ISMAR7 Abu Dhabi
www.ismar7.org
October 9-13th, 2010

ACHIEVING GROUNDWATER SUPPLY SUSTAINABILITY & RELIABILITY THROUGH MANAGED AQUIFER RECHARGE

PROCEEDINGS OF THE SYMPOSIUM ISMAR 7

ISMAR 7. Proceedings of the symposium

Achieving Ground Water Supply Sustainability & Reliability through Managed Aquifer Recharge

International Symposium on Managed Aquifer Recharge

Abu Dhabi (UAE) 2009 Oct 9-13th

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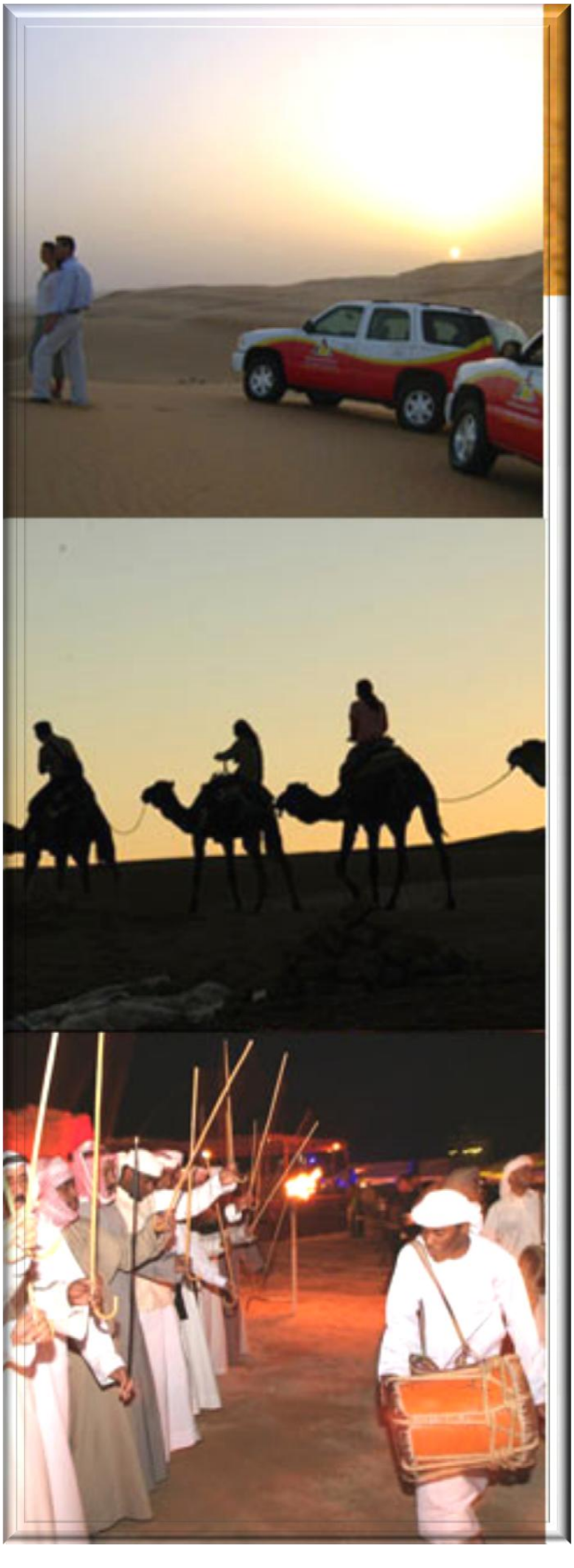

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Introduction

The ISMAR conference series was born in August 1988 in Anaheim California as the 1st International Symposium on Artificial Recharge of Ground Water. Pioneered by the American Society of Civil Engineers (ASCE), it continued in Orlando, Florida (1994) before the International Association of Hydrogeologists (IAH) partnered ASCE in Amsterdam (1998). Following the 4th Symposium in Adelaide (2002) the name changed to International Symposium on Managed Aquifer Recharge to reflect the growing scientific basis supporting overt management of quantity and quality of recharge, and reflected the name of the IAH Commission on MAR, which was established in 2002. ASCE has also adopted this name for its relevant standards committee. Dedicated to a global reach, recent ISMAR conferences have been ISMAR 5 (Berlin, Germany, 2005) and ISMAR 6 (Phoenix, USA, 2007). ISMAR 7 was hold in Abu Dhabi, U.A.E., October 9 - 13, 2010.

The Symposium continued as a joint venture of IAH/ASCE and is the prime international meeting in this field, adopting the Commissions aims, "to expand water resources and improve water quality in ways that are appropriate, environmentally sustainable, technically viable, economical, and socially desirable. It will do this by encouraging development and adoption of improved practices for management of aquifer recharge," IAH, January 2002.

This book collects all the presentations revised by pairs and exposed at the symposium.

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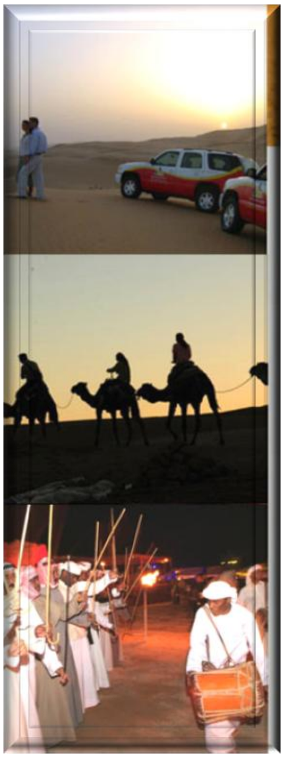

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Areas of learning

In alphabetic order:

- Alternative Recharge Systems
- Case Studies
- Data Management/Monitoring/Automation
- Enhancements in Aquifer Characterization
- Fate of Pathogens/Organics
- Geochemistry during Infiltration and Flow
- Injection and Pond Infiltration Issues
- Integrated Water Management and Strategies
- MAR in Arid Environments
- Modeling and Groundwater Hydraulics
- Operations and Management
- Regulations/Governance/Economics
- Risk Assessment and Uncertainty
- The Role of MAR in Climate Change
- Water Reuse and Managed Aquifer Recharge (MAR)


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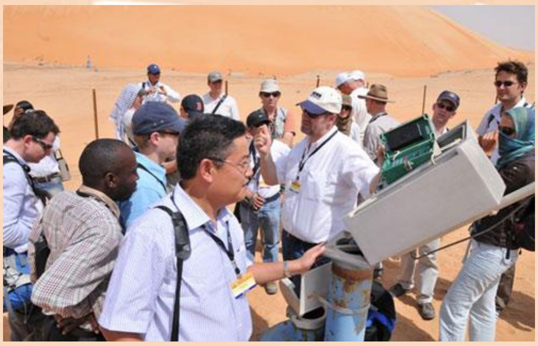

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Influence of aquifer properties on water quality changes during infiltration of treated effluent

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Abstract

The study aims at better understanding the impact of soil and aquifer properties on transport of micropollutants present in treated wastewater during pond infiltration at a test site close to Thessaloniki, Greece. The subsurface is mainly composed of Neogene limestones, sandstones and conglomerates overlain by Pleistocene and Holocene alluvial deposits, and unconfined groundwater conditions are prevailing. At the test site, one infiltration pond (18 m X 9 m X 2 m) was constructed to perform a pilot infiltration study. The work comprised hydrogeological site characterization combined with comprehensive analysis of the water quality data, both at the treated effluent outlet (before infiltration) as well as in monitoring wells after infiltration. Hydrogeological investigations revealed a surface silty layer with a thickness varying between 0.5 – 1.3 meters, a heterogeneous sand body with silt/clay lenses of about 6 – 7 meter thickness, and below this fine sand with silt fractions. Dominating mineral component in zones of relative high hydraulic conductivity is Quartz. The content of C_{org} in zones of relative high conductivity is relatively low. The hydraulic conductivity ranges between 2.0×10^{-3} m/sec and 9.0×10^{-4} m/sec. A groundwater flow and transport model has been set up using Visual Modflow software. Illustrative transient flow and reactive transport scenario simulations have been conducted in a two dimensional vertical cross sectional modeling domain with saturated conditions. The model results show that the aquifer is suitable for infiltration of treated effluent. The model is therefore an important tool to quantify the degree of attenuation of the micropollutants present in the treated effluent and can help to recommend and design further treatment steps before recharge into the subsurface at the test site. The model will be used for further determination of pond operation scenarios and related clogging effects, considering water quality changes during infiltration at the test site.

Key words:

Groundwater artificial recharge, infiltration pond, mathematical modeling, transport processes

INTRODUCTION

Managed Aquifer Recharge (MAR) with reclaimed wastewater and other sources of water is now being widely practiced in various parts of the world, especially in the arid and semiarid regions. Implementation of surface spreading basins (i.e. Soil Aquifer Treatment, SAT) is now a common practice for MAR (Drewes, 2009). SAT is an economical and aesthetic wastewater reuse approach. Since the soil and the aquifer can act as natural filters, SAT systems can remove suspended solids, biodegradable materials, bacteria, viruses, and other microorganisms (Bouwer, 1997).

During SAT, secondary or tertiary treated wastewater infiltrates into the subsurface from an infiltration basin, which continues to percolate through the unsaturated zone and then finally mixes with native groundwater. Percolation of water through the soil column involves several processes in the vadose zone. At the basin–soil interface, the combined effect of sedimentation, filtration, aeration, and microbial growth lead to the formation of a biologically active zone that may be impermeable (Bouwer and Rice, 1984). Due to the formation of this biological "mat," infiltration rate may become reduced with time.

The impact of soil types and aquifer properties on SAT operation has been examined by a few studies. Removal of organics does not depend on soil type, though fine-grained soil has a small advantage compared to other soil types (Quanrud et al., 1996). Later on, a review study by Sharma et al., 2008 suggested that the soil type might have impact on dissolved organic carbon (DOC) removal. Sandy loam has better DOC removal efficiency than others. The influence of aquifer properties on endocrine disrupting compounds (EDC) removal has not been well investigated.

The wastewater treatment plant of Thessaloniki, the second largest city of Greece, treats 180,000 m^3 of wastewater per day with all of the effluent being discharged direclty into the Thermaikos Bay. The continuous operation of the sewage station during the last years has had a strong negative impact on the quality of the seawater in the Thermaikos bay (Soupilas et al., 2009). Marine wildlife has directly suffered as a consequence. Not only is environmental degradation a major issue here, but the degradation of fresh water without possible recycling can be seen as the loss of a significant water resource. Environmental degradation and the waste of a significant water resource are reasons for why now the responsibles for water supply and sewage teatment in the city have decided to check the feasibility of using the aquifer as a storage for treated effluent from the wastewater treatment plant. A study has been performed to check the viability of MAR applied to the aquifer at the city of Sindos, close to Thessaloniki, using secondary treated wastewater. Test site characterization, infiltration tests, field campaign for the understanding the fate of certain emerging pollutants, mathematical modeling to simulate the flow and transport processes within the aquifer have been performed. This paper will focus mainly on the field investigations and groundwater modeling to understand the sub-surface system within the area, which has a great influence on the water quality changes during infiltration of the treated wastewater.

DESCRIPTION OF THE TEST SITE

The test site, having an area of 2 sq. km and located south of the city of Sindos, belongs to the municipality of Sindos. Based on observations from satellite images and topographic maps, the site is most likely on an old point bar of the Sindos River, which lies between the main channel of the Sindos River and an old paleochannel.

Figure 1: Location of the test site and of the wastewater treatment plant (Source: Google Earth)

The Sindos aquifer is attributed to the prograding deltas of Axios and Gallikos Rivers respectively. The subsurface is mainly composed of neogene limestones, sandstones and conglomerates overlain by pleistocene and holocene alluvial deposits. The aquifer system extends to depths between 30 m to 120 m. The Sindos aquifer has been providing water for urban and industrial use in the major city of Thessaloniki. Since 2003 the aquifer is no longer exploited for urban and industrial purposes. The Sindos aquifer is unconfined and characterized by a large degree of heterogeneity, as it is located within the zone of meandering channels of the Axios and Gallikos Rivers. Aquifer permeability is relatively high, ranging between $6.7X10^{-4}$ m/sec and $2.55X10^{-3}$ m/sec (GABARDINE, 2008).

MATERIALS AND METHODS

Test site characterization and experimental set-up

For better understanding of the local subsurface stratigraphic and hydraulic conditions, three drillings (D1, D2 and D3, in Figure 3) were done in a triangular area spanning a distance of 50 – 100 meters between them. Undisturbed samples were retrieved at one location. After drilling, three piezometers (P1, DP2 and DP3) were installed.

During the drilling campaign, 33 undisturbed soil samples were collected from drilling D1 and analysed for soil chemical and physical properties. Detailed grain size analysis was performed, too. In addition, the mineral composition of select soil samples was investigated in the laboratory.

Two experimental infiltration ponds were constructed to both sides of piezometer P1. The experimental infiltration ponds are 18 meters long, 9 meters wide, and have a depth of 2 meters each. Bed surface area of each pond is 10 m X 5 m (Figure 2).

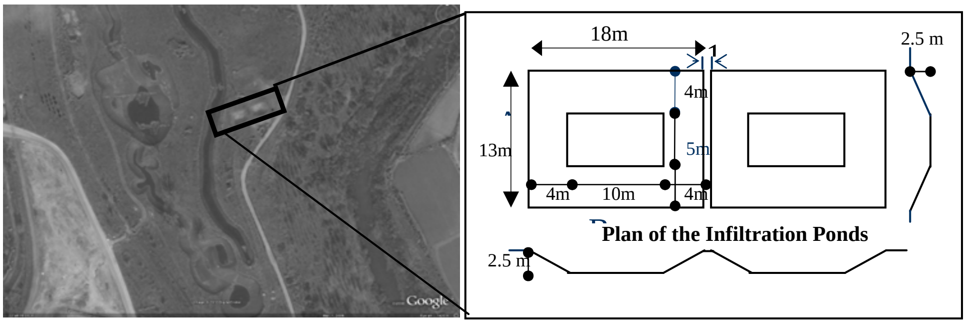


Figure 2: Location of the infiltration ponds (left) and their detail dimension (right) (Source of left image: Google Earth)

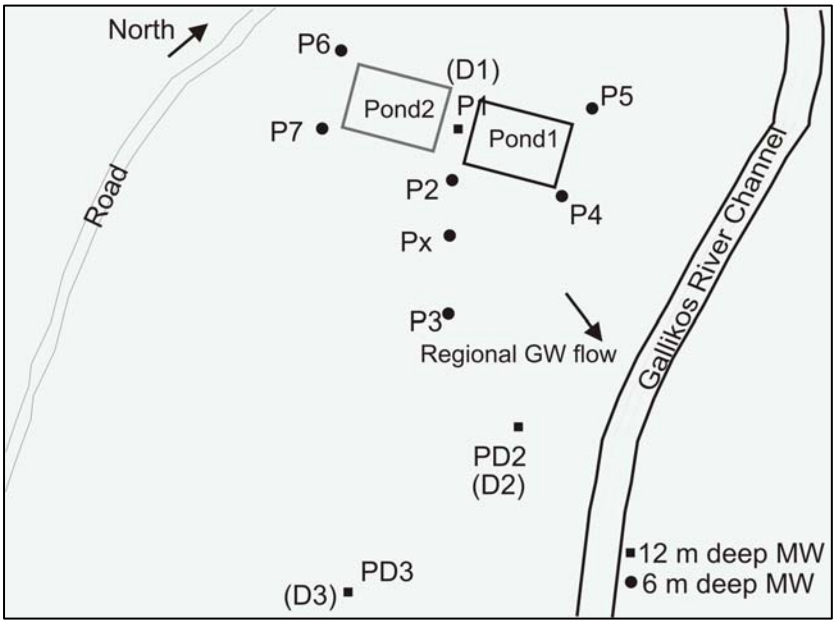


Figure 3: Experimental set-up showing the position of the piezometers relative to the ponds. Pond 1 was used for the experiment

Figure 3 shows the monitoring network layout. Three monitoring wells are 12 m deep and six wells are 6 m deep.

Soil column tracer study

Undisturbed soil samples were collected from the infiltration pond bed in a stainless steel cylinder. The volume of the cylinder is 1041 cm^3 (height=15 cm, inside diameter= 9.4 cm). In the soil column testing apparatus, stainless steel porous plates was placed between the soil and two stainless steel end caps. The porous plates aimed at avoiding washing out of soil fines. Only glass, Teflon and stainless steel were used for the construction of the experimental apparatus to minimize adsorption. Figure 4 shows the simplified layout of the soil column experimental setup.

The soil column was slowly wetted, from the bottom to the top, over a 36-h period with a velocity of 2 ml/min using deionized water. This was done to reduce the amount of entrapped air, to saturate the soil, to condition the column, and to maintain the soil structure.

After conditioning of the soil, an input solution containing two conservative tracers, bromide and chloride, was applied to the soil, from the bottom to the top. A constant flow of 2 ml/min was maintained throughout the experiment. The initial concentrations of chloride and bromide were 99.334 mg/L and 99.108 mg/L respectively. The effluent fraction was collected every 10 min using an automated fraction collector.

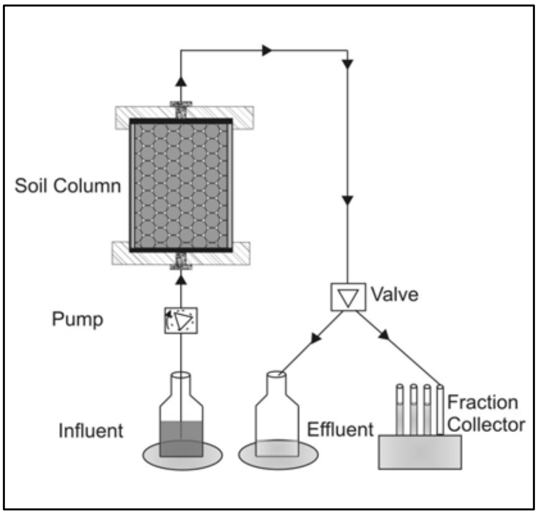


Figure 4: Layout of soil column experiment

The stock solution and the effluent fractions were analysed for chloride and bromide. Bromide and chloride were measured using ion chromatography following standard procedures.

Mathematical modeling

1D modeling of the soil column experiment:

The computer code Studio of Analytical Models for solving the Convection Dispersion Equation, STANMOD, version 2.2, was used to model the tracer experimental data (Simunek et al., 1999). The program uses CXTFIT 2.0 code for estimating transport parameters from laboratory or field tracer experiments. An inverse modeling technique has been used to fit an analytical solution to the observed data in order to estimate transport parameters (e.g. Toride et al., 1995).

2D modeling of field experiment:

A groundwater flow and transport model has been set up using Visual Modflow software. The model involves a two-dimensional cross sectional domain (150 m x 12 m). The horizontal grid size ranges from 0.25 m to 1 m with refined grids around the pond perimeters. The geological layers, along the z axis, have been discretized by using the bore log information. The groundwater flow is from left to right, according to the regional groundwater flow within the test site. Boundary conditions for flow are a constant head on the right boundary and no flow boundaries for the left, bottom, and upper boundaries. At the pond bed surface, a time variable flux boundary has been used. The transport boundaries at the left, bottom, and top surface are no flux boundaries. At the pond, a point source concentration boundary has been used. The flow, transport, and reaction parameters came from the results of the analyses of collected soil samples, laboratory analysis, output of the 1D model and scientific literature. The model has been calibrated using data observed at the test site. Diazepam, used as a tranquilizer, has been reported to exist in the treated wastewater at the Sindos wastewater treatment plant. Therefore, the transport of Diazepam has been simulated using the calibrated groundwater model.

RESULTS AND DISCUSSION

Field investigations

The resulting stratigraphy reveals a surface silty layer of varying thickness between 0.5 – 1.3 meters, a heterogeneous sand body with silt/clay lenses of about 6 -7 meter thickness, and below lies fine to medium sand with minor silt fractions present. During drilling, a series of infiltration tests were performed at different depths to provide a first characterization of the infiltration capacity of the sand layer, which was found to be high enough for the construction of the infiltration ponds. The historical piezometric level confirms 2-2.5 m unsaturated zone, which may be useful for further investigation of unsaturated zone behavior during infiltration.

The grain size distributions show that the soil particles are uniformly distributed in the subsurface. From the soil texture triangle, it can be concluded that the soil sample is sandy loam. The three main hydrofacies of the samples are fine sand, medium sand, and silt. The hydraulic conductivities at different depths, calculated using an empirical formula, range between 2.3x10-8 m/sec and 1.4x10-4 m/sec. From the sediment material composition analysis, it has been found that quartz mineral grains are dominant in the zone of high hydraulic conductivity (Table 1). The samples contain low percentages of organic matter. Total organic carbon in the relatively high conductivity zone is relatively low (Table 2).

