall must be sealed cautiously. In the case of groundwater storeys, they have to be separated by seals, which are installed at the levels of the confining layers. If two aquifers have different piezometric surfaces and the seals between them are not tight, a steady flow of water will occur through the boring from one aquifer to the other, disturbing the hydraulic properties and falsifying the groundwater level measurements. Borings for monitoring wells have a very small annular space. Therefore, it is very difficult to place clay pellets exactly at the intended levels. By this, the use of pellets as sealing material is restricted up to some 10 m depth. Deeper situated seals must be made with sealings, which can be used in a fluid condition. In both cases, the sealing material has to be placed with the help of a drop pipe.

After completion, the monitoring well has to be tested by conducting a recovery test. Thus, a certain quantity of water is poured into the pipe, immediately rising the water level in the pipe. According to the permeability of the screen and the surrounding filter, the water level decreases down to the initial level relatively quickly. For the analysis of a recovery test, Nattermann's formula can be used

$$\mathrm{E} = \frac{2\,(H_O - H)}{t(H_O + H)} \quad (3\text{-}14)$$

where, H_O is the initial water level in cm, H is the water level in cm above the initial level at the time t, t is the measuring time in min.

The monitoring well is able to function when E > 0.0115.

After having examined the ability of the monitoring well, the reference point for the measurements has to be marked at the rim of the uppermost pipe. The position of this point (coordinates, masl, level above or below the earth's surface) must be surveyed very accurately.

Monitoring wells are used for several purposes. They are most commonly used for the investigation and the steady control of the groundwater level, in order to receive information on the range of level fluctuations, the inclination of the groundwater surface and the flow direction. Normally, measurements should be made at weekly or monthly intervals; in special cases daily. To ensure comparable results, the level measurements must be carried out always in the same way and at the same reference point. To find out the flow direction of the groundwater, at least 3 monitoring wells must be installed in the same aquifer. To get general information on the behaviour of the groundwater level, special aquifers should be supplied with a more or less dense pattern of monitoring wells.

Water samples for chemical analyses can be taken from monitoring wells with the help of scooping devices or by special small sized pumps. With all types of sampling, it has to be taken into consideration that the stagnant water in the casing of a monitoring well changed its chemical condition, especially by alterations of the lime-carbonic acid-equilibrium. Therefore, a water sample taken from inside the casing is not representative of the fresh groundwater. Fresh groundwater can only be tapped at the level of the screens, where the groundwater can flow horizontally through the openings or has to be pumped out.

Scooping devices consist of a 30 to 50 cm long pipe of metal, which is equipped with non-return valves at both ends. When lowering into the monitoring well, the valves open and the water flows through the pipe. At the moment the movement downward is stopped, the valves close and a quantity of water is caught in the pipe and can be pulled up. For getting a sample, representative of the groundwater, the stagnant water in the casing should be pumped off and the scooping device should be stopped exactly at the level of the screen pipes.

Pumping can be done with the help of suction pumps, special piston pumps and submersible pumps. Most of these pumps require well diameters of 4 inches or more. During the pumping, the temperature, electrical conductivity, pH-value and dissolved oxygen (DO) concentration of the pumped water should be measured, since the values are not the same in stagnant water as in the fresh groundwater. Normally, the electrical conductivity increases and the pH-value and DO concentration decreases when the stagnant water is pumped off and the fresh groundwater is delivered. After this change the water sample can be taken.

3.7 Water quality monitoring

Water samples should be taken from the River Nile, from every well in operation from its sampling tap and from existing observation wells. If no OWs are available and samples are only taken from PWs, it has to be taken into account that the pumped water is a mixture of bank filtrate and landside groundwater.

The parameters temperature (T), dissolved oxygen (DO), pH and electrical conductivity (EC) should be measured on-site using a multi-probe instrument, e.g. WTW multi 3430 (Wissenschaftlich-Technische Werkstätten GmbH, Weilheim, Germany). Also other parameters as shown in Tab. 3-6 can be determined on-site using a portable, LED-sourced colorimeter, e.g. Hach DR 900 (Düsseldorf, Germany). In addition, determination of the acid capacity $K_{S4.3}$ (bicarbonate) in the field at every sampling event by alkalimetric titration with 0.1 M hydrochloric acid is recommended as basis for ion balance calculations.

Tab. 3-6: Parameters and methods often used for analysis with DR 900 Colorimeter

Parameter	Abbreviation	Unit	Range	Method
Ammonium	NH_3-N	mg/l	0.01 – 0.50	8155
Iron	Fe	mg/l	0.02 – 3.0	8008
Manganese	Mn	mg/l	0.006 – 0.70	8149
Sulfate	SO_4^{2-}	mg/l	2 – 7000	10248
Turbidity	Turb	FNU	21 – 1000	8237

For laboratory analysis, water samples should be collected for determination of dissolved organic carbon (DOC), major cations and anions, redox species and microbiological indicators. All samples for DOC and cations should be filtered immediately after sampling through 0.45 µm syringe membrane filter. The samples for cation analysis should be preserved with 0.1 M nitric acid (HNO_3). Major cations sodium (Na^+), potassium (K^+), calcium (Ca^{2+}) and magnesium (Mg^{2+}) as well as manganese (Mn) and iron (Fe) should be determined in the laboratory, e.g. using AAS or ICP. Chloride (Cl^-), nitrate (NO_3^-), sulfate (SO_4^{2-}) should be determined with ion-chromatography.