Table 1: Mineral composition of soil sample 10 (depth 3.65 to 4 m below ground level)

Description	Percent
Quartz	80
Muskovite	2
Mufites	8
Clay minerals	1
Carbonate	No
Fe Oxides	2
Undefined	7
Gravel:Sand:Silt:Clay of the sample is 5:80:13:2 (in %)	
Hydraulic Conductivity is $2.8X10^{-5}$ m/s	

Table 2: Organic content in soil samples at different depths

Sample No	Depth from the surface (in m)	Percent of Sand, Silt and Clay	C_{org} (Total)
1	0-0.60	78:21:1	0.2
12	4.60-5.00	66:31:3	<0.10
14	6.00 – 6.25	91:8:1	<0.10
28	9.5-10.00	47:43:10	1.1

Soil column study

The breakthrough curves (BTCs) of chloride and bromide are shown in Figure 5. Chloride and bromide ions were transported through soil columns basically by a convective-dispersive process.

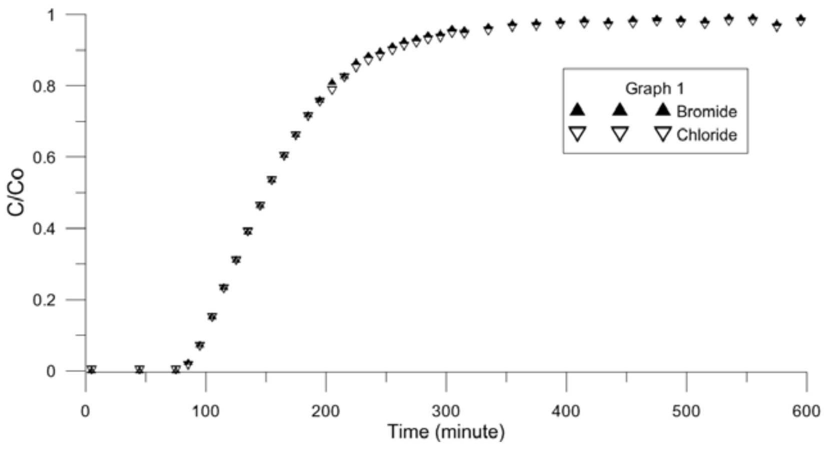


Figure 5: Breakthrough curves of chloride and bromide in the soil column experiment

1D modeling of the soil column experiment:

Figure 6 shows the inverse analytical modeling output obtained from the application of STANMOD to the breakthrough curves. Transport velocity, dispersion coefficient, effective porosity, and dispersivity are presented in Table 3. A dispersivity of approximately 1 cm is observed in the laboratory column. The transport velocity ranges from 1.53×10^{-5} - $1.54 * 10^{-5}$ m/s. The calculated average effective porosity is 29%.

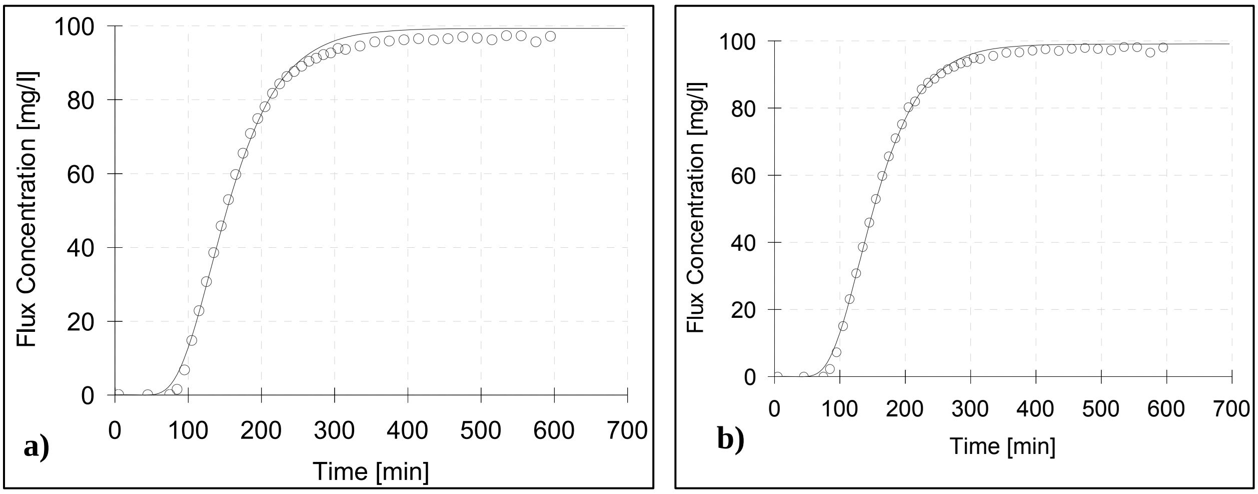


Figure 6: Model output of tracer experiment: a) chloride b) bromide

Table 3: Estimated flow and transport parameters

No	Item	Chloride	Bromide
1	Darcy Velocity (m/s)	4.5×10^{-6}	4.5×10^{-6}
2	Transport Velocity (m/s)	1.531×10^{-5}	1.546×10^{-5}
3	Effective Porosity	29.37%	29.09%
4	Dispersion Coefficient (cm^2/min)	0.104	0.107
5	Dispersivity (cm)	1.08	1.07

2D Flow and transport modeling:

The groundwater flow model has been calibrated using the observed groundwater level data at a well situated 15.5 m distance from the left side boundary of the model domain. Two highly conductive layers below the pond bed surface have been identified. The calibrated model has been used to determine the optimal values of the three main parameters of pond operation, namely pumping rate, injection time, and drainage time. The application of water with a volumetric flow rate of 315 m^3/d for 30 days will fill the pond and cause no overtopping. About 8 days is required to drain the pond totally (Figure 7).

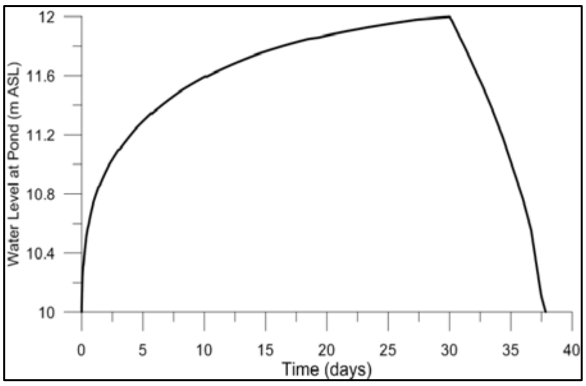
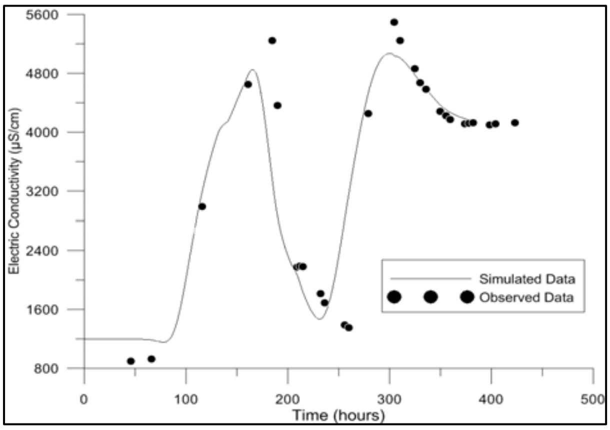


Figure 7: Pond water level showing the wetting and drainage cycle for optimum pond operation

The transport model has been calibrated using the observed electric conductivity data at a well situated 15.5 m distance from the left side boundary of the model domain (Figure 8). The calibrated model has been used to simulate the transport of Diazepam within the aquifer (Figure 8). The resulting K_d value of Diazepam is 0.12 L/kg, under the prevailing site conditions. From K_d, a log K_{ow} value of Diazepam has been calculated to range between 2.65 and 2.85 (considering an f_{oc} equal to 0.04%), using the formula proposed by Karickhoff (1981), Scharzenbach and Westtall (1981), and Karickhoff et al, (1979). The resulting log K_{ow} values comply with the values reported in literature for Diazepam (e.g., Carballa, 2005)



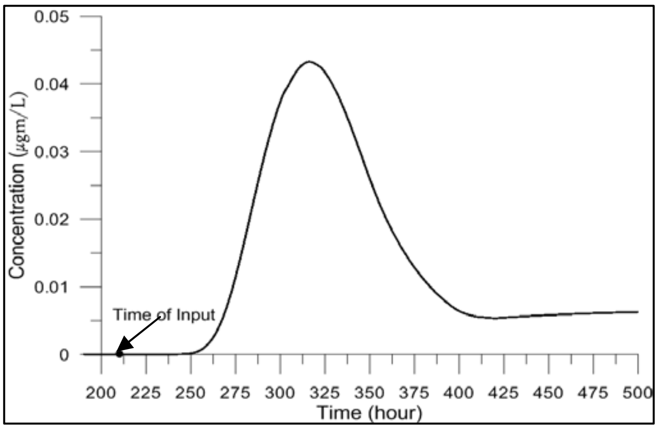


Figure 8: Simulated BTC curves of electrical conductivity (left) and Diazepam (right)

CONCLUSIONS

The site investigation has revealed that the test site is comprised of a highly permeable sandy aquifer with high hydraulic conductivity. The presence of an unsaturated zone is advantageous for installation of infiltration ponds. The high hydraulic conductivity layers have low organic carbon, resulting in lower retardation of pollutants. Groundwater flow and transport simulations were performed to determine the optimal pond operation and to investigate pollutant transport within the aquifer. The simulation results show that for the test installation the optimal pond operation is 30 days wetting and 8 days drainage without considering clogging of the pond bed. In fact, no clogging of pond surface has been observed during 30 days of field testing. The transport model describes the fate and transport of Diazepam during the pond operation. The model results show that the aquifer is suitable for further cleaning of treated effluent over time. The model is therefore an important tool to quantify the degree of attenuation of the micropollutants present in the treated effluent and can help to recommend and design further treatment steps before recharging the effluent into the subsurface at the test site. The model will be used for further determination of pond operation scenarios and related clogging effects, considering water quality changes during infiltration at the test site, and to assess the attenuation of other pollutants observed.

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The Orange County Water District Riverbed Filtration Pilot Project: Water Quality and Recharge Improvements Using Induced Riverbed Filtration

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Abstract

In an effort to reduce suspended solids and organic carbon loading and to increase long-term groundwater recharge rates at Orange County Water District's spreading basins, a pilot project was conducted to evaluate riverbed filtration as a technology to treat river water prior to groundwater recharge. A shallow under-channel lateral drain system was constructed within a channel adjacent to the Santa Ana River to induce and capture infiltration. Water pumped from the drain system was analyzed for a variety of water quality parameters and then recharged into percolation columns to evaluate recharge rates compared to raw Santa Ana River water (without treatment). At the pilot project drain system, phreatic surface and temperature were continuously monitored at thirteen points. River water inflow and outflow and drain system pumping rates were also monitored.

The pilot test was divided into two periods: Period 1 had shallow overflow (3- to 8-cm) within the river channel; Period 2 achieved deeper surface water depths (8- to 30-cm). Lateral drain system pumping during both test periods were incrementally increased to establish the maximum pumping capacity of the drain system for each test period. Monitoring data indicate that riverbed filtration effectively removed essentially all suspended solids and reduced turbidity with the bulk of water captured by the under-channel drain system from induced infiltration. The phreatic surface and subsurface water movement within the drain system area was shown to be very sensitive to changes in surface water flow rates and depth, and drain system pumping rates. In addition, surface clogging was observed. The pilot project results indicate that riverbed filtration is a viable technology for treating surface water prior to recharge operations, however, additional testing and optimization is needed.

Keywords

Recharge, Filtration, Infiltration, Percolation, Water Quality, Total Suspended Solids

INTRODUCTION

The Orange County Water District (OCWD) located in southern California, USA is responsible for managing the local groundwater basin that supplies water to more than 20 cities and water agencies, serving more than 2.3 million people. OCWD primarily recharges the groundwater basin by applying water from the Santa Ana River (SAR) to over 600 hectares of surface spreading basin recharge facilities. SAR flows are primarily comprised of tertiary-treated effluent during the dry season and storm water during the winter rainy season. The concentration of organic and inorganic total suspended solids (TSS) in the SAR can typically range from 5 to greater than 400 mg/L and can be extremely high during storm flow conditions. The suspended solids in SAR water accumulate in the recharge basins, causing the formation of a foulant layer and rapid declines in recharge basin percolation (Phipps et al, 2007, Hutchinson, 2007). For example, Phipps et al. (2007) report foulant layer induced percolation losses of approximately an order of magnitude over the first 60 days of recharge basin operation.

Riverbank filtration as a technology to improve water quality is well documented through numerous case studies (e.g. Ray et al., 2003; Hubbs, 2006). Riverbank filtration is a passive treatment system whereby river water is captured by shallow wells adjacent to the river. River water infiltrates through the underlying sediments within the river channel and suspended organic and inorganic solids (e.g. clay and silt particles, algae cells, and microorganisms) are removed from the water prior to well capture. OCWD desired to evaluate whether pre-treatment of SAR water by percolating it through riverbed sediments could improve recharge basin percolation rates. OCWD operates a channel adjacent to the main SAR channel, which is called the Off-River Channel, for routing and recharging SAR flows. Maximum off-river channel flow rates are less than 650 cubic meters per minute (m^3/min), which result in generally shallow water depths across the riverbed. A pilot study was designed to use the off-river channel to investigate “riverbed filtration” as a treatment technology to reduce solids and organic carbon concentrations in SAR water prior to groundwater recharge operations.

METHODS

A lateral drain system designed to capture 17 m^3/m of filtered water was installed approximately 1.5 m beneath the SAR off-river channel. The phreatic surface is typically within 0.5 meters of the ground surface when water is flowing in the off-river channel. Figure 1 shows a plan view of the lateral drain system, which consists of eight slotted pipes spaced approximately 24 m apart and extending 58 m across the channel to a main line along the river bank, which drained to a water collection vault containing submersible pumps. Each slotted pipe segment had a gate valve at its junction with the main line to allow lateral drains to be isolated. The slotted pipe was backfilled with approximately 0.5 m of washed pea gravel followed by native riverbed soil material to the surface. Additional details on development and optimization of the lateral drain system design are provided in Keller et al. (2010).

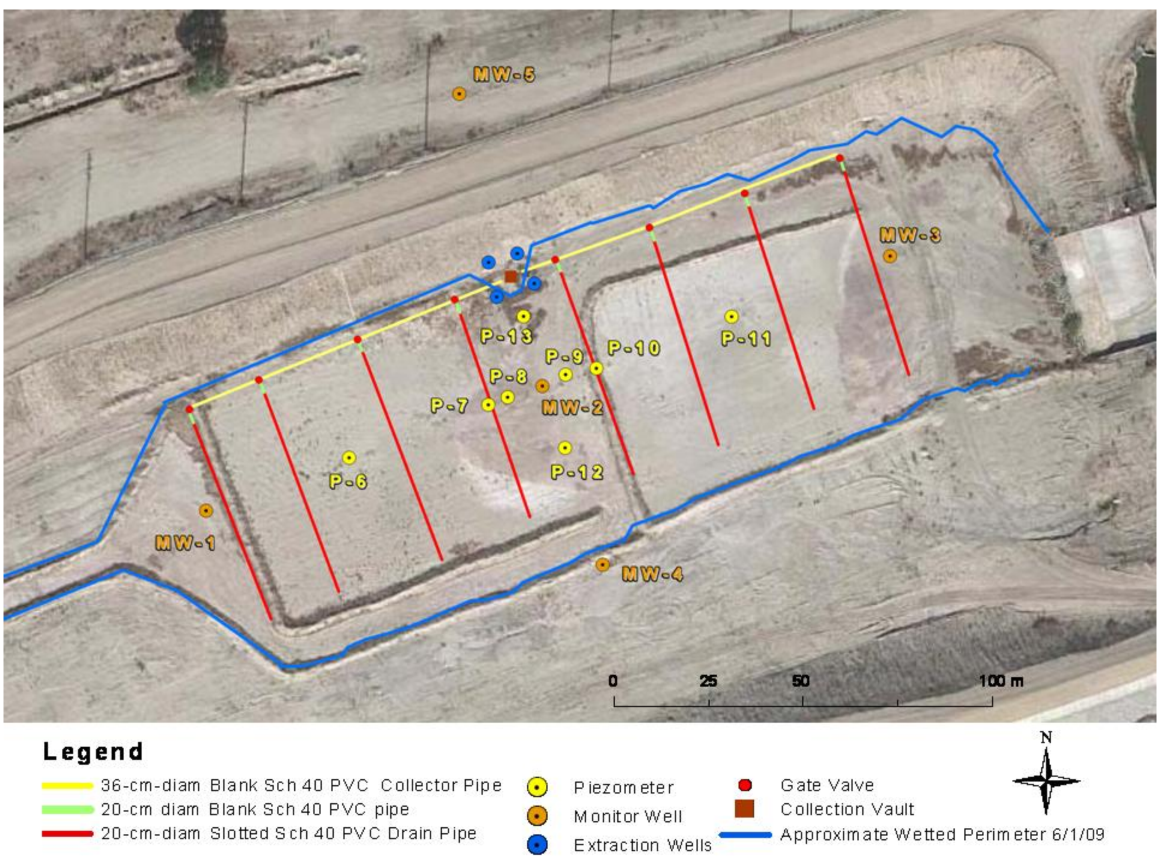


Figure1. Under-channel lateral drain pilot project layout and monitor instrument design.

The pilot study monitoring system consisted of monitoring: inflow and outflow into the drain system area via stream gaging; phreatic level via 5 monitor wells and 8 piezometer points, and; a flowmeter to determine the pumping rates from the under-channel lateral drain system. Locations of piezometers and monitoring wells (i.e. phreatic level monitoring points) are provided in Figure 1. To estimate infiltration and water flux via temperature profiling of heat transport (Constantz, 2008), the piezometers and MW-1 were instrumented with subsurface temperature sensors at 0.3, 1.8, and 3 m below ground surface (bgs). Monitoring wells MW-2 through MW-5 were each instrumented with a single temperature sensor at 3 m bgs.

The pilot test was divided into two periods: Period 1 had shallow overflow (3 cm to 8 cm) within the river channel; Period 2 achieved deeper surface water depths (8 cm to 30 cm) due to the installation of berms within the channel. Lateral drain system pumping during both test periods were incrementally increased to establish the maximum pumping capacity of the drain system for each test period. The maximum sustainable pumping rate was identified as the rate that could be sustained without draining the collection vault or significantly dropping the phreatic surface.

Bi-weekly samples of raw source water and effluent from the riverbed filtration system were collected and analyzed during the first five weeks of Test Period 1 for turbidity, Total Suspended Solids (TSS) and other water quality parameters. Additionally, percolation column testing was performed using raw source water and riverbed filtration system effluent to evaluate percolation decay as an indicator of the effectiveness of the water treatment. The percolation columns were packed with washed sand from an OCWD recharge basin. All columns were saturated from the bottom with riverbed filtered water to avoid air entrapment. Raw water or riverbed treated effluent was then added to the column at constant head conditions and changes in the volume of water passing through the column monitored. The initial percolation rate was measured as the volume of water passing through the column during the first two minutes of the experiment.

RESULTS AND DISCUSSION

Water Quality Improvement: Turbidity, TSS, and Percolation Decay

Table 1. Raw and riverbed filtered water quality

Water Quality Parameter	Influent Value Range	Average Percent Removal
Turbidity	8 - 80 NTU	96%
TSS	7 - 37 mg/L	> 99%
Chlorophyll A	52 - 68 mg/m^3	> 99%
Total Organic Carbon (TOC)	6 mg/L	47%
Total Kjeldahl Nitrogen (TKN)	0.8 - 0.9 mg/L	> 99%
Iron	0.7 - 0.8 mg/L	80%
Manganese	0.06 mg/L	> 99%

Table 1 presents the results of biweekly water quality testing from the first 5 weeks of Test Period 1. Relative to the raw water, the riverbed filtration system significantly reduced the TSS and turbidity by an average of 93 and 86 percent, respectively. Other water quality parameters such as TOC, TKN and iron and manganese showed decreases of 50% or greater. Water quality delivered by the passive riverbed filtration system was significantly better than other active treatment technologies evaluated (data not presented), such as cloth filter, flocculation-sedimentation, dissolved air flotation and ballasted sedimentation (HDR, 2009).

Column percolation decay results using raw water, riverbed filtration water, and conventional filter cloth treated water are presented in Figure 2. Raw water percolation rates decreased to 50 percent of the initial percolation within approximately seven hours. Riverbed filtered water sustained column percolation rates for an extended period of time, decreasing to 50 percent of the initial percolation at approximately 58 hours. However, the percolation rate did not steadily decrease over time and instead variably increased and decreased through-out the column study (Figure 2). Of note, air entrapment occurred in the riverbed filtration column with an initial reduction in percolation rates. However, percolation rates partially recovered once the air was no longer entrapped.

The pilot test water quality and percolation column results indicate that under the study conditions, riverbed filtration significantly reduced turbidity, TSS and other water quality parameters and improved percolation rates. In addition improvements in water quality were superior to other conventional active treatment technologies evaluated.