Tab. 3-7: List of anions and cations to be analysed and required range of quantification

Cation	Limit of Determination [mg/l]	Anion	Limit of Determination [mg/l]
Manganese	0.001	Chloride	1
Iron	0.01	Sulfate	0.5
Sodium	1	Nitrate	0.1
Potassium	0.5	Bicarbonate	1
Calcium	1		
Magnesium	1		

4 Post-treatment of RBF water for drinking water supply

4.1 Removal of iron and manganese

At many RBF sites, anoxic conditions are already present and dissolution of iron and manganese would develop during RBF (Fig. 4-1).

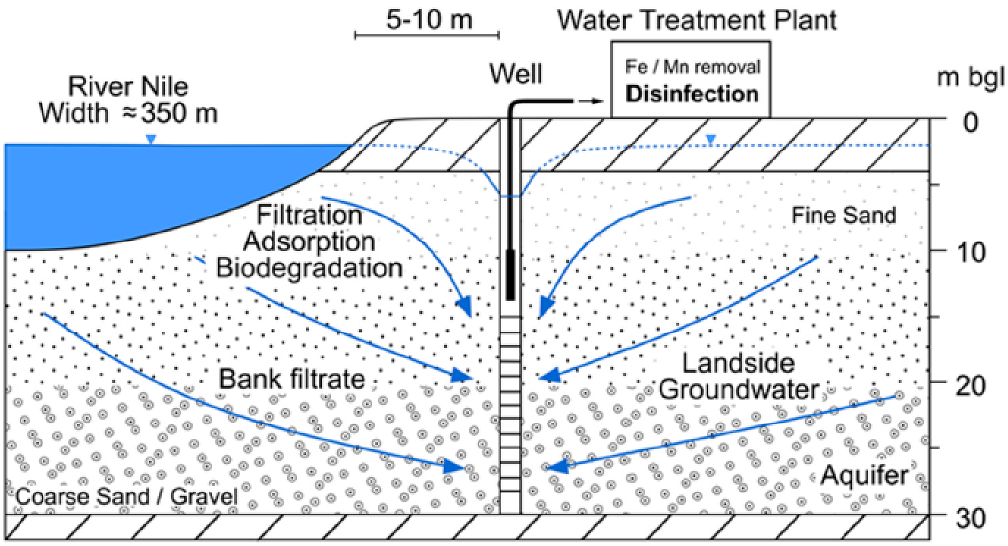

Fig. 4-1: Schematic diagram of processes affecting water quality during bank filtration (Wahaab et al. 2019)

However, processes occurring during RBF may result in lower Mn and/or Fe concentrations than observed in local groundwater. Continuous pumping and low(er) iron and manganese concentrations in bank filtrate would change the local equilibria in the aquifer and result in dissolution and/or desorption of iron and manganese. A prediction of Mn and Fe concentration in pumped water is not straightforward, as the river stage and well operation will affect the portion of bank filtrate (higher portion during floods, lower portion after non-operational periods of wells) and the landside groundwater quality has some dynamics (Grischek & Paufler 2017). Mn and Fe concentrations in pumped water may change within a few months up to one year as shown for sites in Upper Egypt by Wahaab et al. (2019). Thus, the design of post-treatment should be finalized only after a stable concentration of Mn and Fe is achieved in the pumped water or it should be adjusted after one or two years of operation.

The treatment processes that are applied to remove dissolved iron and manganese (Fe^{2+}, Mn^{2+}) are referred to as deironing (sometimes also called deferrization) and demanganization processes. The World Health Organisation recommends an iron and manganese concentration in drinking water of less than 0.3 mg/l and less than 0.1 mg/l, respectively (WHO, 2011). While the European Commission recommends an iron concentration less than 0.2 mg/l in drinking water (EC, 1998), several water supply companies in the Netherlands are aiming at an iron level of <0.05 mg/l in order to minimise the distribution system maintenance costs (Sharma et al. 2005). However, the recommended guideline values for iron and manganese after treatment are lower; typical values are <0.02 mg/l for iron and <0.01 mg/l for manganese (Worch 2019).

For groundwater, the typical deironing/demanganization process consists of aeration followed by filtration. This is the preferred method in developed as well as in developing countries because, compared to other methods, this method is more economical, less complicated and generally avoids the use of chemicals, which is not usually welcome in the water industry (Sharma et al. 2005). If necessary, the process is coupled with deacidification (Worch 2019).

The following sections on deironing/demanganization are extracted from Worch (2019).

The introduction of air can be carried out by open aeration with waterfall aerators (e.g. spray aerators, cascade aerators, cone aerators) or by pressure aeration (direct injection into the pipeline, closed spray reactors supplied with compressed air). In the first case, the introduction of air is accompanied by the stripping of dissolved gases, such as CO_2, CH_4, and H_2S. The central element of the deironing/demanganization process is the filter that retains the oxidation products. The solids accumulated in the filter bed provide the catalytic active surfaces for the further oxidation and also act as carriers for the biofilms formed by iron and manganese bacteria. The most commonly used filter materials are quartz sand or gravel. If a simultaneous deacidification is wanted, also calcium carbonate or half-burnt dolomite can be used. Due to the deposition of oxidation products, the filter resistance increases over the filter run time. Therefore, the oxidation products, deposited on the filter material, have to be removed through backwashing from time to time. Water free of disinfectants has to be used for backwashing to avoid killing of the bacteria in the biofilm.

The process design depends on the chemical composition of the water to be treated, in particular on the content of iron and manganese but also on the content of other constituents relevant for the treatment process. Waters with very low redox potentials are characterized by lower concentrations of iron and manganese and occurrence of dihydrogen sulfide and methane. Such waters can be treated by open aeration with subsequent filtration. A single-stage monomedia filter is typically sufficient (Fig. 4-2). The open aeration allows the stripping of the dissolved gases dihydrogen sulfide and methane and also of dissolved CO_2.