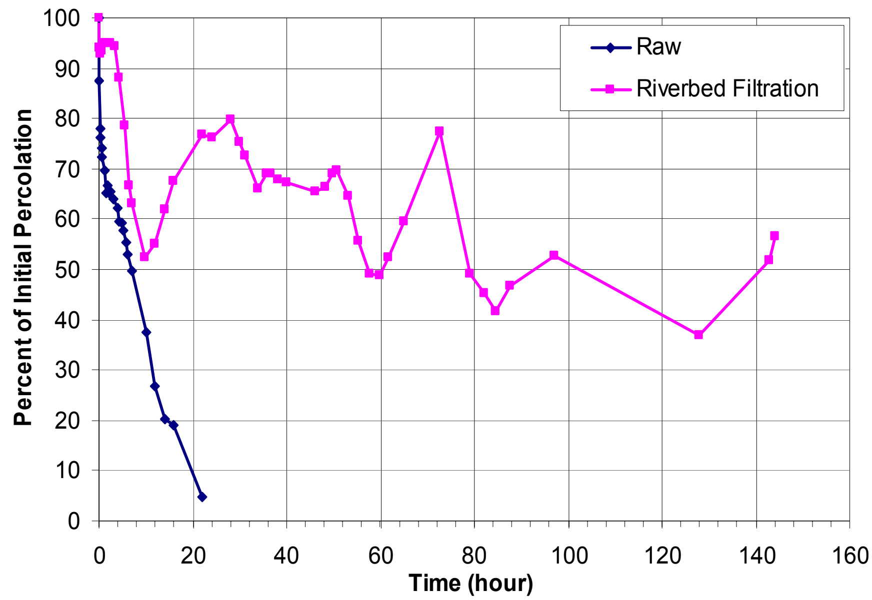
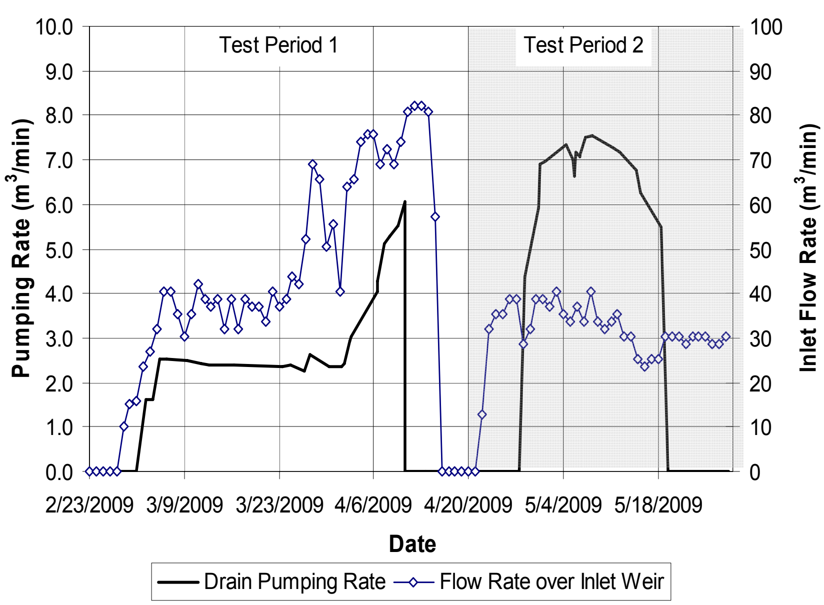


Figure 2. Raw water and filtered water percolation decay.

Hydraulic Performance

Flow and Phreatic Surface Monitoring

Figure 3 presents the drainfield system average daily inlet surface flow rates and pumping rates during the two test periods. During Test Period 1, initial pumping was limited to about 2.5 m^3/min. To increase the depth of water in the channel, surface water inlet flow rates were increased from approximately 40 m^3/min to 80 m^3 /min after which a maximum pumping rate of 6 m^3/min was achieved (Figure 3). However, above 5 m^3 /min pumping rates were not sustainable.



Prior to Test Period 2, berms were constructed to raise the surface water depth over the off-river channel surface. Subsequently, maximum pumping rates of 7.5 m^3/min were achieved during Test Period 2 at channel inlet rates of approximately 40 m^3/min. The achievable maximum pumping rate in both test periods was very sensitive to the flow rate (surface water depth).

Figure 3. Pilot study daily average inlet surface flow and pumping rates.

The response of the phreatic surface to the test period surface flow and pumping activities is shown in Figures 4 and 5 for east-west and north-south monitoring transects through the drainfield system, respectively. The diversion of water over the under-channel drain system is clearly shown in both figures by an increase in phreatic level surface on 3/1/2009. Prior to the start of Test Period 1 pumping, the phreatic surface reached the ground surface only at P-6, indicating an unsaturated zone existed between the surface and most of the under-channel drain system.

Initial pumping in Test Period 1 slightly decreased the phreatic surface east of P-6 (3/26/09, Figure 4). Pumping rates and inlet flow rates were then increased as discussed above. The phreatic levels stabilized at a pumping rate of 4.1 m^3/min and began to decrease at pumping rates greater than 5.1 m^3/min, reaching their deepest Test Period 1 (TP 1) depth at the final pumping rate of 6.1 m^3/min (Figure 4). Slight gradients existed from east to west under non-pumping conditions and, during pumping, a depressed phreatic surface forms between MW-2 and P-11 that reversed the gradient between the east and west sides of the collection vault (Figure 4).

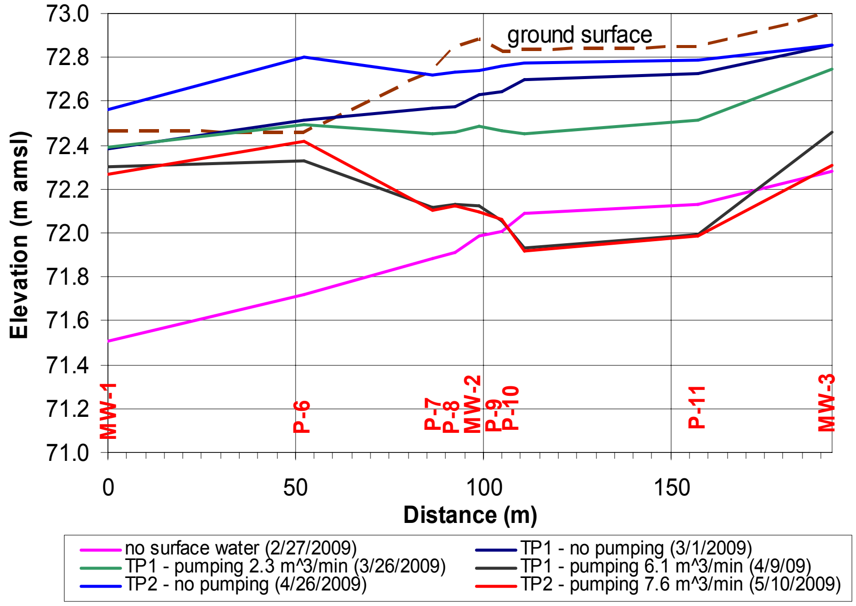


Figure 4. East-west phreatic surface transect data (TP = Test Period)

The Test Period 1 data indicated that the east side of the under-channel drain system is less productive than the west side. In addition, the N-S phreatic level transect shows a steep hydraulic gradient between MW-2 and MW-5 existed during all times of Test Period 1 (Figure 5), indicating that subsurface water flow also occurs to the north away from the under-channel lateral drain system.

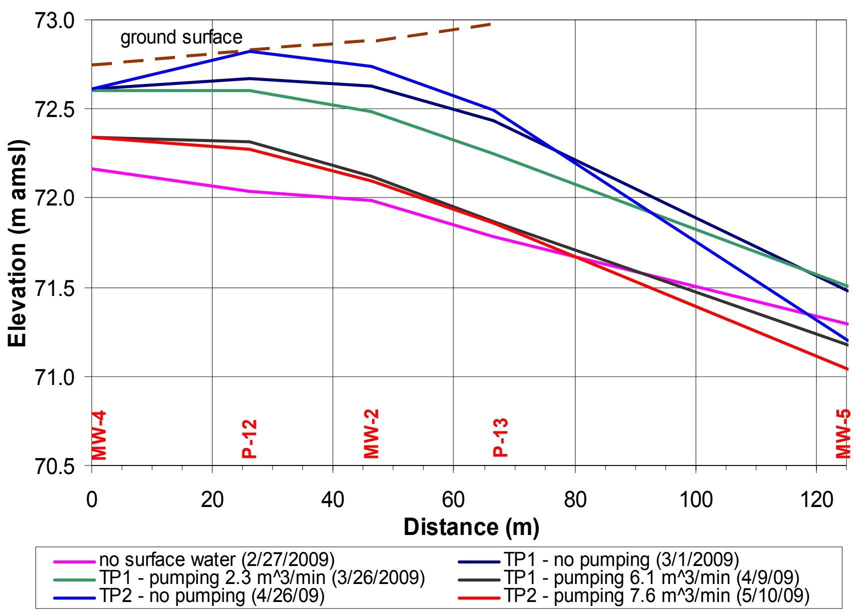


Figure 5. North-south phreatic surface transects data (TP = Test Period).

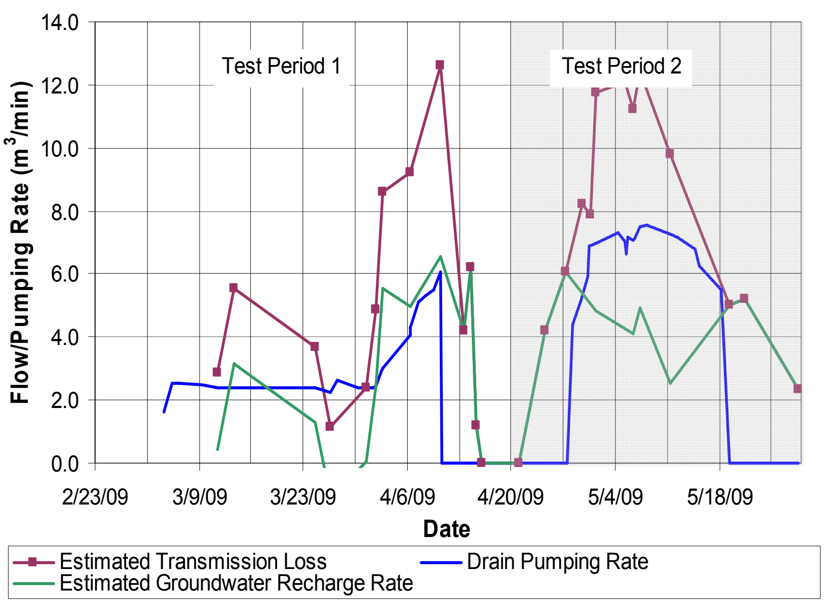
At the end of Test Period 1, the pumps were shut off and the phreatic surface rebounded quickly. West of the collection vault the phreatic surface returned to conditions similar to the start of Test Period 1 pumping. Of note, the construction of berms prior to Test Period 2 and resultant increase in the surface water depth resulted in increased phreatic surface levels at MW-1 and P-6 (4/26/2009 in Figures 4 and 5).

Increases in phreatic surface elevation prior to Test Period 2 were most notable from MW-2 west towards MW-1. East of P-7 remained unsaturated, however the thickness of the unsaturated zone decreased compared to Test Period 1 (Figure 4). During Test Period 2, pumping was again increased daily to identify the maximum pumping capacity of the system. The pumping rate stabilized at above 7.0 m^3/min (Figure 3), at which point the phreatic surfaces began to decrease, until the E-W and N-S transect data closely resembled the phreatic surface elevations observed during maximum pumping rates (6.1 m^3/min) in Test Period 1 (Figures 4 and 5). As in Test Period 1, after pumping was ceased the phreatic surface levels showed less recovery in the eastern monitoring points (data not shown).

The phreatic surface was very responsive to surface water flow (depths) above the drain system. Phreatic surface levels were observed to decrease sharply in response to periods during testing when flows over the inlet weir significantly decreased for a few hours of the day. Finally, it should be noted that the sustainable maximum pumping decreased towards the end of Test Period 2 (Figure 3) due most likely to a combination of reduced surface water flow rates (depth) and possibly clogging of the channel surface.

Estimated Channel Transmission Loss and Groundwater Recharge Rates

The difference between the volume of surface water flowing into the under-channel lateral drain system and the volume of surface water flowing out of the system is defined as transmission loss. Transmission losses were estimated as the difference in measured surface water flow rates over the inlet weir and the stream gage measured outlet (Figure 1). All estimated transmission losses were assumed to have infiltrated into the subsurface within the lateral drain system area. The volume of water going to groundwater recharge is, therefore, the difference between the estimated transmission loss and the volume of water pumped from the under-channel system. Figure 6 shows the estimated transmission loss (difference in flow), pumping rates and calculated groundwater recharge during the pilot study. As pumping rates increased, transmission losses increased, indicating that the under-channel drain system induces percolation. Although the estimated error in the transmission loss calculation is high, the general trends are consistent with the pumping rates. At pumping rates of 7.6 m^3/min during Test Period 2, estimated groundwater recharge volumes began to decline, indicating that the maximum infiltration rates possible under existing conditions (i.e. surface hydraulic conductivities, hydraulic head on the off-river channel sediments) had been attained.



Excluding data prior to 4/1/2009 due to large measurement errors, groundwater recharge rates averaged 4.3 m^3/min during non–pumping periods. During pumping periods, groundwater recharge averaged 4.1 m^3/min. The similarity between estimated recharge rates with and without pumping indicates that the bulk of water pumped from the under-channel lateral system was from induced percolation.

Figure 6. Pilot study estimated transmission loss, groundwater recharge, and pumping rate.

Percolation / Water Flux Estimates from Temperature Data

Near surface percolation and downward water flux estimates at nine different monitoring locations were calculated using down-hole temperature data as a heat tracer. A thorough description of the heat tracer method to estimate subsurface water flux can be found in Constantz (2008).

The average estimated downward flux rate for the maximum and no pumping periods are presented in Table 1. Due to low measured temperature response at most 3 m bgs depth sensors, flux estimates were not made for the 1.8 m to 3 m bgs depth interval. However, temperature response at 3 m bgs was observed at all monitoring locations except MW-5, which is out of the channel, indicating that some water was percolating past the collection laterals and recharging into the local aquifer. Conversely, estimated flux rates over the 0- to 0.3 m bgs interval were greater than estimates between the 0.3- to 1.8-m bgs interval during all time periods. Differences in flux rates between these depths likely reflect lateral flow caused by the steep hydraulic gradients to the north and a decrease in downward flux rates from below the lateral drains (located at 1 to 1.5-m bgs) during pumping,

Table 2. Average estimated fluxes (m/day) using heat tracer data at each monitoring point.

Monitoring Well / Piezometer	0 to 0.3 m bgs			0.3 to 1.8 m bgs		
	Test Period 1, Max Pumping	Test Period 2, No Pumping	Test Period 2, Max Pumping	Test Period 1, Max Pumping	Test Period 2, No Pumping	Test Period 2, Max Pumping
Average West (MW1,P6,P7,P8)	0.49	0.56	0.84	0.36	0.24	0.89
Average East (P9,P10,P11)	0.46	0.35	0.68	0.26	0.22	0.66
Average All	0.46	0.49	0.86	0.28	0.22	0.68

The average estimated downward water fluxes displayed variability at different locations within the under-channel lateral system. Estimated water fluxes during Test Period 2 maximum pumping rates were approximately double those observed during Test Period 1 maximum pumping rates, and the average estimated flux rates at the monitoring locations west of the collection vault were higher than monitoring locations east of the vault. The spatial variability in estimated flux may represent local differences in hydraulic conductivity and the hydraulic head (surface water depth) at that location. At 0 m to 0.3 m bgs the slight increase in estimated flux during Test Period 2 no pumping compared to Test Period 1 maximum pumping rates most likely reflects the increased surface water depth after the berm addition. The increase in estimated downward flux rates during Test Period 2 maximum pumping period also confirms that the under-channel lateral system induces increased percolation in response to increased pumping rates.

CONCLUSIONS

The riverbed filtered water quality results and percolation column results indicate that riverbed filtration significantly reduces turbidity, TSS and other water quality parameters and improves percolation performance in addition to outperforming conventional active treatment technologies.

The sustainable maximum pumping capacities were highly affected by surface water depths in the channel. Additionally the maximum pumping capacities achieved during the different test periods equated to 30 percent and 44 percent, respectively, of the design collection rate (Keller et al., 2010). The discrepancy between the design collection rate and maximum pumping rates achieved during the pilot study was due to primarily to the field conditions not matching assumptions used in the pilot system design. Most notably, the pilot system design assumed saturation below the off-river channel surface due to underlying geologic conditions. Unsaturated conditions below most of the drainfield and the steep hydraulic gradient to the north reduced the available hydraulic head to induce flow into the under-channel drain system.

The phreatic level data also confirm that the under-channel drain system performance is sensitive to both spatial and temporal variability in surface water depth, such that decreases or increases in surface water flow and hydraulic head substantially decreased or increased the phreatic surface. As evidenced by the higher phreatic levels in the western portion and induced hydraulic gradients to the east portion of the drain system, performance may also be affected by variability in channel sediment permeability and reduced hydraulic conductivity as the underlying sediments become unsaturated. In the future surface and subsurface treatments (i.e. ripping/scarifying or removal of surface sediment) may be evaluated to determine whether these treatments can improve channel sediment hydraulic conductivities.

The temperature data also confirmed that the under-channel drain system induces percolation during pumping operations and that most of the water collected by the under-channel drain system is from induced percolation. Estimated transmission losses, groundwater recharge, and downward water flux rates from temperature data were significantly greater during Test Period 2 than those estimated during Test Period 1. The increase in estimated flux rates during Test Period 2 correlate to increased surface water depth and increased drain pumping.

Results from the pilot study indicate that riverbed filtration is a viable and superior method to other commercially available active treatment technologies to improve water quality and increase downstream recharge basin percolation rates. Ongoing testing is needed to determine the optimum conditions for surface water flow rates/depths and the under-channel drain system performance. In addition, long-term maintenance of channel sediment clogging and operational treatments will need to be assessed. Finally, results from this pilot study can be used to guide future design of other OCWD riverbed filtration systems.

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Prospects of Artificial Recharge for Augmentation of the Upper Dupi Tila Aquifer in Dhaka City, Bangladesh

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Abstract
Dhaka, the 10th largest megacity of the world, is facing severe shortages of water despite receiving an annual rainfall in access of 2000 mm/year. Unplanned urban development due to rapid population growth has been the cause of encroachment on retention and natural drainage areas creating obstacles to natural recharge to the aquifers beneath the city. This study attempts to evaluate the prospects of artificial recharge to augment the depleting aquifer storage by way of identifying the potential zones for the implementation of site-specific artificial-recharge techniques using GIS analysis.

Satellite image analysis and GIS mapping reveal that about 36.33% of the city area allows natural recharge at present conditions which is much lower compared to the heavy abstractions for municipal and industrial supplies. Historical data show sufficient rainwater is available during the monsoon which can be used as a source of artificial recharge. If the rainwater is diverted directly to the subsurface from the roofs, there should not be any major quality concern. There is a thick dewatered zone in the depleted upper Dupi Tila aquifer which can provide enough storage space for artificial recharge. Based on the clay thickness, depth to aquifer, groundwater level, rainfall distribution pattern, lithology and hydraulic properties of the Madhupur clay, four potential zones for artificial recharge have been identified where various infiltration methods can be implemented to augment aquifer storage. However, pilot studies are necessary to confirm the findings of this mapping exercise.
Key words: Dupi Tila Aquifer; Artificial Recharge; Dhaka Mega city; GIS; Madhupur Clay.

INTRODUCTION
Dhaka, the capital of Bangladesh (Figure 1a), has a population of about 15 million who mainly depend on groundwater for municipal water supply. The pressure on ground water continues to rise to meet the exponential rise in water demand. About 86% of the present municipal water supply comes from groundwater sources with the remainder supplied from surface water sources (DWASA, 2008). Dhaka Water Supply and Sewerage Authority (DWASA) produces 1.6 Mm^3/d of groundwater through nearly 500 deep wells drilled in the upper and lower Dupi Tila aquifers. Exponential growth in the number of wells and estimated accumulated pumping volumes give an impression of an overexploitation rather than managed abstractions. The problem has been further compounded by large-scale urbanization in the low lying areas which has reduced open lands available for natural recharge significantly.

The mechanism of groundwater recharge of Dhaka city is not well understood and needs to be further investigated. However, it has been reported that the recharge is primarily by topographically-driven vertical leakage through the Madhupur Clay (Hasan et al 1998). Radical changes in groundwater recharge regime have been reported in other parts of the world due to increase in paved area (Lerner, 1990). Filling up the wetlands and natural drainage channels for urban development is also reducing the recharge area and thus vertical recharge significantly. The rate of water table decline has reached up to 3.5 m/year in the central part of the city, and the upper Dupi Tila aquifer has changed from initial confined conditions to unconfined conditions due to these over-abstractions (DWASA and IWM, 2008). The decline in water levels is also making Dhaka more vulnerable to ground subsidence.