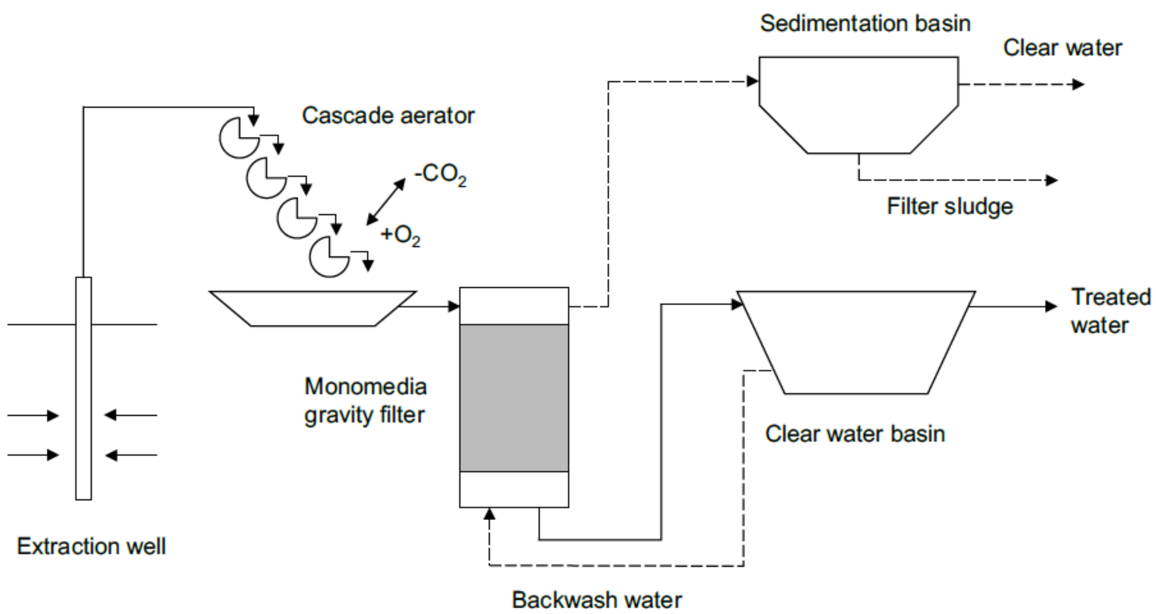

Fig. 4-2: Process scheme of single-stage deironing/demanganization with open cascade aeration (Worch 2019). The dashed lines show the flow regime during backwashing

Higher concentrations of iron and manganese are typically found in waters under moderately reduced conditions. In principle, raw waters with higher iron and manganese concentrations can also be treated in a single-stage monomedia filtration process. Alternatively, a dual-media filter (e.g. sand and anthracite) or a two-stage filtration with two monomedia filters may be suitable (Fig. 4-3). A two-stage filtration may be in particular necessary when the manganese concentration is very high. This technique allows separation of the iron and the manganese oxidation and permits adjustment of the process conditions according to the different requirements (e.g. different filter run times, intermediate CO_2 removal to increase the pH for the subsequent demanganization, filter conditioning only for the second filter).

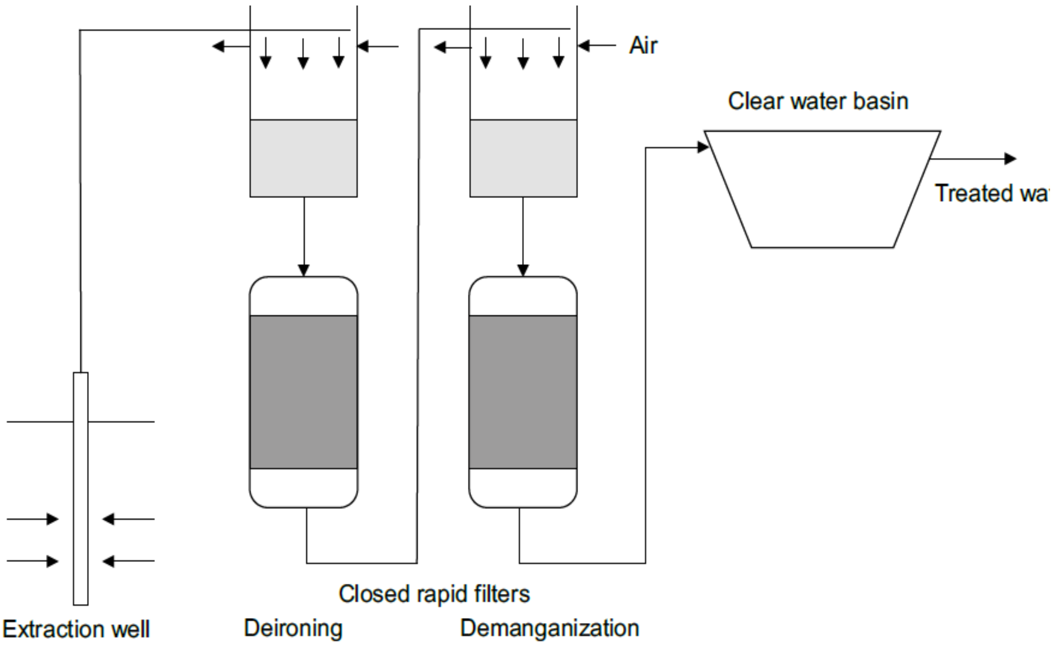

Fig. 4-3: Process scheme of two-stage deironing/demanganization with intermediate aeration (Worch 2019).