Decline in water levels has also caused increased drilling and pumping costs as wells need to be installed in deeper and deeper levels. If the unrestricted withdrawal of groundwater continues, it will not only affect the city's future water supply, but will increase the risk of earthquakes induced ground subsidence of the clay soil cap on which Dhaka is developed. In the backdrop of extremely high demand of this natural resources, it is of paramount importance to artificially recharge the depleted aquifer to augment the over exploited resources and to mitigate the associated adverse impacts.

Different techniques of artificial recharge are available that could provide a management option to overcome this critical situation. Available studies on artificial recharge indifferent regions of India under similar climatic and geologic conditions like Dhaka have been reviewed to evaluate the potentiality of artificial recharge in Dhaka city.

ARTIFICIAL RECHARGE, ITS REQUIREMENTS, NECESSITY AND TECHNIQUES

The concept of artificial recharge has been known for a long time. The practice began in Europe during the early nineteenth century. However, the practice has rarely been adopted on a large scale, with most large-scale applications being found in countries such as the Netherlands, Germany, and USA. In India, the applicability of artificial recharge technologies has been evaluated through a number of studies conducted by the Central Ground Water Board and the State Ground Water Boards (CGWB, 2000).

The basic purpose of artificial recharge of groundwater is to replenish aquifers depleted by excessive withdrawal. Artificial recharge is one of the supplemental means available to restore declining groundwater levels and to reduce risk of land subsidence. It can also be implemented in areas where saline water intrusion takes place.

The amount of natural recharge varies greatly from region to region and within the same region from place to place depending upon the amount and pattern of rainfall, characteristics of soils and rocks, terrain conditions and other climatic factors. As a result, availability of water from sub-surface storages has considerable geographic variation.

Advantages of artificial recharge are many. It doesn't require large storage structures and enhances the dependable yield without evaporation loses. It Improves water quality by dilution and natural attenuation processes. There is no risk of surface inundation and thereby no displacement of local population and loss of crops. Artificial recharge is environment friendly and economically viable and cost-effective. It also utilizes and reduces the surplus surface runoff and thus also helps in reducing the storm runoff disposal challenges in cities.

A prime prerequisite for artificial recharge is the availability of source water, which is assessed by analyzing the seasonal rainfall-runoff pattern. Detailed geological and hydrological knowledge of the area is necessary for delineating the recharge capture zones and the type of recharge system. Factors such as geological boundaries, hydraulic boundaries, inflows and outflows of surface waters, storage capacity, porosity, hydraulic conductivity, transmissivity, natural discharge of springs, water resources available for recharge, natural recharge, water balance, lithology, depth of the aquifer and water table, and tectonic boundaries need to be considered. The availability of sub-surface storage space and its replenishment capacity ultimately govern the potential extent of recharge.

Artificial recharge techniques can be broadly divided between surface and subsurface methods. Typically, unconfined aquifers are recharged by surface methods, whereas confined aquifers are generally recharged through subsurface injections. Surface methods require relatively flat or gently sloping lands, while topography has little effect on subsurface recharge methods. Dense urban and industrial concentrations may result in subsurface artificial recharge schemes as the preferred option. Subsurface injection methods like injection wells, shafts or small pits require highly controlled water supplies and little land area. In this study, an attempt has been made to identify zones favorable for the application and implementation of site-specific artificial-recharge techniques for augmentation of groundwater through GIS mapping.

TOPOGRAPHY AND CLIMATE OF DHAKA

Topographically Dhaka is almost flat with many depressions. The elevation is between 2 to 13 m above the mean sea level. The city is characterized by subtropical to tropical monsoon climate. The long-term mean annual rainfall is over 2,000 mm, and about 80–90% of this occurs during the monsoon (May–October) and average evaporation ranges from 80 to 130 mm/month.

GEOLOGY AND AQUIFER SYSTEM

Dhaka lies at the southern edge of the Plio-Pleistocene Madhupur Tract and surrounded by four rivers (Figure1b). Stratigraphically, the area is characterized by an unconsolidated sequence of fluvio-deltaic deposits hundreds of meters thick that are usually composed of gravels, sands, silts and clays of Plio-Pleistocene age. The Madhupur Clay formation, which is composed of characteristically red plastic clay to silty clay and silt, is unconformably overlain by alluvial deposits and is underlain by fine to coarse-grained micaceous, quartzo-feldspathic sands of the Pliocene Dupi Tila formation. The low lying areas along the edges of the tract are covered by Holocene alluvial silt and clay and marshy clay (Figure 1b).

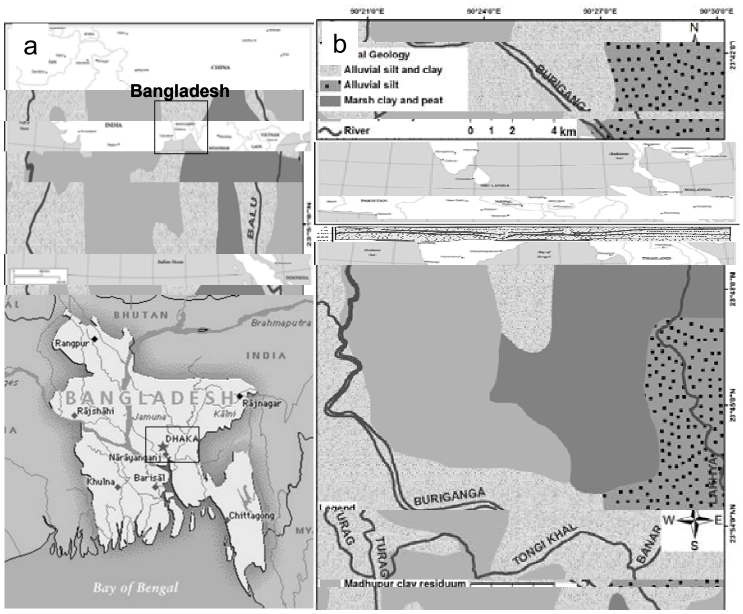

Figure 1 (a) Map showing the regional setup of Bangladesh and location of Dhaka city and (b) Surface geology of the city

The multi-layered Dupi Tila formation is the principal aquifer underneath the Dhaka city area. It is effectively confined by the semi-pervious Madhupur Clay. Based on grain-size distribution of the aquifer materials and hydraulic properties, the aquifers can be separated in to three units; the Upper Dupi Tila Aquifer-1, Upper Dupi Tila Aquifer-2 and Lower Dupi Tila Aquifer (DWASA and IWM, 2008) (Figure 2).

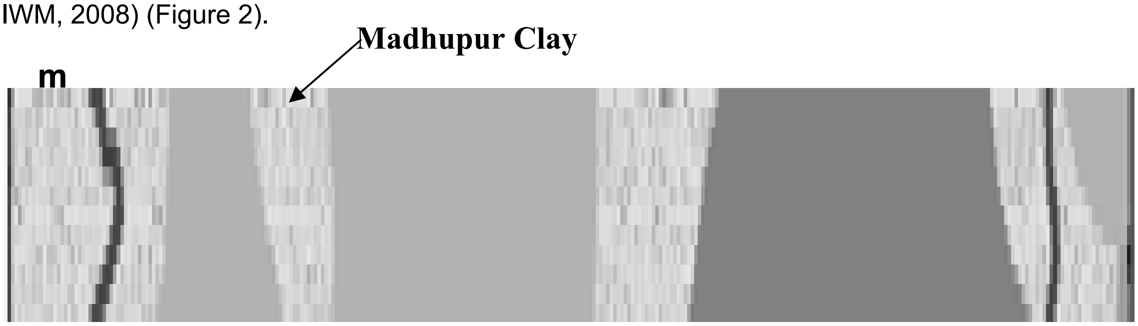

Figure 2 Aquifer System in Dhaka city

METHODS

Advanced Space borne Thermal Emission and Reflection Radiometer (ASTER) data acquired in November 2004 has been used to generate a potential recharge area map. ERDAS Imagine 9.1 image analysis software has been used to process the image. Major land cover classes have been digitized by ArcGIS 9.2 mapping package using visual interpretation (Figure 3b). A contour map of clay thickness was prepared based on the borehole logs from DWASA wells. Mean depth to groundwater level of 2006 for the upper aquifer has been mapped based on data from 29 observation wells of Bangladesh Water Development Board. Historical average annual rainfall data for 15 rain gauges collected from the Bangladesh Meteorological Department have been analyzed to produce an isohyetal map. The thematic layers were then ranked, reclassified, and overlaid and analyzed in a GIS environment. Based on this analysis, the Dhaka area has been divided into four zones according to the potentiality for artificial recharge.

RESULTS AND DISCUSSIONS

Recharge Area map

Five major land cover classes were identified to prepare the potential recharge area map: (1) open or unpaved area (vegetation reflection), (2) open area (sand reflection), (3) low-lying area (depression), (4) natural water body (beside periphery) and (5) water body (Figure 3b). The map reveals that the recharge area in Dhaka city extends over 110 km^2 within the total city area of 300 km^2 i.e. 36% of the city is acting as recharge area. A major portion (about 40%) of the total recharge area map comprises of natural water bodies followed by a small proportion of open areas having sand reflections.

Clay Thickness Map

The thickness of upper clay increases from west to east in Dhaka City (Figure 3b). This layer is much thicker in the north-middle and southeast portion of the city and ranges from 30 to 40 m and reaches up to 50 m to some locations. It ranges from 10 to 20m in the north-western parts except Mirpur area. In the southern part of the study area, the thickness of upper clay layer also ranges between 10 and 20m. A comparatively think clay layer (<10m thick) has been mapped at the southern middle portion.

Depth to Groundwater Level

Depth to water level has reached to more than 60m in three spots, viz. Mirpur, Dhanmodi and Sabujbagh in the city (Figure 3a). Water level depth is about 40 to 50m in the middle part such as Tejgaon, Motijheel and Ramna; and 20 to 30m in the areas like Sutrapur which is close to the river Buriganga.

Rainfall Distribution map

The major portion of the city receives rainfall in the amount from 1950 to 2000 mm whereas the northern part is characterized by comparatively low rainfall. There is a pattern of increasing rainfall from north to south of the city (Figure 3a).

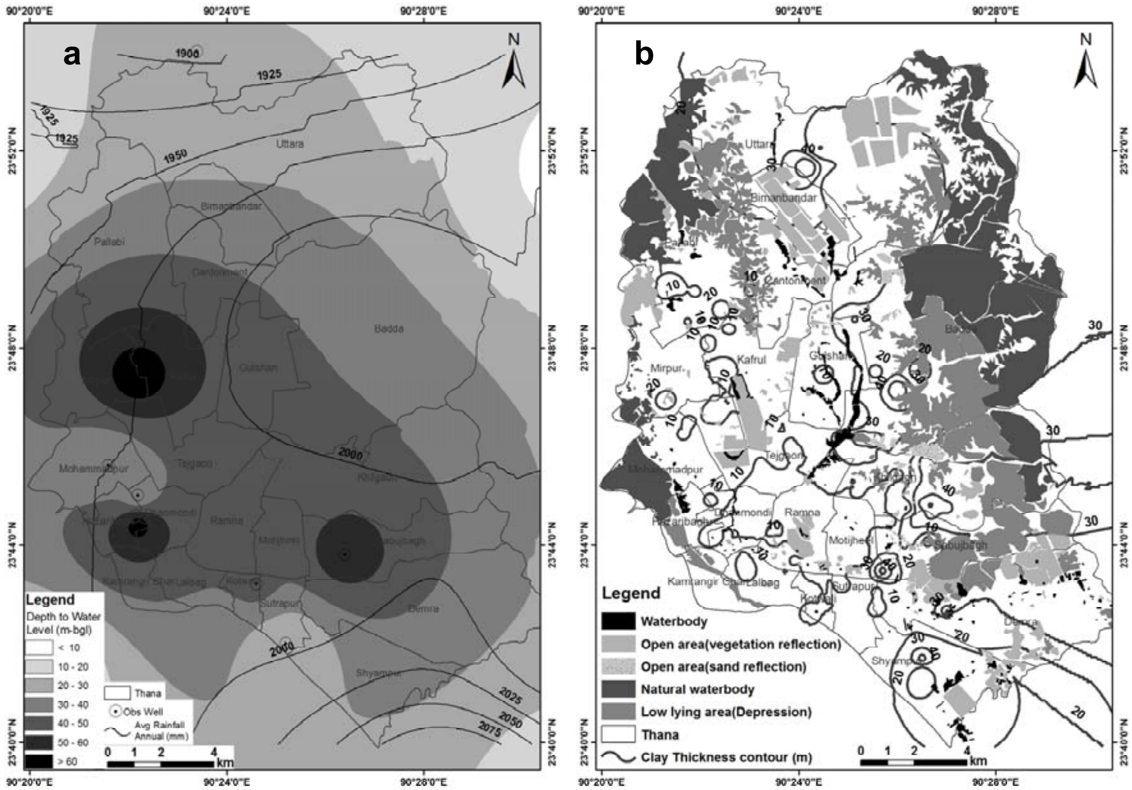

Figure 3 (a) Depth to groundwater level and rainfall distribution contour Map in and around Dhaka city (b) Recharge area map with classes and clay thickness contour map of the study area.

Suitability and applicability of Artificial Recharge in Dhaka

The evaluation of the potential aquifer areas requires data on the thickness and lateral extent of unsaturated zone, which control the total volume potentially available for recharge. Fine-grained Upper Dupi Tila Aquifer 1 is approximately 40–50 m thick and occurs at a depth of 8 to over 45 m below the ground surface. The coarse-grained Upper Dupi Tila Aquifer 2 is approximately 80 m thick (Ahmed et al 2010). Upper part of the Dupi Tila Aquifer 1 has already been dewatered. The permeability, transmissivity and storage coefficient of the aquifer is 15-30m/day, 500-2000m^2/day and 7×10^{-6} to 5×10^{-5} respectively (Ahmed, 1999) which indicate good hydraulic properties for the storage and transmission of water. The Dupi Tila aquifer can be considered as "warehouse" for storing substantial quantity of water as a significant part of it has been dewatered already (Hoque et al, 2007). The aquifer can also serve a conduit function, which could reduce the cost of an intensive surface water conveyance system. The upper aquifer is most suitable for artificial recharge as it provides the necessary permeability and storage space.

The Madhupur Clay mainly consists of kaolinite and illite with very small amount of illite-smectite down to 5 m depth (Nairuzzaman et al, 2000). The coefficient of permeability is related to the amount of illite-smectite mixed-layer clay minerals. The vertical permeability of the Dupi Tila aquifer varies from 6.5×10^{-4} to 1.5×10^{-2} m/day (DWASA, 2008). The Madhupur Clay can thus neither yield significant amounts of water to wells nor transmit appreciable water to the aquifer below. So any artificial recharge scheme in Dhaka must penetrate the clay rather than trying to regulate recharge through it.

About 36% of the city is acting as natural recharge area, which is insufficient to meet the rate of discharge through hundreds of municipal and industrial wells and this area is being reduced continuously due to rapid increase in urban development. Over-draft conditions have formed a major depression in groundwater levels at Mirpur, Dhanmondi and Shabujbag areas. The continuous decline of the water level with little or no fluctuation is typical of overexploited aquifers and requires groundwater augmentation as reductions in abstractions is not feasible.

The average annual rainfall of Dhaka city is about 2000mm which is enough to supply water needed for artificial recharge. Rain water collected from rooftops of the buildings can be used for this purpose. According to Statistical Year Book (BBS, 2006) the city has about 678,000 concrete roofs. If we consider the area of each roof is 110 m^2, about 75 km^2 roof area is available to catch rainwater. Considering the annual average rainfall, more than 400 MLD of rain water will be available to recharge the aquifer artificially. If 60 % of this water can be collected, then more than 200 MLD water will be available for recharging the aquifer. There are about 265 abandoned wells in the city which can potentially be used as injection wells for artificial recharge.

CONCLUSIONS RECOMMENDATIONS

At present about 36% of the city is acting as natural recharge area which is insufficient to maintain the dynamic equilibrium in groundwater levels at current rates of pumping. Recharge area is being continuously decreased due to rapid urban development to cope with the ever increasing population growth. Four potential zones have been identified for artificial recharges to increase aquifer storage is shown in Figure 4.

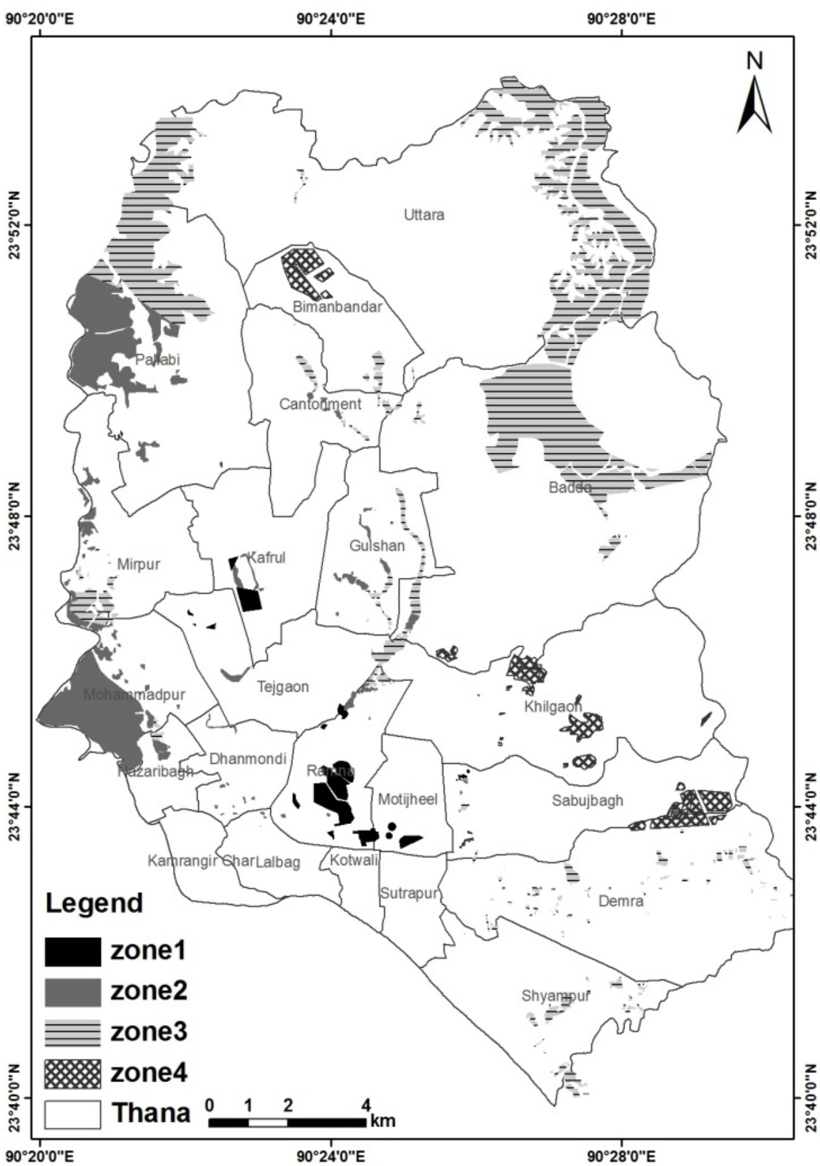

Figure 4 Map showing the potential zones favorable for various AR techniques.

Zone 1 is delineated where clay thickness is 0 to 10 m and open area is available. Zone 2 is the region where thickness of clay is 10 to 15 m and water bodies are available very close to it. Zone3 and 4 have 15 to 25 m and 25 to 50 m of clay respectively. Where the clay is thick, artificial recharge has to introduce using injection wells.

The results of this investigation indicate that Dhaka has high potential for artificial recharge, which can play a critical role in maintaining a sustainable water supply for the city dwellers. Remote sensing data, conventional geological data, and GIS overlay analyses combinely provide a powerful and practical approach to identify potential zones for artificial recharge. The applicability of various techniques in the identified zones need to be assessed by pilot studies and if successful has to be adopted as a management option to augment the depleting groundwater resources of Dhaka aquifers and also to avoid adverse environmental impacts.

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Sustainable and climate-specific rainwater management by water-permeable pavements in the Middle East region

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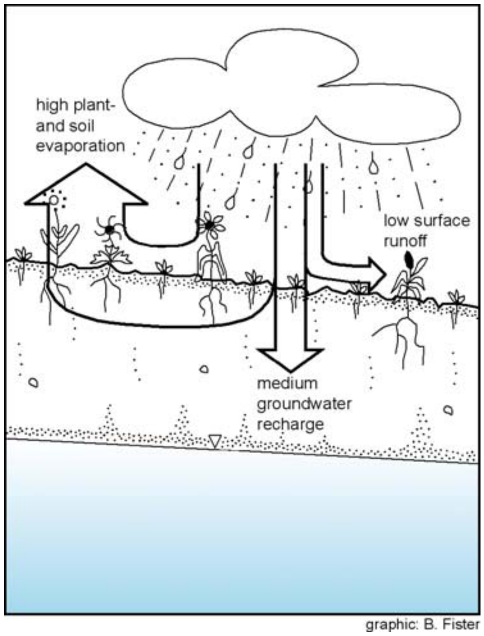
Abstract
The main functions of water-permeable pavements, made of pervious concrete, are to infiltrate precipitation water and so reduce surface runoff and, in turn, increase the groundwater recharge. The pervious material means that evaporation rates are greater than from normal pavements and this has a positive effect on the city climate as well as on the water balance. In arid regions, high evaporation rates represent a loss of scarce water and therefore, evaporation rates and surface runoff should be as low as possible.
By the proper use of different street building materials, the evaporation rates of these surfaces can be controlled. For example, fine grained materials show enhanced evaporation characteristics, while coarse grained materials are classified as diminishing evaporation. The filling of seams and the design of the pervious concrete allow higher or lower evaporation rates, compared to impermeable pavements.
This results in water-permeable streets with extremely high infiltration rates for the Middle East region. Furthermore, it is possible to store the water in the deeper layers, such as the sub-base, which have no significant effect on the evaporation rates. Therefore, stored water is protected against evaporation and can be subsequently used for irrigation, for example.

Keywords
Evaporation rates, groundwater recharge, rainwater management, urban climate, water-permeable streets, water storage

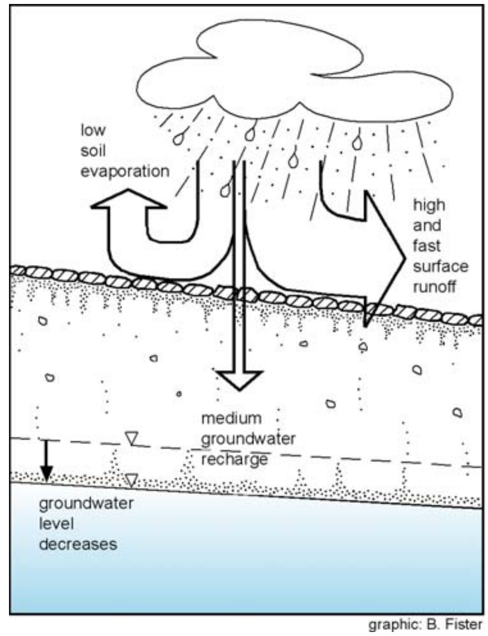
INTRODUCTION

The temporal variance in the availability of (rain-) water is characteristic for Middle Eastern climate. In contrast, the water demand, caused by the fast growing population of mega cities in the region, increases. Natural water resources are already used over capacity. The groundwater recharge is low, while a part of storm water is discharged by surface runoff. Surface discharge results in a large amount of water lost from the water supply.

In contrast, the moderate and (semi-) humid climate of Europe provides a surplus of available water. The natural groundwater recharge is, in many areas, high enough to supply the population with fresh water. Therefore, one of the dominating problems in regard to the water cycle is not groundwater recharge, but high surface runoff rates in urban areas. Here, the soils are sealed by the use of impermeable materials, for example in street building. Evaporation and infiltration are greatly decreased. Precipitation is mainly discharged by surface runoff (Pic. 1 and Pic. 2) (Schueler, 1994; Arnold & Gibbons, 1996). This leads to several problems like floods and a low quality city climate (Kuttler, 2008; Wessolek, 2008). To increase the infiltration rates, and to unburden sewer systems and recipients, water infiltration systems and water-permeable pavements are now used increasingly.



Picture 1: Water balance of natural ground (Modified by Göbel et al. 2004)



Picture 2: Water balance of sealed ground (Modified by Göbel et al. 2004)

In a research project at the University of Muenster, the evaporation rates of water-permeable pavements were the focus of attention. It was assumed that evaporation rates from these water-permeable pavements (made of pervious concrete) are higher than from impermeable pavements. In an research project (Starke et al., 2010a; 2010b), it was discovered that water-permeable pavements actually have a 16% higher evaporation rate than impermeable pavements. Therefore they help to improve the city climate through a cooling effect.
Furthermore, it was found that the deeper layers (i.e. sub-base) have no significant effect on evaporation rates. The upper layers create a barrier against evaporation. Graf (2004) observed the same effects on the water balance with other porous layers. His field of research was the Enarenado agriculture at Lanzarote, where layers of Lapilli, a basaltic and porous material, protect the infiltrated water against evaporation.

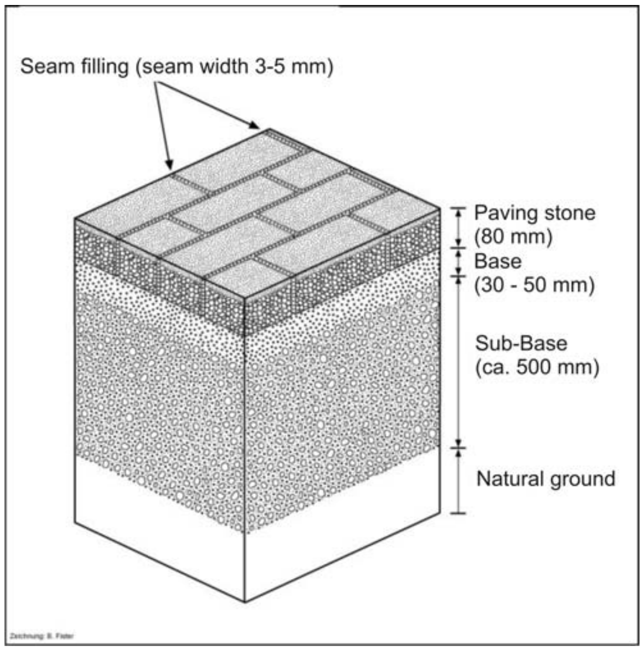
In spite of this, the nature of the top layer affects evaporation rates in a significant way (Starke et al. 2010b) and changes in the composition of the paving stones cause different evaporation rates. For example, a change in the paving stone colour creates 19% higher evaporation rates. Although a small proportion of the surface, seams can act like an interconnection between the base and the atmosphere. Furthermore, a significant amount of water can be stored in the seams themselves. Therefore, the effective pore volume V_{eff}, as a dimension for the maximum water content is an important parameter, as is the water retention capacity *wrc*, which is the amount of water kept against gravity. Furthermore, the permeability k_f (i.e. amount of infiltrated water) and the capillary rise c_r (water from deeper layers) are also important. These four important hydraulic attributes were measured for different street building materials. In addition to tests on loose materials, new paving stone prototypes were also designed and produced.

In a newly developed lab-test, the evaporation rates of the prototypes were tested. This test was invented so as to create a pre-selection for the later field tests. The device consists of an evaporation cell with a balance on which a moistened sample is placed. An overhead heat source then causes evaporation. A constant horizontal airflow over the sample surface ensures a continuous discharge of water vapour. The testing method will be further discussed in Starke et al. (in preparation).
This research showed that the use of pervious concrete can not only increase evaporation rates in regard to impermeable surfaces, but also decrease them. In connection with the high infiltration rates, this results in a high groundwater recharge rate, thereby making it possible to regulate the different parts of the water balance in a regional specific context. To avoid a poor city climate caused by reduced evaporation, evaporation rates can be increased.
Reverting to the Middle East’s water problems, water-permeable pavements can decrease the evaporation rates and, simultaneously, increase the groundwater recharge. With different street body

designs it can also be possible to use the street body as water storage body. By pumping the stored water off, it can be used directly for irrigation, for example.

METHODS

A common pavement construction consists of different layers (Pic. 3), topped with paving stones and seam fillings. The seam filling is a fine grained material (majority of grains <0.25 mm) with water-permeable pavements mainly having grain sizes between 1 mm and 3 mm (1/3 mm). Both are bedded on the base where the variations in the grain size distribution are greater. Mostly fine grained or sandy material is used, but there are also base materials without fine grained material up to 2/5 mm. The sub-base is the bottom layer of the street construction. It comprises coarse grained material which transfers superimposed loads to the underlying ground and protects the pavement from mechanical damage. The common grain sizes are 0/32 mm up to 0/45 mm with a high variation in the part of the fine grain size fraction.
From all these different layers used in street building, suitable materials were collected and then tested in the laboratory.



Picture 3: Block scheme of a street body (Starke et al., 2010b)

The first research phase related to the grain size distribution of the loose material. It was assumed that the grain size will have an effect on the hydraulic attributes. Therfore a classification was developed as such materials with a proportion >15% of fine grains are classified as fine grained. Materials with 15 % or less are classified as coarse grained.
The effective pore volume V_{eff} (Vol.-%) was tested according to DIN EN 1097-6:2005.
For measuring the water retention capacity *wrc* (Vol.-%), the sample was completely saturated with water and then placed on a grate. Five minutes after water had ceased to drip out, the sample was weighed. The remaining mass of water, in regard to its volume and the volume of the sample, showed the water retention capacity.
The capillary rise c_r (cm) was tested in a 65 cm long plastic tube in which the oven-dry sample material was compacted. After that, the bottom of the tube was placed in a water bath with a constant water head. There the sample rested for at least 3 hours, after which the tube was removed from the bath, laid on a table and opened with a longitudinally cut. In the tube, the open tread was observed and the height of the wet sample was measured. The water-permeability k_f (m/s) was tested according to DIN 18130-1:1998 at proctor density.

In addition to the loose materials, the effect of different grain size distributions in pervious concrete was also tested. The effective pore structure of pervious concrete is similar to the loose material and this effective pore structure is maintained because the grains (concrete aggregates) are cemented together at just a few points. That is why water can percolate through the stone itself and the street becomes water-permeable. As an example, two pervious prototypes were developed (Pic.4). Prototype A was produced with fine grained aggregate having a grain size fraction of 1/2 mm and prototype F coarse grained aggregate with coarse grains in the fraction 5/8 mm. For both prototypes, all hydraulic parameters were tested as being equal to the loose material.

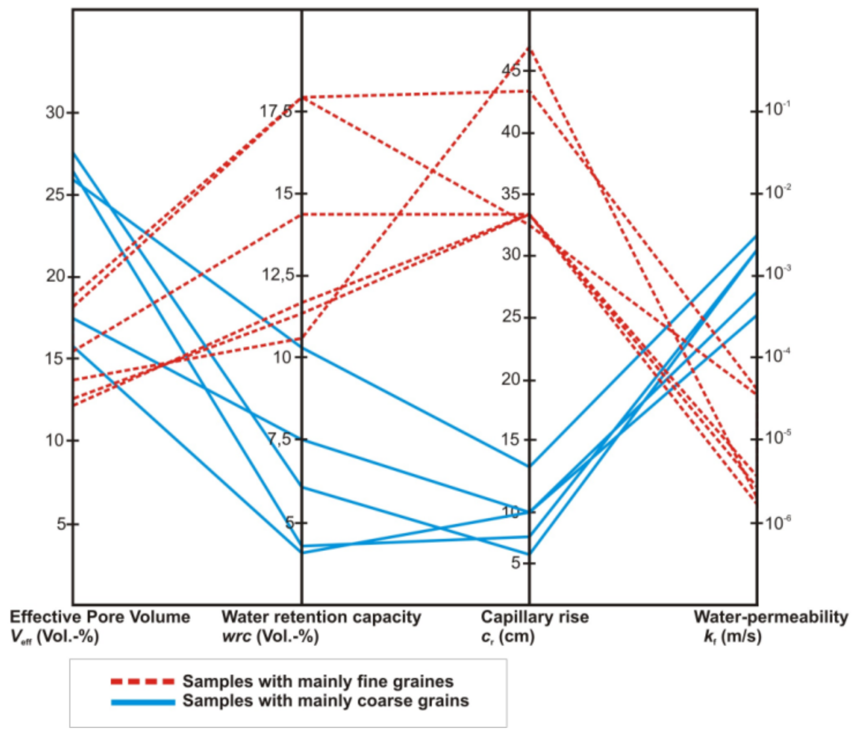
Picture 4: Prototype A (left) and prototype F (right)

By the laboratory evaporation test, the evaporation rates ET_{lab} [mm/h] of the samples were compared. The samples were moistened with 20 g water which was evenly distributed over the sample surface, and then, the sample was measured for 7.5 hours in the laboratory evaporation device. The loss of mass, logged by the balance equates to the evaporated water and, based on this, the evaporation can be converted into mm/h, as a mean value of the evaporation rate. Afterwards a relative estimate of the evaporation rates in the field was made

RESULTS

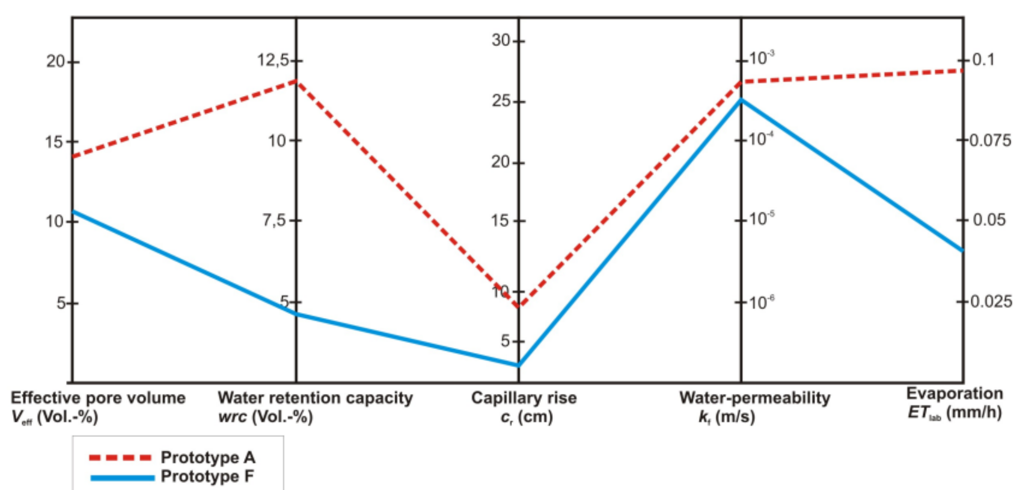
The loose materials show a significant correlation between the part of the fine grains (<0.25 mm) and the hydraulic attributes. Picture 5 shows the different hydraulic attributes of the loose materials. The blue graphs are materials with a proportion of less than 15% fine grains (<0.25 mm), the red doted graphs are materials with more than 15%. It is obvious that a high proportion of fine grains cause a high water retention capacity of between 13.2 and 17.7 Vol.-%. The materials with less than 15% of fine grains show a water retention capacity of between 4.4 and 11.5 Vol.-%. The same effects can be observed at the capillary rise where the fine grains increase the capillary rise up to 47cm. The coarse grained materials show a 13 cm rise (maximum). The effects on the water-permeability are opposite to water retention capacity and the capillary rise. Here fine grains reduce the water-permeability. Therefore the materials with the less fine grain fraction are more water-permeable (up to $k_f = 2.76 \cdot 10^{-2}$ m/s).

The effective pore volume, which equates to the amount of water the pores can contain, is at mean lower at the fine grained materials. In spite of this, this trend is not as significant as the trends of the other hydraulic attributes.



Picture 5: Hydraulic attributes of loose material

The prototypes show nearly the same characteristics (Pic. 6). The fine grained prototype A shows a high water retention capacity and a higher capillary rise. The capillary rise in Picture 5 is shown as being 8 cm. The maximum capillary rise is limited at 8 cm because of the 8 cm high prototype which was completely imbued. So the capillary rise is, strictly speaking, at minimum 8 cm and can be higher in reality. The effective pore volume of prototype A is 14.6% higher than for prototype F. The water-permeability $1.64 \cdot 10^{-3}$ m/s and $2.16 \cdot 10^{-3}$ m/s is almost identical.
In contrast to the loose materials, the laboratory evaporation rate ET_{eff} is shown in Picture 6 also. Prototype A shows a 2.5-times higher laboratory evaporation rate than prototype F.



Picture 6: Hydraulic attributes and ET_{lab} of the Prototypes

DISCUSSION

In the case of impermeable layers under the street body (either natural or man-made), water can be stored. Therefore, the effective pore volume V_{eff} should be as high as possible This can be observed for materials which have little or no fine grain-fraction. The laboratory tests show that the effective pore volume can vary about 15%, with a direct effect onto the storage capacity of the whole pavement.

The highest potential for water storage is in the sub-base. Here, a minimum thickness of 50 cm (requested by the unique national technical approval for pervious concrete paving stones (DIBt, 2006)) leads to a storage capacity of 30 l/m^2 just in the sub-base. By increasing the thickness, the storage capacity increases correspondingly. Furthermore, the research by Starke et al. (2010b) shows that there is a good reduction in evaporation losses from the sub-base. If the base-material shows a high pore volume and has no capillary effects, the evaporation from the sub-base should not be significant. This can be achieved if the base material has no fine grain fraction. Because of the high permeability, the water infiltrates very quickly into the sub-base. This improves the protection against evaporation losses further and avoids an afflux from the pavement surfaces at storm events.

The water retention capacity shows the amount of water that is kept against gravity, and this is available for evaporation. The laboratory tests show that the proportion of fine grains is critical to the water retention capacity. Therefore, near-surface layers, like the seam material or the paving stones, should have a lower water retention capacity. The maximum grain size of the seam material cannot be more than 3 mm because of a seam width of 3-5 mm. A basaltic material (1/3 mm) shows a low water retention capacity. A high water-permeability and a low capillary rise cause a quick discharge into the base-layer and point at low evaporation rates.

By changing the design specification for the paving stones, the evaporation rates can be influenced. Starke et al. (2010b) showed that a change in colour from grey to anthracite increases the evaporation by about 19 % (in the middle European summer half-year). A change of grain sizes can also influence the evaporation rates. Prototype A, with fine grained concrete aggregates, evaporates 2.5 times more than the coarse grained prototype B. This is a result of the low water retention capacity and the lower capillary rise. So the water available for evaporation as well as the supply from the inner pore-system is less than of prototype A. The high difference between the water retention capacity and the effective pore volume shows that most pores are air filled. This air cushion can act as an effective evaporation break.

CONCLUSIONS

For the sustainable use of water-permeable pavements, the right choice of the street building materials is very important. With similar materials used in different combinations, it is possible to reach very different results. For example, evaporation can be further increased by about 20% (in regard to normal water-permeable pavements) to get a more near-natural water balance and prevent sultriness and dry heat in urban areas. Thus the increasing urbanisation and the consequent mega-cities can benefit in social, economic and ecological ways.

Furthermore, evaporation rates can be made even lower than from impermeable pavements. In regard to the high permeability of water-permeable pavements, this will lead to a higher groundwater recharge and so can benefit water supplies in arid regions. A further important effect can be dew-formation. It is assumed that porous surface promotes dew formation which can then infiltrate through the stone. This will increase the groundwater recharge further.

If it is unwished to increase the groundwater recharge, it can also be possible to store water in the sub-structure layers. If a water-permeable pavement is built on impermeable undergrounds, the storage capacity can be more than 30 l/m^2. The impermeable underground can be natural or clayey grounds or man-made, for example with water-impermeable geotextiles. The stored water is shielded from evaporation by the porous paving stones above and the capillary breaking base layer. This water can be used for irrigation or as raw water.

Based on this knowledge, water-permeable pavements can be used as one instrument in regional-specific rainwater management and ensure a prospective water supply.

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Artificial Recharge to Manage Groundwater Quantity and Quality in Bangladesh: GIS Mapping and other Investigations

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Abstract
Bangladesh depends heavily on groundwater for water supply and irrigation and development of this resource faces differing constraints at different parts of the country. As a result, access to water for drinking and irrigation is not uniform. Presence of arsenic in shallow groundwater is the most severe constraint over a large part of the country and salinity in groundwater is common in the coastal regions. Large scale withdrawals for irrigation and domestic uses cause water level to decline in areas like the Barind Tract and Dhaka City respectively. To mitigate the situation an action research has been undertaken to assess the potentials for artificial recharge using rainwater as a means of groundwater buffering. GIS analysis of relevant data was used to identify areas where groundwater buffering is necessary and feasible. Series of field investigations including shallow exploratory drilling, vertical electrical sounding, electric conductivity of existing water sources and field verification have been carried out to finalizethe location for field testing of artificial recharge systems. Innovative site specific designs have been made based on the results of investigations using indigenous techniques and materials. The GIS analysis proved to be a effective method to identify locations and characterize sites for artificial recharge in the coastal regions of Bangladesh. The same approach can be applied to other regions of the country where there is a need for managed aquifer recharge.
Key words: Bangladesh, groundwater, artificial recharge, water quality, GIS Mapping

INTRODUCTION
Groundwater resources are globally becoming more vulnerable due to population growth and climate changes (Vorosmarty et al., 2000). Managed aquifer recharge (MAR) or artificial recharge has long been considered as a management option for improving both quantity and quality of groundwater (Brown and Singor, 1974; Sakthivadivel, 2007; Thoa et al, 2008). Groundwater has long been the chief source of safe water and irrigation to attain food security in Bangladesh. However, occurrence of arsenic and other quality parameters above acceptable limits is gradually making the groundwater unsuitable for drinking and cooking usage (BGS & DPHE, 2000). Also water levels are declining very fast due to impacts of over exploitation in urban areas (Hoque et al, 2007) and irrigation pumping in rural areas (Shamsudduha et al, 2009). Groundwater in Bangladesh is also vulnerable to climate changes, particularly in coastal areas.