A general approach for the process design is based on the maximum filter velocity as the essential design parameter. The maximum filter velocity is that filter velocity that still allows a complete conversion of the considered reduced species within the filter. The reaction time that is needed for a complete conversion is determined by the reaction rate and has to be derived from the rate equation. The knowledge of the required reaction time is the crucial point of this model approach. Due to the complexity of the oxidation processes and the various hydrochemical impacts that have to be considered, in practice, the filter type (monomedia, dual-media), arrangement (single-stage, two-stage), material, and design are therefore often selected on the basis of experiences with raw waters of comparable composition or on the basis of pilot-plant experiments with the water to be treated.

However, according to Sharma et al. (2005), the presence of iron oxidising bacteria in iron removal filters could contribute to increasing their efficiency by catalytically oxidising Fe^{2+} to Fe^{3+} if the rate of oxidation of Fe^{2+} is very slow in water. This type of iron oxidation is only possible if the iron is entering the filter bed in Fe^{2+} form. However, in the presence of oxygen,

Fe^{2+} oxidation starts immediately (after the aeration step) in the supernatant and filter bed. Moreover, the adsorption–oxidation mechanism will occur in the filter bed as well, implying that biological iron removal is likely to be supplementary to conventional physicochemical iron removal (Sharma et al. 2005).

4.2 Disinfection

Water for human consumption has to guarantee the microbiological pureness in any case from the waterworks up to the customers tap. Due to the different quality of raw waters used for drinking water production and according to their pollution level, a conventionally or enhanced treatment regime supplemented by a final disinfection is necessary in most countries. This process is always connected with the undesirable formation of organic as well as inorganic disinfection by-products (DBPs), some of which are toxic. Hence the disinfection process has to consider both a complete inactivation of all microbiological pollutants and DBP formation. The latter should be as low as possible after the treatment process.

The method of choice for drinking water disinfection worldwide is the application of chlorine in form of chlorine gas as well as hypochlorite solution. Nevertheless, there are some other options which are quite efficient and applied in a number of countries. These are the disinfection regimes using chlorine dioxide and UV-radiation.

This sub-chapter summarizes fundamental knowledge of drinking water disinfection in the field of (i) framework conditions, (ii) the characterization of the disinfection agent applied and (iii) the avoidance of undesirable reactions in water that cause the formation of DBPs including taste and odor. This basic knowledge is a pre-condition for safe drinking water disinfection and the application of an optimized process which is focused to the actual water quality.

Frameworks, guidelines and regulations

Framework conditions of drinking water quality including the disinfection process are published by the World Health Organization (WHO 2011). On the other hand, each country has to establish its own standards, which consider local conditions appropriately. In Tab. 4-4 the DBP limits set by the WHO and European Union are summarized.

The dosage of chlorine causes the formation of several DBPs. In general, these are halogenated inorganic and especially organic compounds, which can be of human toxicity and/or form disagreeable taste and odor of the water (described as musty and foul by consumers). The most important organic DBPs belong to the so called trihalomethane (THM) group (Müller et al. 1993; ISPRA 1996; EU directive 98/EG; Bundesgesundheitsblatt 2002). This group consists of four compounds: chloroform ($CHCl_3$), bromodichloromethane ($CHBrCl_2$), dibromochloromethane ($CHBr_2Cl$) and bromoform ($CHBr_3$). The WHO guideline value of 200 µg/l is not ambitious, nevertheless it considers the rule: disinfection first and is thereby most important. In order to guarantee the least possible formation of DBPs in water, special knowledge concerning the characteristics of chlorine and its reactions in water are necessary.

Tab. 4-4: Relevant standards of disinfection by-products

Disinfection by-product	Formula	EU guideline	WHO standard

		µg/l	µg/l
THM	CX_3H	100	200[1,2]
Chlorite[3]	ClO_2^-	-	700
Bromate	BrO_3^-	10	25
Chlorate	ClO_3^-	-	700
Perchlorate[4]	ClO_4^-	-	-
Chloral hydrate	Cl_3CCHO	-	10
Dichloroacedic acid	HCl_2CCOOH	-	50
Trichloroacedic acid	Cl_3CCOOH	-	100
Formaldehyde	HCHO	-	900

[1] 200 µg/L: to Chloroform regarded; [2]$[CHBr_3]/100 + [CHBr_2Cl]/100 + [CHCl_2Br]/100 + [CHCl_3]/200 \leq 1$;
[3] in case of chlorine dioxide application; [4] under discussion

Some of the inorganic DBPs are also relevant for human consumption. These are chlorite in case of chlorine dioxide application, chlorate which is formed in hypochlorite stock solutions by higher temperatures and effects of light, both of which are relevant to Egyptian conditions. Bromate is also a relevant DBP because it is known that this ion shows carcinogenic impact (Gordon et al. 1993, 1995; Schmidt et al. 1994; von Gunten and Hoigne 1994).

Those compounds which give the water a musty and septic impression are so called chlorinated amines which are formed by chlorination of amino acids, a group of compounds found ubiquitous in natural waters. Nevertheless, problems concerning taste and odor arise if the concentration of these acids is at a "normal" level (<10 µg/l). On the other hand, in case of algal growth in raw water used for drinking water production and/or impact of wastewater, e.g. if river water is used as a source, the level of amino acids can be quite high.

Another aspect which has to be considered is the bromide concentration in the water. Bromide is always a part of waste water impacted source water and cannot be removed during drinking water treatment. Bromide is the decisive pre-cursor for bromate and bromo-organic DBPs formation (Schmidt et al. 1993). Today it is known that brominated by-products are of higher toxicity than the chlorinated byproducts.