Despite large dependency on this vital resource groundwater management remains in a pitiful state in Bangladesh. However, the scenario is changing and use of managed aquifer recharges (MAR) to resolve water quality and quantity issues are being discussed these days. A study was carried out recently to assess the applicability of AR for managing groundwater resources in the Dhaka City (Sultana, 2009). Current paper describes another study conducted by the consortium of Department of Public Health Engineering (DPHE), Department of Geology, University of Dhaka and Acacia Water (Netherlands) with funding support from UNICEF Bangladesh.

In the first phase of this study few candidate sites were chosen to test the potentials application and suitability of MAR in Bangladesh. Remote sensing, GIS mapping, geophysical survey, exploratory drilling, electrical conductivity survey of existing water sources, laboratory analysis of water and sediment samples were conducted to identify these locations and characterize the sites. The methods has been chosen based on experiences in similar hydrogeological environments in different countries including India (Sehgal, 2008; Saravi et al., 2006; Ghayoumian et al., 2005; Anbazhagan et al., 2005; Ting et al., 2002; Bouwer, 2002; Rushton and Phadtare, 1989). In this paper we are presenting methods results and findings of image analysis, GIS mapping, geophysical survey, EC survey, water and sediment analyses in order to identify the locations and characterize individual sites for testing the applicability of AR for improving water quality and quality.

METHODS

Data Collection

All relevant existing data from various agencies have been collected for interpretation and mapping. Table 1 presents the type, nature and source of data collected for the study.

No	DATA TYPE	NATURE OF DATA	SOURCE
1	Groundwater Quality	Point	BGS-DPHE Study
2	Rainfall	Point	BWDB
3	Evaporation	Point	BMD
4	Temperature	Point	BMD
5	Groundwater Level	Point	BWDB
6	Surface Water Level		BWDB
7	Ground Elevation	Point	PWD
8	Flood Level		FFWC
9	Tide Level		BIWTA
10	Water bodies	Polygon	DPHE-DU
11	Rivers	Polyline	DPHE-DU
12	Saline Area	Raster (ASTER image)	DU
13	Soil Texture	Polyline	SRDI
14	Area of Irrigation	Polygon	BADC
15	Population	Point	BBS
16	Water Consumption	Polygon	BADC, BBS
17	Bore log	Point	DPHE
18	Base Map Data Layers	All layers as shape files	DPHE-DU

GIS Mapping

From the GIS analysis of the source data various thematic layers such as areas with poor water quality (occurrences of As, Fe and Mn above allowable limits), areas with saline groundwater and declining groundwater level, have been generated using ArcGIS 9.2 mapping package. Areas with no local sources of surface water have been mapped using 1 km buffer along the perennial rivers. Then these layers were ranked, reclassified, overlaid and analyzed using GIS tools. Overlaying of these various groundwater parameters through union and intersection GIS analysis helped to identify areas where the water quality and /or quantity did not meet the desirable standards. Further sifting of these areas were carried out based on logistics and other local conditions to finally select the areas to conduct water buffering techniques.

Image Analysis

High resolution Quickbird images (1x1m) from Digital Globe, freely available from Google Earth (Image acquisition date 6 November, 2002) and moderate resolution ASTER images (15x15 m, Image acquisition date 14 November, 2004) have been used for the study to map the landuse-landcover of the selected areas. The settlement areas with roof tops and ponds/water bodies were of primary interest to the study and these were mapped from the Quickbird images. These were then mosaicked using Erdas Imagine (image processing software) and the mosaic was geo-referenced with aid of the ASTER image.

Field Verification of Image

Printed maps from the georeferenced images were used to locate the sites that have been delineate by GIS analysis. However all proposed sites have been visited during field work and a number of GPS coordinate have been collected to verify the locational accuracy of the image. An inspection of current water use, water sources and water storage (including rain water harvesting and PSF) has been carried out. Field verification of features such as water bodies and roofs has also been carried out.

Exploratory Drilling and Sediment Sampling

Exploratory drilling was carried out in 20 sires in the selected locations using hand percussion reverse circulation drilling method to construct the subsurface lithological borelog. Disturbed sediment samples were collected at 1.5m interval for sieve analysis.

Grain size Analysis

Two to three aquifer sand samples were selected from each exploratory drilling site based on the depth and variations in grain size. 100g of dried sediment samples have been sieved following the standard method using 0.5 phi interval. Standard histogram and cumulative curve were prepared from grain size data and statistical parameters were computed on the basis of the formulae and verbal scale proposed by Folk (1966, 1974). Hydraulic conductivity have been estimated from the grain size analysis using available methods.

Vertical Electrical Sounding

Vertical electrical soundings (VES) were carried out using Schlumberger configuration at 38 points in the target locations. Current and potential electrodes were maintained at the same relative distances and the coverage was progressively expanded around a fixed central point. At each location, an array of 100m provided measurements up to a depth of 50m. Apparent resistivities of the subsurface materials were matched with the master curves to obtain true resistivity values. The results interpreted from VES data were later validated using the lithologs prepared for the test boreholes drilled at the study locations.

EC Survey and Water Sampling

Groundwater samples were collected from hand tubewells and ponds in and around the study locations. Specific electrical conductivity has been measured during sampling using a portable EC meter with temperature compensation (Hanna instrument HI 8033, Portable multi-range conductivity meter). A GARMIN 12 channel GPS was used to record the coordinates of every sampling points. All relevant site information including depth of wells was recorded.

RESULTS AND DISCUSSION

GIS Mapping for Delineating Target Areas

Series of GIS maps were generated using various water quality parameters, occurrence of surface water and depth of groundwater level for the identification of the target locations with acute water quality and quantity problems. Parameters such as areas with poor water quality (As, Fe, Mn above allowable limits), areas with deep water tables (>7.5m), no nearby surface water (1km from perennial rivers), and areas with shallow groundwater salinity have been used for this purpose.

BGS & DPHE, (2000) data base has been used to map the areas with poor groundwater quality as shown in Figure 1a. A second GIS layer was generated from the 987 BWDB monitoring well data depicting the water table conditions of 2006. This data were classified into two categories; (a) where the groundwater table lies below 7.5m depth, and (b) where the groundwater table is within 7.5m from the ground surface. The map of all the perennial water bodies and rivers were overlaid with this depth to groundwater level map to find areas where water table was more than 7.5m and no surface water body within 1 km. To obtain this map, a 1 km wide zone was created along the boundary of the water bodies and rivers using ‘buffer’ analysis of GIS. Then this layer was merged with the depth to water level map and queried to find out the desired areas as presented in **Error! Reference source not found.**1b.

The salinity map was derived by visual processing of ASTER images of winter 2001-2003 and from these images growing crops in the field were identified. Saline areas were delineated as the areas in the south where crops were not visible. It is known that all winter crops in Bangladesh require irrigation either from surface or subsurface sources. When cultivation is not practiced in winter over a sizable region, the underlying assumption is that in those areas water from both surface and subsurface sources are saline and not suitable for irrigation and hence cultivation of crops. This was cross-checked with irrigation data from Bangladesh Agricultural Development Corporation (BADC). The resulting salinity map is shown in Figure 1c.

Finally all the three maps have been overlaid to identify the areas where water conditions are acute in terms of either quality or quantity or both (Figure 1d). A number of such areas have been selected for further investigations in order to test the applicability of AR. To comply with UNICEF strategy, areas in the southwestern coastal region have been selected for the preliminary investigations.

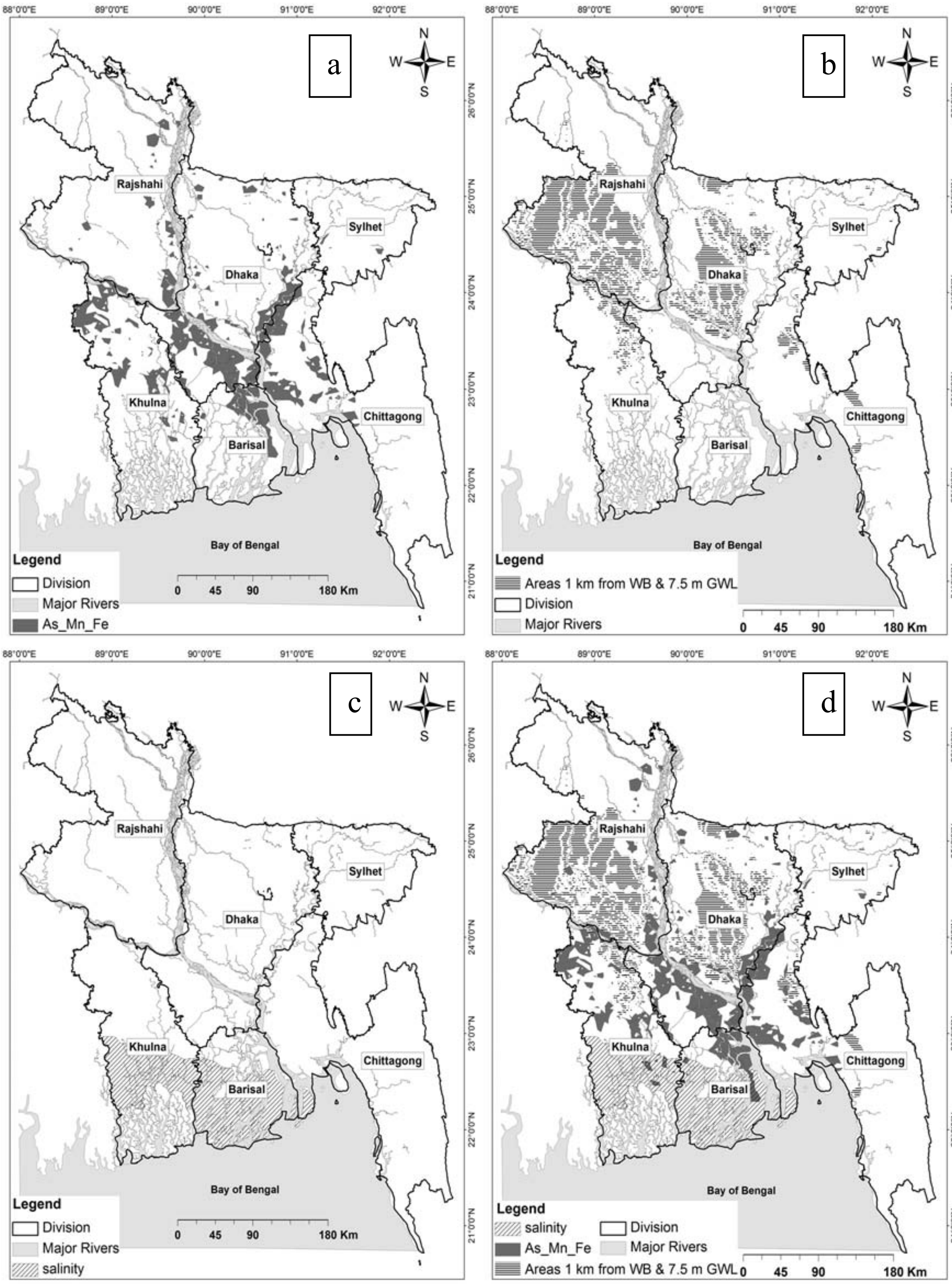

Figure 1: GIS mapping for identification of target areas for artificial recharge testing sites: a) areas with water quality problems, b) areas with water level deeper than 7.5 m and surface water within 1 km, c) areas with high salinity in shallow groundwater, d) overlay map (of the three maps with various parameters) showing the areas with acute groundwater quality and quantity problem.

Investigations carried out in the target locations

Image Analysis

After identifying the target locations on national scale map, satellite images was used to study the conditions important for artificial recharge for the target locations. Features such as surface water bodies, suitable roofs, access, land use etc have been studied.

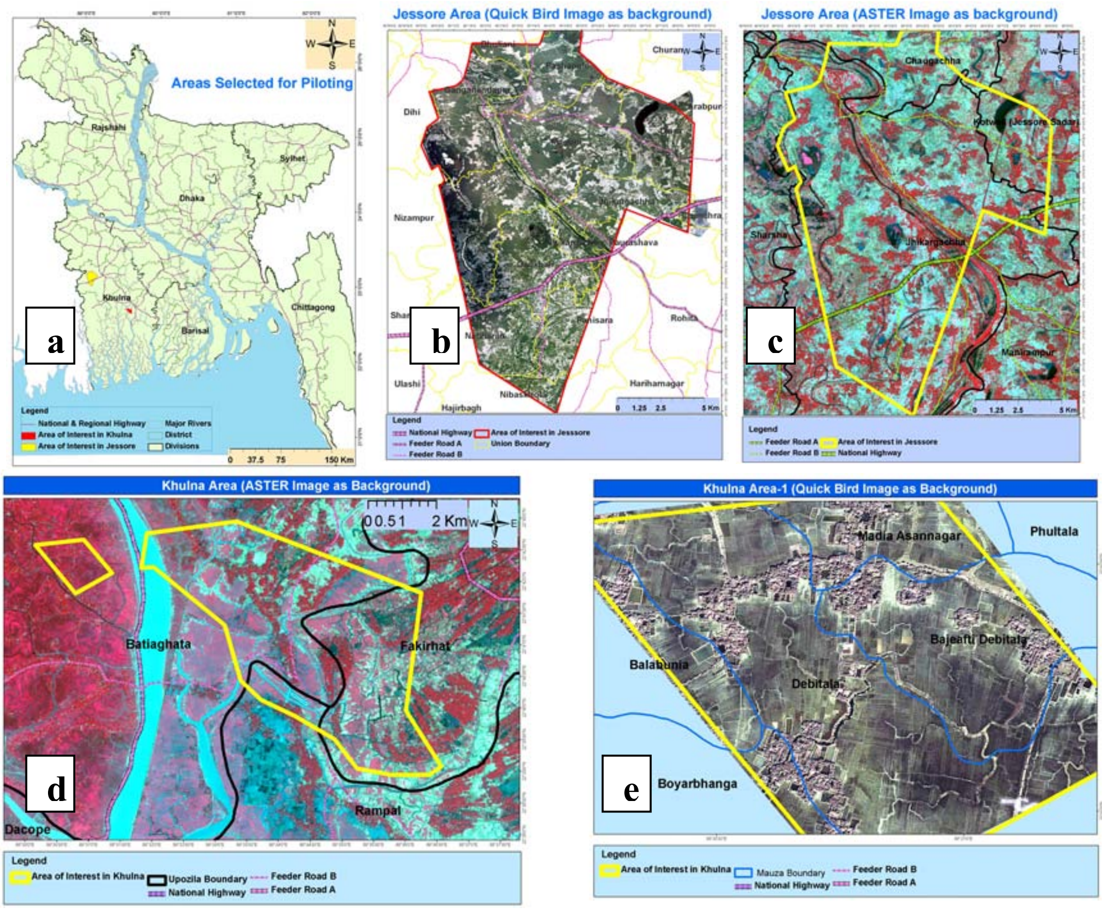

Figure 2: Image analysis carried out in the two target areas a) Two blocks identified for field investigations (yellow and red north and south of Khulna); b) Quick Bird image and c) AESTER image of Jessore area used for identification of roofs, ponds, surface features; d) ASTER image of Khulna and e) Quick Bird image of part of Khulna

Subsurface Resistivity Layers

VES data from each target locations have been used to draw subsurface resistivity sections which is reflection of lithology and nature of formation fluid. There is significant local scale variability in the subsurface lithology and salt content of the groundwater in the porous horizons. In Khulna area, resistivity of the sand zone varies from 2 to 6 Ωm except at one where it is 20 Ωm indicating that water content higher amount of dissolved solids. In contrast, resistivity values in Jessore area ranges from 28 to greater than100 Ωm indicating presence of fresh water in the shallow aquifer.

Figure 3 presents two electrical resistivity sections from two probable test sites. A shallow aquifer has been identified in both locations which can be targeted for artificial recharge. In one case there is no overlying clay indicating that conditions are suitable for natural recharge. However, in the other location the shallow aquifer is overlain by a thin clay layer and conditions are not ideal for natural recharge. The shallow aquifer with upper clay has been selected for testing of artificial recharge.

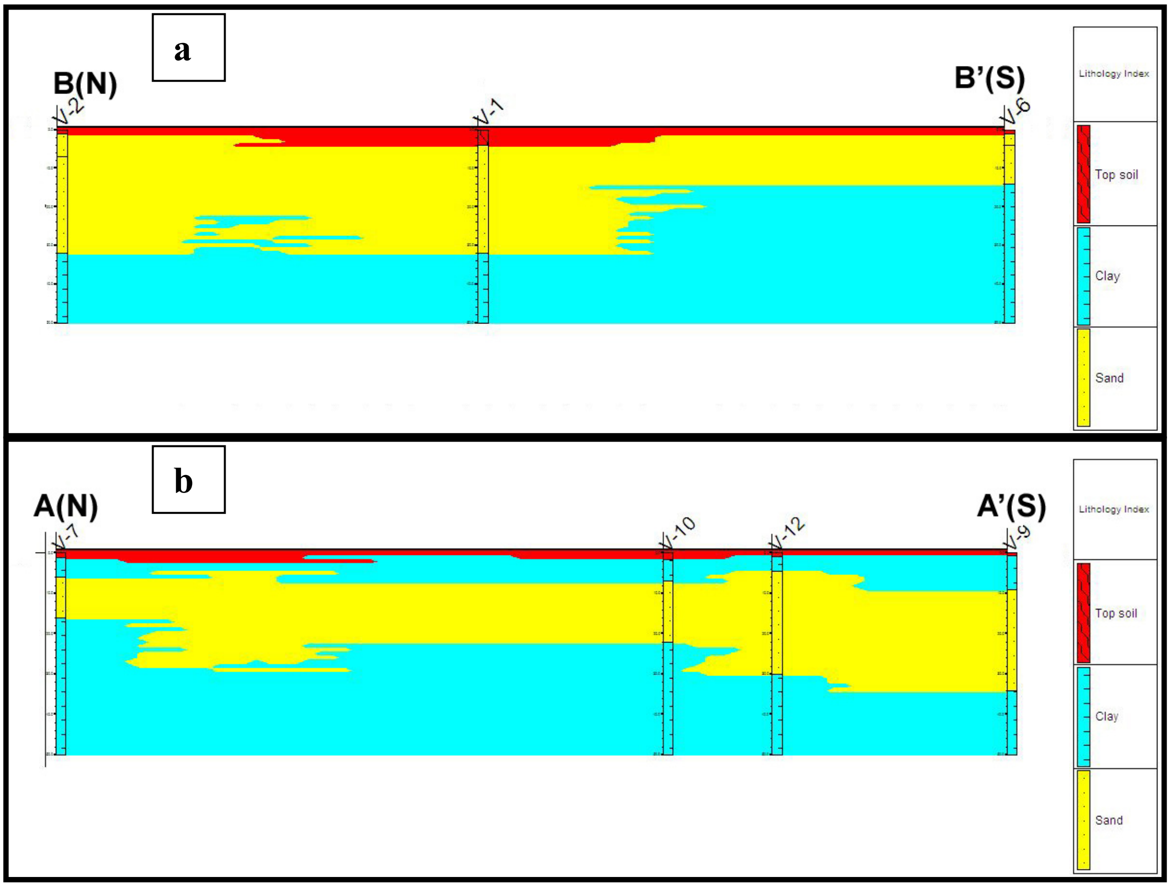

Figure 3: Subsurface electrical resistivity sections of the two target areas showing varying lithology and salt content in the aquifer: a) shallow aquifer with no overlying clay and b) shallow aquifer with a thin overlying clay.

EC Survey and water Sampling

Electrical conductivity of existing surface water sources such as pond, canal and river, and groundwater sources such as shallow well and deep wells have been measured in the field. There is high variability in EC of groundwater from the shallow aquifer where it reaches up to 10,000 μS/cm at certain locations. EC in the entire target area is high with occasional pocket of relatively low EC groundwater.

Rainfall and Shallow Groundwater Level Fluctuations

Daily rainfall data for the target study locations have been summarized into monthly total and weekly groundwater level data has been summarized to mean monthly data. Figure 4 presents the hydrograph of rainfall and water level elevation for the two locations. It is evident from the figure that there is enough rainfall during the pre-monsoon and monsoon periods with annual total in access to 2000 mm.

Groundwater levels fluctuate in response to the rainfall event, i.e. levels start declining as soon as the rain stops in October and continues to decline until May when it reaches the minimum elevation. Water level starts rising in May and continues to rise until reaches the peak in October and the annual fluctuations are 1.5 to 2.5 m in the target locations.