Characteristics of disinfection agents

The most common disinfection agents are chlorine gas and hypochlorite solution, which are strong oxidation agents. This attribute is the basis of disinfection, because in consequence the cell walls of the microorganism present in water will be oxidized and thereby destroyed. The scientific base of this process is described by the NERNST equation (eq. 4-1).

$$E = E_0 - \frac{R \cdot T}{z \cdot F} \ln \frac{a(Ox)}{a(\mathrm{Re}d)} \tag{4-1}$$

where, E: electromotive force in Volt, E_0: normal potential in Volt, R: gas constant, T: absolute temperature, F: Faraday constant, z: current equivalent and a: redox activity.

In this equation the term E_0 describes the so called normal potential, a constant which is characteristic for different redox pairs of chlorine in water. These values are pH-value dependent and describe the rate of oxidation - or disinfection power (Christen 1973).

For the normal pH-range of drinking water, the states of hypochlorous acid and hypochlorite ion are relevant. The oxidation power of the acid is much stronger than that of the hypochlorite ion. Therefore, the disinfection efficiency of chlorine in water is pH-value dependent. The higher the pH-value, the lower is the disinfection power. In the case of drinking water this can be problematic, because the optimal pH-value for drinking water lies between 7 and 8. The reaction of chlorine with water is described by equations 4-2 and 4-3.

$$Cl_2 + H_2O \Leftrightarrow HCl + HOCl \quad (4\text{-}2)$$
$$HOCl \Leftrightarrow H^+ + OCl^- \quad (4\text{-}3)$$

A decisive factor for disinfection is the contact time of chlorine. It is known that chlorine needs up to 30 minutes for the process of cell destruction of the microorganisms to be completed. The undesired, but not avoidable reaction is that of chlorine with the bromide ion, which can occur in natural waters in concentrations of several µg/l (eq. 4-4 and 4-5).

$$2Br^- + Cl_2 \Leftrightarrow Br_2 + 2Cl^- \quad (4\text{-}4)$$

$$Br_2 + H_2O \Leftrightarrow HBr + HOBr \quad (4\text{-}5)$$

This active bromine has also a disinfection impact, nevertheless, it is much lower than that of active chlorine and it forms brominated DBPs, such as bromoform. Today it is well known, that the kinetics of the formation of brominated DBPs is favoured against the chlorinated structures. Active bromine reacts more easily with organic compounds. This is always a problem, because brominated DBPs are more toxic for humans. In order to classify and estimate the disinfection efficiency, very often the so called ct-concept is used. This concept describes the disinfection by the mathematical product of c (concentration of the disinfection agent) and t (contact time of the disinfection agent). In Tab. 4-5 the ct-values achieved under laboratory conditions for complete disinfection of several microorganisms are summarized as an example. It should be noted that in practice these values can be different because of the occurrence of particles in water which can reduce the disinfection efficiency of all agents significantly.

Tab. 4-5: ct-values for 99% elimination of several microorganisms, ct-values in $mg \cdot min \cdot l^{-1}$

Microorganism	**chlorine**	**chlorine dioxide**	**ozone**
E. Coli	0.062	0.118	0.02
Enterococcus faecium	0.070	0.339	
MS 2	0.477	0.101	
PRD 1	0.082	0.047	
Polio 1			0.1-0.2
Rotavirus			0.006-0.06
Gardia lamblia			0.5-0.6
Gardia muris			1.8-2.0
Cryptosporidien			3.5-10

Besides the halogenated DBPs formation, there are some other negative effects of disinfection, which are caused by the oxidative impact of the disinfectants especially on organic molecules. This process is called “cracking” of high molecular organic structures. As a consequence, the so called biologically degradable organic matter dissolved in water (BDOC) is formed, which is generally of lower molecular weight (Werner 1984).

Disinfection by-products formation

The most important factors for DBP formation are:
- dose of disinfection,
- contact time,
- concentration of the reaction partners with chlorine, the so called precursors, and
- pH-value of the water.

The reaction of chlorine with natural organic matter is the reason of THM and chlorinated acid formation which are the precursors of the THM (eq. 4-6).In any case the rule, which should be noted is: the higher the chlorine dose and the higher the contact time of free chlorine the higher the THM formation in disinfected waters.

$$NOM\ (e.g.:\ RCOCH_3) \xrightarrow{Cl_2/HOCl/OCl^-} CHX_3 + R\text{-}COOH \qquad (4\text{-}6)$$

CHX_3: chloroform, dichlorobromomethane, dibromochloromethane, bromoform
NOM: natural organic matter

Due to the reaction kinetics it can be noted: one of the most important factors of DBPs formation is the content and the character of NOM. This parameter is analyzed in form of DOC. The knowledge concerning DOC (as exact as possible) is important for all waterworks. Nevertheless, the analysis is difficult and requires special instruments, e.g. TOC/DOC-analyzers and/or fluorescence spectrometers. In cases where this technique is not available simple mathematical correlations with the UV-254 coefficient can be used. Several portable field techniques are available for measurement of UV signals. This configuration is much cheaper. However in this case an exact individual calibration for each source of water is necessary.

Formation of odor

One of the most aggravating side effects of water disinfection with chlorine is the formation of musty and moldy/septic odor. This is especially the case when the residual chlorine during distribution of the water decreases. The odor threshold concentration of those organoleptic by-products is much lower than that of chlorine which is always connected with a fresh impact. On the other hand, if chlorine reacts with nitrogen-containing organic compounds an intensive organoleptic odor is possible. The sources of N-containing organic matter are manifold. Most relevant for the River Nile is the algae growth. The cells of the algae consist of peptides and biopolymers such as proteins. The components of these materials are amino acids, which are well removed during RBF, causing less odor problems.

The removal of organic matter dissolved in water is one of the most promising actions for improvement of the disinfection efficiency and the reduction of DBP formation. Organic matter in water means humic compounds (in most of the cases). These organic macromolecules are non- toxic for humans. Nevertheless, they form DBPs if chlorine is used. RBF is known for its high efficiency in NOM-removal, which can be expected in the range between 30 to 50%. Otherwise, the removal of humic compounds is expensive. Today, activated carbon application is common and more and more membrane filtration is used in modern facilities.

5 Operation and maintenance of RBF schemes

5.1 Operation principle for well scheme (continuous vs. discontinuous)

RBF wells should be operated continuously to avoid changes in mixing portions of bank filtrate and groundwater and thus water quality. Especially if anoxic conditions occur and post-treatment is required to remove iron and/or manganese, continuous well operation is of advantage. Operation should be monitored by frequent readings of water levels in the well and total volume of water abstracted (water metering).

RBF schemes with more than 2 wells have the advantage of being able to compensate for short-term failures of individual wells. However, this is only valid if the wells are situated close to each other. Modeling results show that switching off one well in a well gallery of 3 wells (distance to each well 10 m) the groundwater flow conditions hardly change, if the shutdown is only for a few hours. The lowest effect on groundwater flow was observed when switching off the central well of a well gallery, as the neighboring wells can compensate for the loss in groundwater flow. By switching off one outer well, the flow times of the bank filtrate are extended significantly, if pumping is stopped for more than 2 days. Switching off several RBF wells at the same time should be avoided as the travel times of the bank filtrate will be greatly extended, which increases the risk of stronger anoxic conditions in the subsurface passage.

From a practical point of view, maintenance work on individual wells can be carried out without big impacts on water quality as long as it is limited to one working day. A power failure and the resulting short-term failure of all pumps for a few hours do not affect groundwater flow conditions to great extent. However, it is essential to avoid long periods of disabling all pumps.

Submersible pumps are designed and manufactured to be operated continuously over weeks without problems. Therefore, it is unnecessary to switch them off (to “rest” them). As some waterworks in Upper Egypt have been facing problems with breakdowns of submersible pumps, the reasons should be identified. At a few sites submersible pumps have been replaced by other pumps installed above ground, which - according to local staff reports - can run only 16-18 hours a day. Again, such pump operation is questionable and not common. Thus, a problem discussion including an assessment of pump selection, operation and maintenance is required to develop an appropriate training and guidance for technicians.

5.2 Maintenance requirements

Apart from the installation of any RBF system, some form of routine maintenance is needed to ensure the system does not incur any significant reduction in performance. Some wells require regular mechanical cleaning. Intensive pumping or compressed air bubbles may be used for vertical well systems. Table 5-1 provides a summary of the routine and non-routine maintenance requirements which may or may not be applicable to any given situation.

Tab. 5-1: Maintenance requirements

Routine maintenance	Non-routine maintenance
Well yield test	Clean well by pumping/ air compressor/ manually
Service of pump sets	Replace pump sets
Inspect water quality	Clean the gravel pack using hydraulic and chemical well rehabilitation methods
Flushing pipeline system	Corrosion prevention

Pumps should be inspected regularly, at least annually. The abstraction pipe and pump have to be removed from the well and cleaned. Precipitates should be removed, the pump inspected and wearing parts eventually replaced. Inspection frequency should be determined based on regular energy measurements (kWh/m^3 pumped water) and measurements of bottom level of wells to check if there is any sand intrusion. If the sand trap is filled by more than 0.5 m within a year, a well inspection using a well camera is recommended. Before re-installation of the pump, any oil/grease should be removed from pumps, pipes and fittings, no oil should enter the well.

When considering maintenance, it is well known that it is frequently ignored until the failure occurs. Such crisis maintenance is likely to be expensive as well, as it may require re-excavation and cleaning of much of the intake portion of the well. However, those personnel who do regular maintenance will recognize that regular maintenance will prolong the life of the system. Recurring costs includes replacement costs, maintenance and repair costs.

5.3 Monitoring concept: data collection and analysis

A monitoring system should be introduced for all RBF sites in order to react quickly to changes in water quality or well ageing processes. Quality & Environmental Affair Department in coordination with both the Water Reference Laboratory at HCWW and the Affiliated Companies shall take the responsibility for proper monitoring of all RBF units at the governorates. They will be also responsible for providing the required technical support to the staff to ensure proper operation and and maintenance of the RBF Units. Special attention should be paid to recording a few meaningful parameters capturing the many different factors affecting the RBF scheme. Framework conditions for the measurement of these parameters must be defined to keep the parameters comparable. To control the efficiency of the wells the specific drawdown, as the ratio of drawdown and abstraction rate, can be used. If the specific drawdown increases in the time series, this indicates a decrease in well performance. Timely and cost-effective well rehabilitation measures can thus be initiated. In addition, monitoring of the filter resistance (difference in water level between the inner and outer piezometer) is advised, especially due to the iron and manganese concentrations in the groundwater of Upper Egypt. Based on comparable abstraction rates, the filter resistance is a good indicator for the condition of filter screen and filter package of the well.

Regular measurement of the abstraction rate could be used to document an increase in pump performance in the event of a drop in well performance. Furthermore, based on the given boundary conditions (pump specification, well construction, geology), limit values for minimum and maximum abstraction rates can be defined.

A frequent assessment of the hydraulic conductivity of the riverbed would be useful due to its great influence on the RBF system. But a practical implementation is difficult as the measurement methods require high technical and time effort.