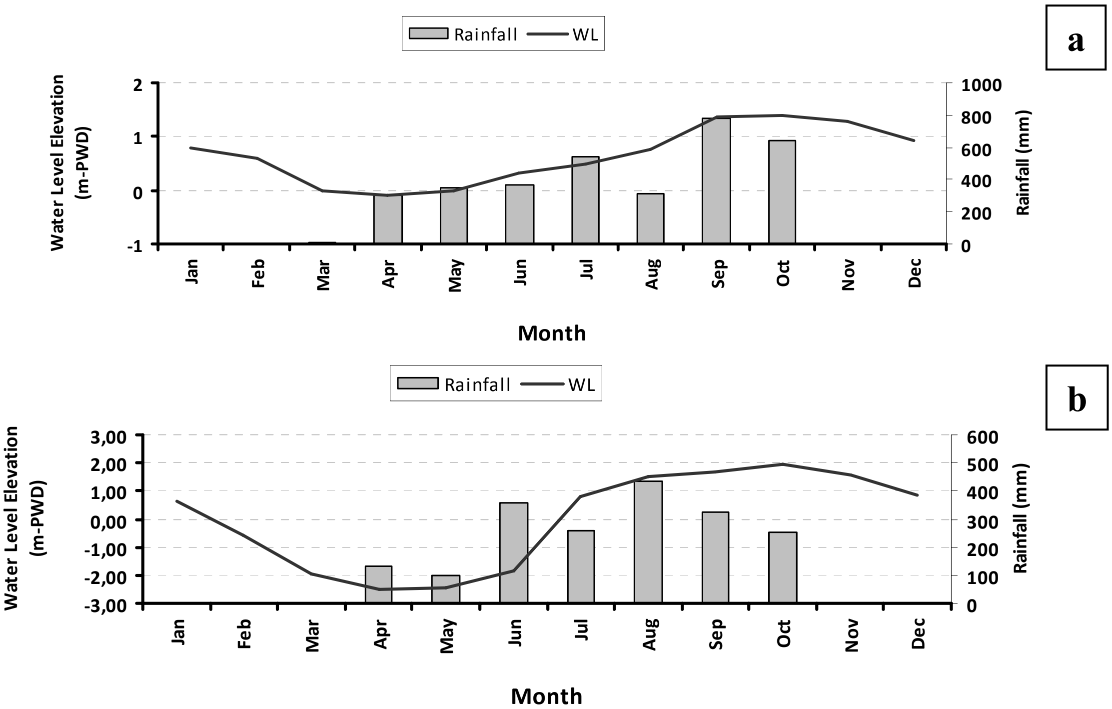

Figure 4: Pattern of rainfall and groundwater level fluctuations in the pilot study areas: a) Satkhira; b) Dacope, Khulna (Rainfall data from Bangladesh Meteorology Department; groundwater level data from Bangladesh water Development Board).

CONCLUSIONS

The concept of artificial recharge is new in Bangladesh and no field studies have so far been completed. This is the first systematic approach to apply this management options particularly in the coastal areas. There are published papers from neighboring India about the successful utilization of the approach in urban, rural and coastal areas. We have undertaken similar approach for identifying the locations for testing the applicability of MAR in Bangladesh where there is plenty of rainfall during the monsoon. However, the main challenge is to identify areas suitable for application of MAR. We have used the field investigation data to design site specific infiltration systems. The sites are being constructed using innovative design, indigenous techniques and locally available materials.

We have demonstrated that RS, GIS mapping along with field investigations are essential in finding suitable locations for MAR. It was not possible to test the sites for the quantity of water that can be infiltrated and changes in quality due to mixing of infiltration water and aquifer water. It is possible to upscale the AR techniques outside the coastal area using the same approach for identification of specific sites and designing the infiltration systems. Monitoring results on quantity of infiltrated water and changes in resultant water quality are vital is assessing the technology as management option for improving water quantity and quality in areas where current availability is low.

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The Role of MAR in Sustainable Water Supply A Case Study-IGI Airport, Delhi

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ABSTRACT
Artificial recharge and water conservation techniques for augmentation of groundwater resources are finding fast application in India to cope up with the increasing demands for the newly established infrastructures. The success however depends on proper understanding of the site conditions and the identification of suitable methods for recharge.

Indira Gandhi International (IGI) Airport, Delhi is spread over an area of 21 sq.km out of which, 3sq.km is paved area which is likely to be increased to 11sq.km. In view of the ongoing developments the demand for water resources is increasing and to mitigate the stipulated demand; water conservation and artificial recharge to aquifer systems become the best management options. The declining trend of ground water levels emphasize on the need to utilize the monsoon runoff for recharging the depleting aquifer to enhance sustainability of aquifer system, improve the water quality and save on the energy cost.

The area is hydro geologically very sensitive as the fresh water aquifers are underlain by saline aquifers. It is underlain with Quaternary alluvial sediments comprising of sand, silt, and clay admixed with kankar. The depth to water level in the area is reported to be varying from 30 m to 35 m bgl and the water level is declining at the rate of 0.6 m/yr.In the drain the total open area is about 22 km^2 and estimated runoff is 4.57 MCM. In addition to reported 62 recharge wells, 46 more recharge wells are required for accumulating 4.57MCM/yr runoff. In buildings, the total runoff estimated is 0.03 MCM of which 0.009 MCM can be captured and efficiently used for recharging the depleted aquifer

KEYWORDS: Monsoon runoff, Artificial recharge,Sustainable,Groundwater Resources,Annual recharge

1.0 INTRODUCTION

With the continuous growth of population & necessity to develop/enlarge infrastructure facilities, the demand for water has continuously being increasing. The area which could not be served with the piped water supply depends on the ground water resources. The groundwater development in Delhi is 170%, the Annual ground water replenishment being 297 MCM & Annual groundwater draft 509 MCM, with the result that the water levels are continuously declining at the rate of 0.6m/yr and in the last one decade the decline is recorded about 6-10 m. In IGI airport, Delhi, covering 21 sq. km area the water supply is mostly from the Groundwater and about 40 tube wells are in operation for a cumulative discharge of 3.2 mlpd. The unsaturated thickness is increasing and also the energy input to pump the water.

The annual rainfall is about 611mm/yr & a total of 4.57 MCM runoff is non committed which is planned to be used for recharge through the construction of different recharge structures. Out of 4.57 MCM only 26,156 Cubic meter or .03 MCM of rainwater harvested is being utilized for supplementing the sub-surface storages as there is no reported construction of artificial recharge structures.

2.0 GEOLOGICAL & HYDROGEOLOGICAL SETUP

2.1 Geology

Geologically, the area comprises of quartzites of Delhi Supergroup (~1500 m.y), occasionally interbedded with mica schist and intruded by pegmatites. These rocks are uncomformably overlain by Quaternary to Recent unconsolidated alluvial sediments (> 1.64 m.y) covering about

80% of the area.Older alluvium comprises of interbedded & lenticular deposits of clay, silt & sand ranging from very fine to coarse grain with occasional occurrence of kankar (calcareous nodules). The thickness of the alluvium is highly variable and is dependent mainly on the configuration of the basement. Newer Alluvium is confined to the flood plains of Yamuna River. It comprises of clay/silt mixed with small mica flakes, and medium to coarse-grained sand and gravel. The bedrock in the southern part of Delhi is overlain by the Aeolian deposits which are mainly loam, silty loam & sandy loam

The exploratory drilling & the tubewell data in and around the airport area show that the major part of sub surface lithology is made up of impervious clay or clay with kankar and sand & the depth to bedrock is at about 180 m or more.

Based on the drill hole data and geophysical survey a fence diagram **(Fig.1)** is prepared which clearly indicates that except in the eastern extremity the entire thickness of nearly 100 to 150 mts is predominantly silty-sand with intercalations of varying thickness of sand horizons. The thickness of alluvium has been reported to be more than 200 mts in western part of the IGI airport towards Dwarka area adjacent to the project area, the basement rock has not been encountered up to the explored depth. The entire project area is underlain by thick pile of alluvial sediments and seems to be more productive, the tube wells in these areas are capable of yielding 500 to 600 LPM for moderate drawdown.

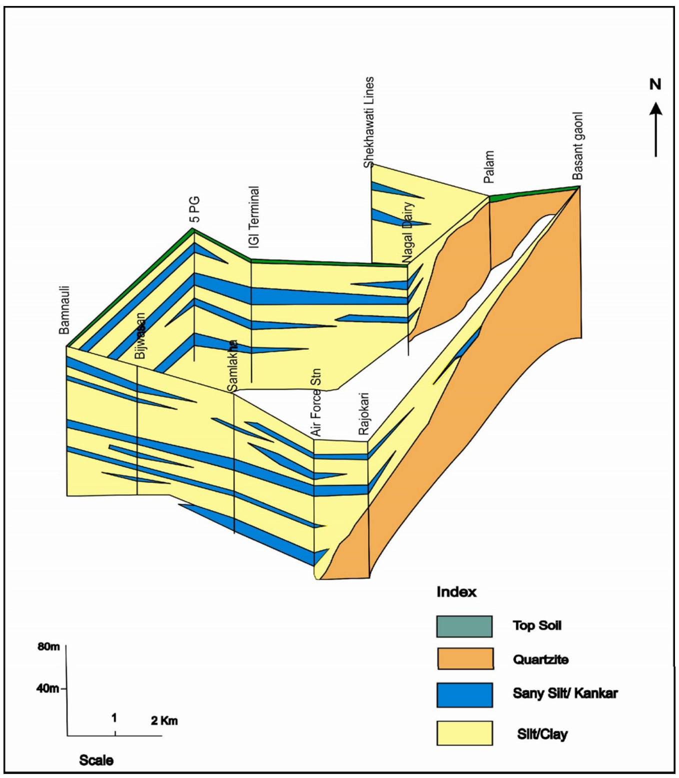


Figure 1: Fence diagram showing aquifer disposition around IGI,Airport

2.2 Hydrogeology

It is observed that hydrogeological conditions are fast changing because of the continuous development of ground water resources irrespective of its annual replenishment and aquifer sustainability. The water table is continuously declining and the ingress of brackish/ saline aquifers towards fresh aquifers is disturbing the saline-fresh interface. The decline in water table is continuously monitored and between the years 1998 and 2008, the water table has declined in varying degrees .95% fall in water levels is observed from 0.49 to 4m **(Fig. 2).** Only 5% wells of North –West, West and New Delhi have observed a rise in water levels in range of 0-2m because the ground water is not developed as the ground water quality is not good.

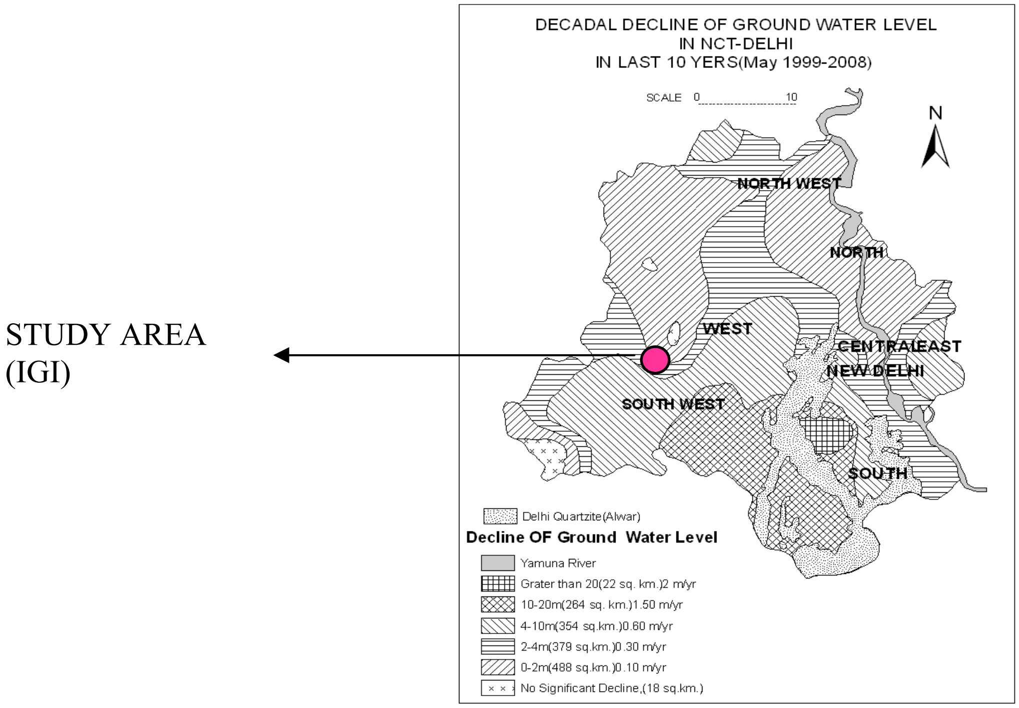


Figure 2: Decadal decline in groundwater level in NCT, Delhi (1999-2008) (After CGWB)

The decline in water table is recorded through a network of 98 observation wells four times in a year.the behaviour of water table near the airport is given in **Fig. 3.**It shows the decline in water table by 20 m in about 12 years.

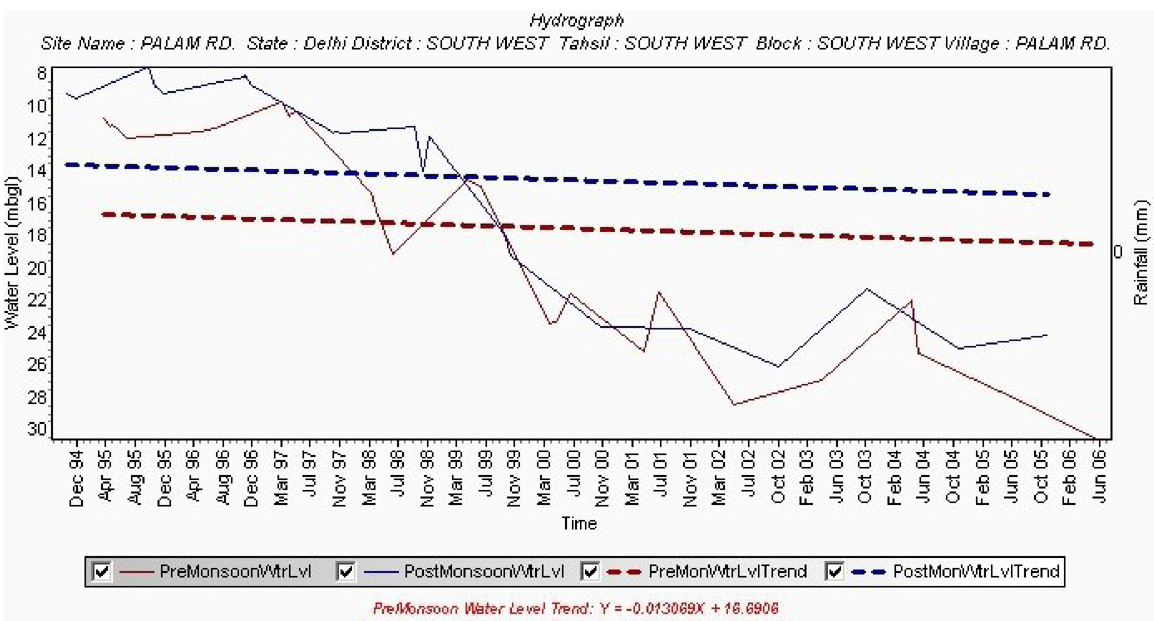


Figure 3: Hydrograph of Wells at select Locations around Study Area

2.2.1 Saline fresh Interface

The distribution of saline – fresh water interface has been represented through 3D diagram as shown in **Fig. 4.** Electrical conductance (EC) and Total dissolved solids (TDS) are two major parameters indicative of overall quality of ground water except contamination from organic constituents or trace elements. EC is measured directly in the field to get an approximate idea about the extent of mineralization of ground water.

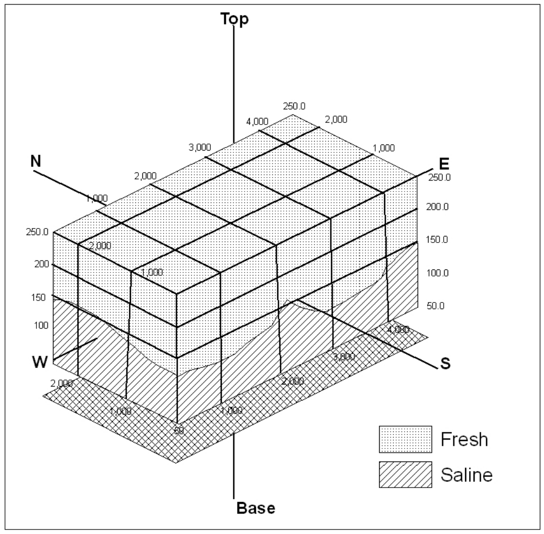
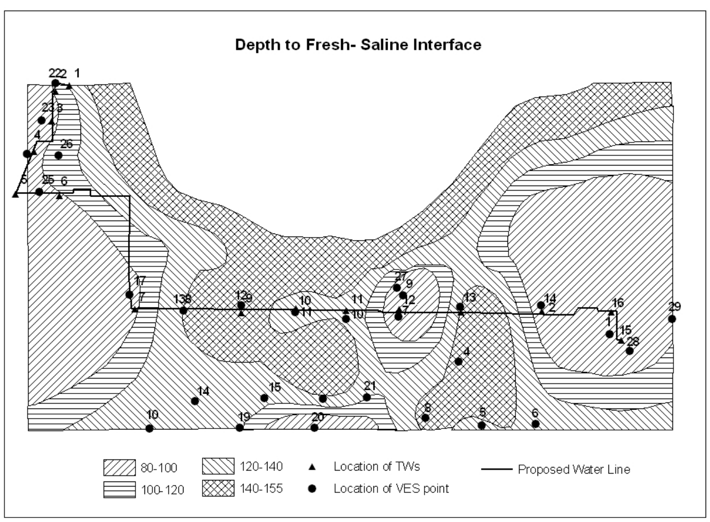


Figure 4: 3D Plot of Saline – Fresh water Interface

Chemical quality of ground water in the study area varies with depth and space. The fresh ground water aquifers are generally separated by clay layer. The aquifer below the clay layer is brackish to saline. A geochemical profile depicting the depth of saline–fresh water interface has been attempted based on the geophysical sounding data(**Fig. 5)**. It can be observed that in general the thickest fresh water bearing aquifer is available in the central part of the project area where the depth of saline–fresh water interface is deepest in the range of 140 to 150 m bgl.



Map Not To Scale

Figure 5: Geochemical Profile depicting the depth of saline–fresh water Interface

3.0 RAINFALL & RUNOFF POTENTIAL FOR RECHARGE

The normal annual rainfall of Delhi is about 611.8 mm. the rainfall increases from the southwest to the northwest. It is observed that in general, there are 27 rainy days (rainfall > 2.5mm) per year. Out of 27 days, during monsoon months the rainy days are 19. The peak rainfall intensity is 30mm/hr for 20mins. About 81% of the annual rainfall is received during the monsoon month's viz. July, August and September. The rainfall deviations are from -25.5 % to + 90 % from normal rainfall, affecting the natural recharge to ground water each year

Table 1: Annual rainfall and deviations from Normal Rainfall

District	Station	2001		2002		2003		2006		2007	
		Annual rainfall in mm	Deviation from Normal rainfall	Annual rainfall in mm	Deviation from Normal rainfall	Annual rainfall in mm	Deviation from Normal rainfall	Annual rainfall in mm	Deviation from Normal rainfall	Annual rainfall in mm	Deviation from Normal rainfall
New Delhi	Safdarjung	693.8	+ 13%	+ 561.2	- 8.3%	1161.3	+ 90%	754.60	+23.34%	827.20	+35.21%
	Rashtrapati Bhawan	585.0	- 4.4%	--	--	--	--	--	--	--	--
North	Delhi University	669.9	+ 9.5%	664.1	+ 8.55%	703.5	+ 15%	--	--	--	--
	Delhi Ridge	657.4	+ 7.4%	611.5	- 0.05%	558.3	- 8.64%	--	--	--	--
South west	Palam	529.0	- 13.5%	456.0	- 25.5%	881.4	+ 44%	422.50	-30.94%	630.20	+3.01%
South	Ayanagar	659.9	+ 7.4%	596	- 2.58%	1119.1	+ 83%	--	--	--	--
NCT, Delhi Mean		**632.0**	**+ 3.3%**	**572.47**	**- 6.43%**	**884.7**	**+ 44.6%**	--	--	--	--

The runoff from the paved & unpaved has been computed and given in **Table 2.**

Table 2: Total Runoff of Paved, Open Area and Building Area at IGI,Airport,Delhi.