The evaluation of the Nile River water quality parameters revealed a periodic oscillation of the electrical conductivity. Since this was also observed in the water of RBF wells in Upper Egypt, a permanent EC monitoring is suited for RBF sites in Egypt. Thereby travel times from the river into the wells as well as the portion of the bank filtrate of the total abstracted water can be determined precisely.

Since intermittent well operation affects groundwater flow conditions, the shutdown processes should also be recorded. This can be done most easily by measuring the water level in the wells, using electrical data loggers. This makes it possible to check whether the fluctuations in manganese or iron concentration of the abstracted water can be directly linked to the switch-off processes. This, of course, requires a reliable and regular determination of the water quality. The collected data should be stored in digital form to ensure subsequent analysis.

5.4 Operation of post-treatment facilities (Fe/Mn removal, disinfection)

Post-treatment units to remove Mn and/or Fe should be operated continuously to prevent breakthroughs and to achieve maximum treatment efficiency. Continuous monitoring of pressure differences within filters is a precondition for timely start of backwashing. Conducting backwash cycles and defining procedures (water, air/water, backwashing time in min, etc.) should be done based on local experience and on-site investigations. Optimum operation of filters depends mainly on local water quality, possible filter velocities and filter design (material, single/dual layer filtration, geometry of filter unit, filter heights). If there is limited experience in operation of filters, regular inspection by an experienced engineer and a special on-site assessment using on-site analytical methods is advised. If filter material is filled into filter units for the first time or has to be replaced, at least 10% of used material from another filter should be added/mixed to take advantage of autocatalytic effects for Mn removal. During the initial phase of filter operation or periods with higher demand, addition of $KMnO_4$ before filter input may help to achieve greater Mn removal. Backwash water should never be spilled or discharged in to the river or river bank in front of the RBF wells because this will enhance riverbed clogging and lower abstraction of bank filtrate. Filters should be operated to remove as much Mn and Fe as possible, even below the required thresholds to prevent settling of particles in the network and turbidity peaks due to remobilization during periods of high flow velocities in the network.

Continuous drinking water disinfection has to be ensured all the time and adjusted to the actual demand / discharge to prevent overdosage and resulting corrosion. Handling of disinfectants, such as chlorine and chlorine dioxide, is dangerous and requires strict work safety procedures. Thus, maintenance should not only focus on dosing pumps but also on leakage prevention and control measures and work safety.

5.5 Risk assessment

When considering managed aquifer recharge (MAR), generally in many cases the recharge water (water induced into an aquifer) is of comparable quality to water already in the aquifer and enhancing recharge may improve the quality of groundwater, for example by freshening brackish groundwater (Dillon et al. 2016). However, RBF may also introduce microbiological or chemical pollutants to aquifers, or mobilize minerals from the aquifer matrix, which may be harmful to human health or adversely affect the aquatic environment. If the same aquifer being recharged is also used as a drinking water source, it should be an obligation of those enhancing recharge to protect the health of those whose drinking water is affected by their operations (Dillon et al. 2016). And accordingly, when control measures are followed intentionally to protect human health and the environment, it is called "managed aquifer recharge". In this context, a risk assessment means the application of a common holistic framework, which provides a staged approach to assess the highest priority hazards commonly encountered in RBF operations and provides a scientifically-founded basis for further risk management plans.

Where the intention is to use RBF for drinking water production and in addition to assessing the risk from chemical parameters, it is necessary to focus on a thorough assessment of the potential pathogen risks to human health using quantitative microbial risk assessment (QMRA; Page et al. 2010). An example is given in Tab. 5-2. Microbial risks are more acute compared to chemical hazards, because the latter may cause subtle, chronic health effects on account of their potential toxicity, carcinogenicity or being suspected endocrine disruptors (Schwarzenbach et al. 2010). Considering the above, a risk assessment and management plan for a RBF system should cover the following aspects:

- RBF removal efficiency for pathogens,
- disinfection as necessary post-treatment to safeguard against microbial risks,
- risks from floods and mitigation measures at RBF schemes.

Tab. 5-2: Risk-assessment stages (modified only for RBF from Dillon et al. 2014)

1. Simple assessment (answer with "yes (Y)" or "no (N)")
a. Is the aquifer, from which RBF wells are / will be abstracting water, is or will be used for drinking water supply? **Y** b. Is the scale of RBF larger than domestic rainwater harvesting? **Y** c. Does the source water (river) contain sewage effluent, industrial wastewater or urban stormwater? **Y** d. Is the area around the recharge area ever waterlogged? **N** *Simple assessment is satisfied if all answers are "No". Then no need to continue assessment because there is a low inherent risk. However if any answer is "Yes" proceed to Viability assessment.*
2. Viability assessment (Y/N)
a. Is there a sufficient demand for water? **Y** b. Is there an adequate source of water available for RBF? **Y** c. Is there a suitable aquifer for RBF? **Y** d. Is there sufficient space available for RBF and eventual post-treatment measures? **Y** *If the answer to any question is "No", then the project is not viable or else it has a major constraint. If answers are "Yes", then proceed to Guidelines applicability assessment.*
3. Guideline applicability assessment (Y/N)

a. Is the source of water for RBF only from a natural catchment including rural / overland runoff with less human activity and / or snow melt (i.e. not much affected by sewage effluent, industrial wastewater or urban stormwater)? **Y**

b. Is the aquifer unconfined and not polluted? **N**

If answers are "Yes", then monitor / compare quality of water from RBF wells to Egyptian water quality standard and if any parameters exceed permissible limits, then consider post-treatment measures

4. Sanitary survey (Y/N)

a. Is there a latrine (unsealed, open-pit), open sewer or leaky sewer or human (open defecation) or animal faeces within the catchment area of the RBF wells? **Y**

b. Is there a latrine, open sewer, leaky sewer or animal faeces in close proximity to the RBF wells? **Y**

c. Are there industrial, transport or agricultural activities generating stockpiles, wastes, spills or emissions reaching the surface of the catchment area of the RBF wells? **Y** (only road / petroleum transport)

d. Are there industrial, transport or agricultural activities generating stockpiles, wastes, spills, or emissions in close proximity to the RBF wells? **Y** (road / petroleum transport, wastes)

e. Is there post-treatment of water to be recovered? If so describe its design and resilience to power and mechanical failure, and any alarm systems. **Y** (only chlorination)

f. Does the existence and condition of any barriers around of the RBF wells prevent short circuit of contaminated water? **Y** (concrete base around wells; short circuit may occur if crack / fissures in base)

*Any question answered by **Yes** needs to be taken into specific account in the Water Safety Plan. Even if not observed, the possibility of these hazards occurring or barriers being breached also needs to be taken into account. Proceed to Aquifer assessment.*

5. Aquifer assessment (Y/N)

a. Does source (river) water have low quality; is water turbid, coloured, contains algae, has a surface slick or does it smell? **Y** (high turbidity periods, presence of pathogens year-round in River Nile)

b. Does the unconfined aquifer have a shallow water table, say < 8m in urban area and say <4 m in rural area? **N** aquifer is confined

c. Are there other groundwater users, groundwater-connected ecosystems or a property boundary within 100 m of the recharge site? **Y** (a few private wells near some RBF wells)

d. Is the aquifer known to contain reactive minerals (e.g. pyrite) or is groundwater in this area known to contain arsenic? Does the aquifer contain soluble minerals such as calcite and dolomite? **N**

e. Is the aquifer composed of fractured rock or karstic (fissured or cavernous) limestone or dolomite? **N**

f. Is the proposed project of such a scale that it requires approval? Is it in a built up area; built on public, flood-prone or steep land or close to a property boundary? Does it contain open water storages or engineering structures; or is it likely to cause public health or safety issues (e.g. falling or drowning), nuisance from noise, dust, odour or insects (during construction or operation), or adverse environmental impacts? *(In this case the RBF scheme already exists and is operational)* **Y**

*Any question answered by **Yes** needs to be taken into specific account in the Water Safety Plan below. Even if not observed, the possibility of these hazards occurring or barriers being breached also needs to be taken into account.*

The timely response to risks affecting a RBF scheme is important to guarantee the supply of potable water even during emergency situations. Responses can be framed into a WHO-based water safety plan (WSP) that is specific for each RBF site and which should be implemented prior to more engineered post-treatment options. Such WSPs address preventive measures to control general pathogen-related risks in the catchment and in the treatment process (Tab. 5-3).

Tab. 5-3: Measures to control general pathogen-related risks in well-catchment and treatment process taking the example of an RBF scheme in India (Bartak et al. 2015b)

Measure	Hazard arising from	Identified specific improvement plan	Time frame
Well sanitation	Leakage from pipe joints and valves, insufficient sanitation, well head housing and use as public washing place	Clean well heads (house) Waterproof the floors and base around the vertical line shaft pumps in the well house to prevent ingress of foreign matter into the well-caisson Prohibit defecation, housing, public washing and cattle	Immediately
Well protection I	Unrestricted accessibility	Restrict access to wells (house) Liaise with landholder about security of premises Fence or buffer zone around well head	Immediatelyto short
Well protection II	Wet season and flooding Ingress	Install water-tight covers on entrance hatches on top of well-caisson and their regular maintenance; Improve well head seal (e.g. WHO/WEDC 2011)	Short to medium
Ground-water protection	Unsanitary defecation in well catchment zone Use of unsealed pit latrines	Liaise with municipalities to improve design of existing pit latrines or design and implement alternative solutions	Medium
Source water protection	Partially treated sewage discharge Untreated stormwater run-off	Liaise with municipalities Reduce discharge of partially treated wastewater and treat storm water run-off	Long
Improve disinfection	Requirements for residual free chlorine >0.2 mg/l not met within the distribution system Insufficient disinfection	Improve disinfection (residual chlorine >0.2 mg/l) Increase chlorine contact time Add additional chlorine injection points within the network	Medium
Well management	Not existing	Investigate well performance Rehabilitate wells Implement well operation philosophy	Medium Medium
Improve distribution network	Unrestricted tapping, insufficient pressure head Wastewater ingress	Liaise with municipalities Increase pressure, reduce unaccounted for water Increase awareness to minimize wastage (open/ dripping taps)	Long

5.6 Training of staff

Staff should be trained according to their major tasks and responsibilities.

Capacity building at the level of engineers, geologists and chemists from affiliated companies of HCWW should include a basic understanding of hydraulics of RBF, water quantity and quality monitoring requirements, well construction, operation and maintenance, and well head protection measures. Suggested training could include the application of templates and Excel spreadsheets to check the status of RBF wells by analysis of water levels, drawdown, water abstraction rates, energy consumption, and water quality data. For engineers, online-tools could be developed in addition to regular training courses that including hands-on evaluation and calculation exercises.

Capacity building at the level of technicians and un-/semi-skilled workers should include awareness creation about importance of regular and precise measuring and documentation of abstraction volumes and water levels, documentation of operational schedule, careful use and maintenance of tools and measuring devices (e.g. dippers for water level measurement, water flow meters), and well head protection measures.

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