1. Paved Area	
i. Total Area	$3x10^6$sqm
ii. Annual Rainfall	611mm
iii. Coefficient of runoff	0.7
iv. Total volume and runoff	1.28 MCM
2. Open Area	
i. Total Area	$18x10^6$sqm
ii. Annual Rainfall	611mm
iii. Coefficient of runoff	0.3
iv. Total volume and runoff	3.29 MCM
3. Building (Roof top) Area	
i. Total Area	$1x10^6$sqm
ii. Annual Rainfall	611mm
iii. Coefficient of runoff	0.9
iv. Total volume and runoff	0.55 MCM
Total Available Runoff	**5.12 MCM**

4.0 CONCEPTUALIZATION OF RECHARGE STRUCTURES

In view of the rapidly increasing water demand & the time lag in making the surface water supplies possible from the surface reservoirs, the artificial recharge was considered as the best immediate management option. The favorable hydrogeological set up, presence of unsaturated aquifer zones & the availability of sufficient monsoon runoff from different areas, make the airport area suitable for effective recharge.

A total volume of 26,156 cubic meter or 0.026 MCM is captured from the rooftop area of 47,560 sq.m, of different buildings(Terminal A,Terminal B ,Cargo Import and Export building)**(Table 3).**

Table 3: Monsoon runoff from different areas of airport

S.No	Building	Area sq.m.	Total volume of Rain water Harvested(Cu M)
1	Part I, Terminal 1A	7,000	3,850
2	Part II, Terminal 1A	7,660	4,212
3	Cargo Import Building	10,600	5,830
4	Cargo Export Building	10,600	5,830
5	Terminal 1B	11,700	6,434
	TOTAL	**47,560**	**26,156**

The designs of the recharge structures are given below in Fig 6,Fig.7 & Fig. 8:

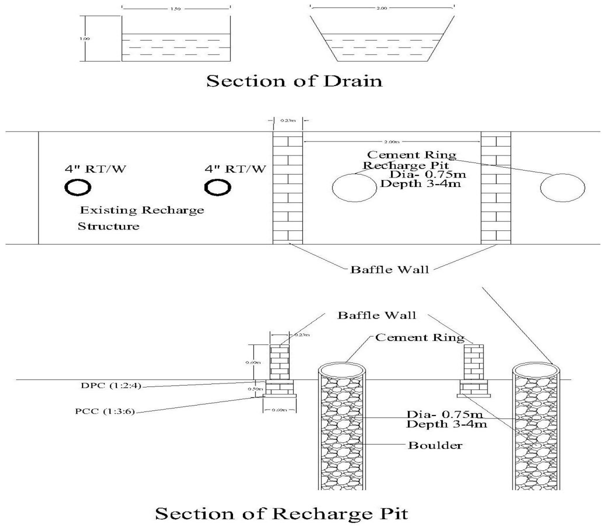


Figure 6: Recharge Structures in Drains at IGI, Delhi

There are about 62 artificial recharge structures constructed in the drains.
The part of the green area (unpaved) gets flooded during monsoon period restricting the movement of vehicles. In order to regulate the flow and reduce the flow, the gravel packed trench type structures are designed**(Fig. 7).**

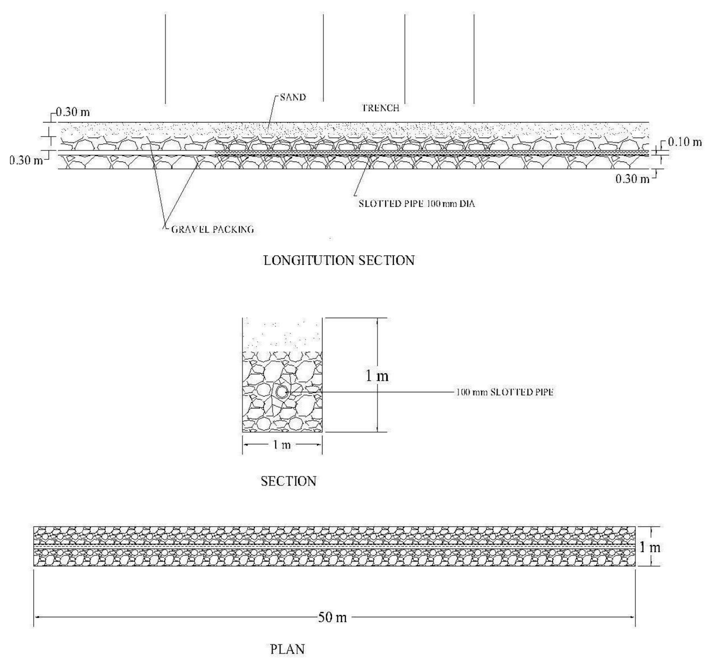


Figure 7: Rain Water Harvesting in Open Areas at IGI,Delhi

In case of recharge from the rooftop of different terminal buildings, the recharge will be made through a recharge well, extending in depth up to 50 m, i.e. below the depth of the water table.

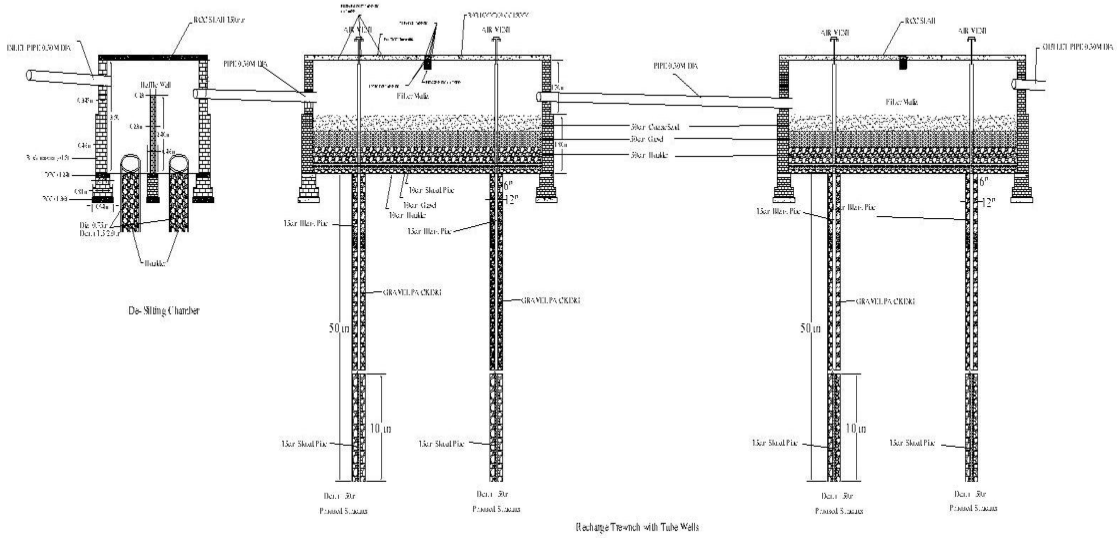


Figure 8: Section View of Rooftop Rain Water harvesting & Artificial Recharge Structure for Buildings (with Drain Pipes)

This is so because the granular zones are not vertically continuous & the different granular Zones are separated by thick clay layers, therefore the recharge has to be through a tube well/injection well.

5.0 CONCLUSION

The part implementation of the MAR project has shown the expected benefits, the decline in water level has been reduced and also the ingress of the saline water around the airport area. The total structures for the optimum utilization of the flow in the drain are estimated 260 out of which 62 have already been constructed i.e. project is in the process of full implementation. The runoff potential from the terminal buildings is estimated as 0.03 MCM the recharge of which will further check the declining trend of the water table and improve the water quality. The implementation of MAR is now mandatory and included in the urban by laws. Such demonstrative project is encouraging the other developers to take up MAR in all the infrastructures such as residential buildings, metro, highways etc. as part of the programme.

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Combining the improved soil moisture balance and water level fluctuation methods to estimate recharge: A case study from limestone aquifer

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ABSTRACT

Increasing demand for groundwater in Jaffna district are forcing to develop approaches for recharge estimates, building a more thorough understanding of recharge and groundwater balance studies. A relatively simple and practical approach for calculating groundwater resources in dry zone environment with relatively shallow water table in limestone aquifer is estimated and measured by using improved soil moisture balance method (ISMB) and water level fluctuation method (WTF) respectively for the period from April 2007 to April 2009. Water level fluctuation was measured on daily basis from sixty wells and analyzed by two approaches; graphical extrapolation and direct calculation. Results reveled that the ISMB model is a reliable approach for potential recharge estimation in a wide variety of situation. Direct check can be made of the recharge estimates, results from soil moisture balance calculations are compared with well hydrograph. Groundwater recharge rates of the limestone aquifer range from 23% to 25% of annual rainfall as determined ISMB model for 2007 and 2008 whereas by WTF it was 19% and 27% of annual rainfall. There is an acceptable agreement in identified main period of recharge by both methods. But graphical extrapolation mostly agrees with result of the ISMB. Hence the model ISMB and graphical expression of WTF could be a useful tool to estimate the recharge in limestone aquifer.

Key words: Limestone aquife , Recharge estimation, Soil moisture balance, Water table fluctuation,

INTRODUCTION

Quantification of the rate of groundwater recharge is a basic pre-requisite for efficient groundwater resource management. It is particularly important in regions with large demands for groundwater supplies like as Jaffna peninsula, Sri Lanka where such resources are the key factor for life and to economic development. Because groundwater is the only source of water for domestic, agriculture, industry and water supply schemes with seasonal rainfall. In recent years, increasing demand for groundwater has raised concerns about resource sustainability and has highlighted the need for reliable estimates of groundwater recharge. Groundwater is a renewable resource, hence it is not only sufficient to assess the potential of groundwater but it is also necessary to manage it efficiently, so that long term benefits can be achieved. Knowledge of groundwater recharge is essential in virtually all groundwater hydrology investigation and it is depending on the application, which needs to be estimated at a variety of spatial and temporal scale as stated by Delin *et al*., 2007 and Scanlon and Cook, 2002.

While groundwater recharge is one of the most important parameters required to support sustainable management of groundwater resources, it is one of the most difficult to evaluate accurately, due to the numerous factors involved in recharge processes. Groundwater recharge is that amount of surface water which reaches the permanent water table either by direct contact in the riparian zone or by downward percolation through the overlying zone of aeration (Rushton and Ward, 1979). Also De Vries and Simmers, 2002 defined as in a general sense as the downward flow of water reaching the water table, forming an addition to the groundwater reservoir. It really expresses the total quantity of groundwater resource available and their supply potential.

Many different approaches exist for estimating recharge. Statement of Scanlon *et al.*, 2002 emphasized that choosing an appropriate technique for a particular site is not straight forward and depend on several factors including field constrains and availability of field data. However technique based on groundwater levels are among the most widely applied methods for estimating recharge rates if well hydrographs are available (Healy and Cook, 2002). Scanlon and Cook, 2002 considers many aspects of groundwater recharge estimation. Reviews of methods applicable to semi-arid and arid environments state the importance of using a variety of independent techniques, as they can complement each other in terms of time and space scales (Scanlon *et al.,* 2006*).* Conrad *et al*., 2004 stated that chloride mass balance method was used to identify the main source of recharge namely direct recharge, recharge from river flow and lateral inflow and also stated that Geographic Information System based approach could be used to address the significant spatial variability of recharge. De Silva (2005) mentioned about the spatial variability of recharge using chloride profiling method and stated that spatial variability in recharge was observed over small areas in dry zone in Sri Lanka.

Rushton, 2003 developed improved soil moisture balance method and applied in different situations and climatic conditions. Chen *et al*., 2005 stated that the standard techniques of estimating recharge most often involve applying a soil moisture budget where the moisture content of the soil is traced through the time. Rushton *et at*., 2006 used soil moisture balance method to estimate the recharge by incorporating data from meteorological and field situation insights. Sophocleous, 1985 stated that a practical, simple and generally reliable procedure for estimating natural groundwater recharge in relatively flat areas with a shallow water table less than 10 m is to combine the soil water balance analysis with a corresponding analysis of water table rises. The selected study area consists both characters means flat areas with shallow water table. Hence the objectives of this study is aimed as estimating the recharge by modified soil moisture balance method in different field situation and water table fluctuation methods and to observe suitability and the correlation between each other.

MATERIALS AND METHODS

Nature of water resources in the study area

The Jaffna Peninsula lies in the Northern most part of Sri Lanka. The Jaffna peninsula experiences typical dry zone climate of Sri Lanka, characterized by a wet and a dry season. The major rainy season occurs during October to December due to the North-East monsoon and the minor rainy season occurs during April and May due to the South-West monsoon with an average rainfall of 1300 mm annually. The recharge is taking place mainly during major rainy season from October to December. The Peninsula is dependent on groundwater for all its water requirements. The one surface stream, Valukai Aru is active only during the height of the monsoon and there are no reservoirs of a perennial nature. The Peninsula is uniformly low with a maximum elevation of about 10 m, has an area of 1000 km^2. The limestone is the main aquifer of groundwater. This aquifer has several isolated caves and caverns capable of storing groundwater without evaporation losses. The entire groundwater is generated from percolated rainfall and its forms a freshwater lens beneath the Peninsula. It is found that the freshwater lens do not extend below the base of the limestone. This freshwater lens is sustained by the buoyancy of freshwater in relation to seawater.

Improved soil moisture balance method (Rushton, 2003)

The method provides periodical estimates of direct recharge based on changes in the moisture content of the soil. A daily estimate of the soil moisture balance is made with an input of precipitation plus irrigation plus near surface soil storage minus run off and losses due to actual evapotranspiration and drainage, which may include aquifer recharge. According to the model, direct recharge occurs when the soil moisture content reaches field capacity. At the field capacity, any additional net influx of water will not be stored within the soil but will drain to under laying aquifer. To determine when the soil reaches this free draining condition, it is necessary to simulate soil moisture conditions on daily basis throughout the year.

Water table fluctuation method

The water-table fluctuation method (WTF) is based on the premise that rises in groundwater levels in unconfined aquifers are due to recharge arriving at the water table. In an unconfined aquifer water table serves as the upper surface of the zone of saturation in which groundwater table can fluctuate freely. Healy and Cook, 2002 stated that WTF approach is applicable only to unconfined aquifer. Recharge is calculated as the change in water level over time multiplied by specific yield. This approach is a gross simplification of a very complex phenomenon, namely, movement of water to the water table. The method is based on relating changes in measured water table elevation with changes in the amount of water stored in the aquifer (Delin *et al*., 2007)

$R(t_j) = Sy\ \Delta H(t_j)$

In which

- t_o - Initial time
- t_j -Time taken to reach the peak water table
- $R(t_j)$ - Recharge occurring between times t_o and t_j in cm.
- Sy - Specific yield and
- $\Delta H(t_j)$ - the peak water table rise attributed to the recharge period (cm).

$\Delta H(t_j)$ is estimated as the difference between the peak of a water level rise and the value of the extrapolated antecedent recession curve at the time of the peak.

Two approaches were used to estimate $\Delta H(t_j)$ in the WTF method such as graphical extrapolation and direct calculation – RISE programme approach. In the graphical approach used in the WTF method the antecedent recession curves were extrapolated manually to obtain $\Delta H(t_j)$ on the basis of inspection of the entire data set. The RISE programme approach used in the WTF method calculated the daily rise of water levels in an observation well as the amount by which the water level on that day exceeded that of the previous day. If the result was negative it was set to zero for that day. Since the recession of the water level sharp or quick with short duration, the rise of the water level was estimated as the difference between the peak of the water level rise and the value of the extrapolated antecedent recession curve at the time of the peak (Delin *et al.,* 2007). The WTF is attractive if groundwater level observations are available even though WTF provides information on temporal and spatial recharge variation, also can be misleading if the water level fluctuations are confused with those resulting from pumping, barometric or other cases.

Collection of data

The three different cropping situations were selected for recharge estimation by ISMB such as unirrigated grassland with small trees, irrigated banana and irrigated chilli. Environmental parameters required for the estimation of potential evapotranspiration such as monthly average mean temperature, humidity, wind speed and sunshine hours were taken from the meteorological station, Jaffna. Crop data; date of planting, full emergence of crop, duration of initial, development, mid and late stage, date of harvesting, root zone depth and crop irrigated and the extent of land under irrigated crop, percentage of cultivable extent were recorded from the field. Irrigation practices; frequency of irrigation, rate of pumping and duration of pumping were monitored to estimate the irrigation amount. Crop coefficients for required crops were taken from Allen *et al*., 1998. The gravimetric water content at field capacity, permanent wilting point (Joshua, 1990) and bulk density (Black, 1986) were measured by normal field method, pressure plate apparatus and field method respectively. The magnitude of runoff was estimated as fraction of rainfall and related to rainfall intensity and soil moisture deficit. Daily water levels were measured by using dip meters at sixty wells from April 2007 to April 2009 from a variety of sites across the study area consisting different cropping pattern and land use. The specific yield of 0.27 was taken from pumping test analysis. Finally the comparison was done between recharge predicted from ISMB in different field with actual WTF.

RESULTS AND DISCUSSION

Recharge estimation through MSMB

Table 1 summarizes all important process such as total amount of recharge, the day in which recharge starts, number of recharge days and amount of recharge in each month from permanent grass, irrigated chilli and banana. It was found that there was very little variation in the annual recharge and no variation in number of recharge days. This has the additional advantage that the annual recharge is commonly used as a measure to control abstraction from aquifers in different cropping systems since annual recharge is not much varied. It explains that the moisture content at the end of the cropping season, amount of irrigation and irrigation interval influence the annual total and start of the recharge process.

Table 1: Summary of recharge process year 2007

	Permanent grass	Irrigated Chilli	Irrigated Banana
Total annual recharge (mm)	294.3	303.08	290.06
Beginning of the recharge	21st October	21st October	21st October
Number of recharge days			
January	Nil	Nil	01
October-	08	08	08
November	03	03	Nil
December	06	06	05
Amount of recharge (mm)			
January	-	-	36.9
October	126.43	132.12	115.11
November	11.62	11.62	0
December	156.26	159.34	138.05

Recharge estimation through WTF

The Figure 1 shows the annual groundwater level fluctuation with rainfall for selected wells from January to December 2008 as example. Most of the wells respond quickly to the onset of rainfall and their water levels start to drop shortly after the end of the rains. This response may be due to their shallow depth and located in unconfined aquifer. There were substantial differences in the responses of water level in monitored wells due to recharge in both years. Observation of well water level showed that difference response of groundwater fluctuations in time, rate and the space. Examples from selected wells are shown in figure 2 for the month of October 2007.

In order to estimate the actual recharge the water table rises were carefully observed along with the daily rainfall. The amount of recharge was not same in all the wells. Table 2 shows the average actual recharge from wells by groundwater table fluctuation method with standard deviation. The actual recharge was 26% and 29% of rainfall for 2007 and 2008 respectively.

Table 2: Recharge (mm) estimates from WTF method with standard deviation

Recharge month	*WTF method
October 2007	98 ± 15
November 2007	62 ± 8
December 2007	172 ± 12
March 2008	42 ± 5
October 2008	62 ±10
November 2008	460 ± 66

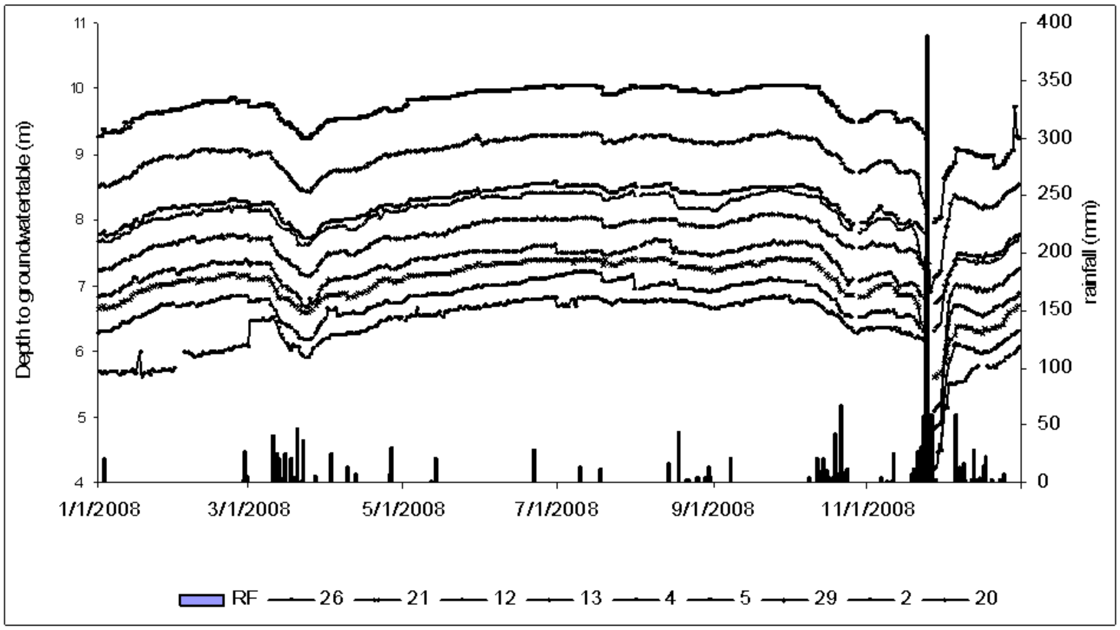

Figure 1: Water level fluctuation with rainfall - 2008

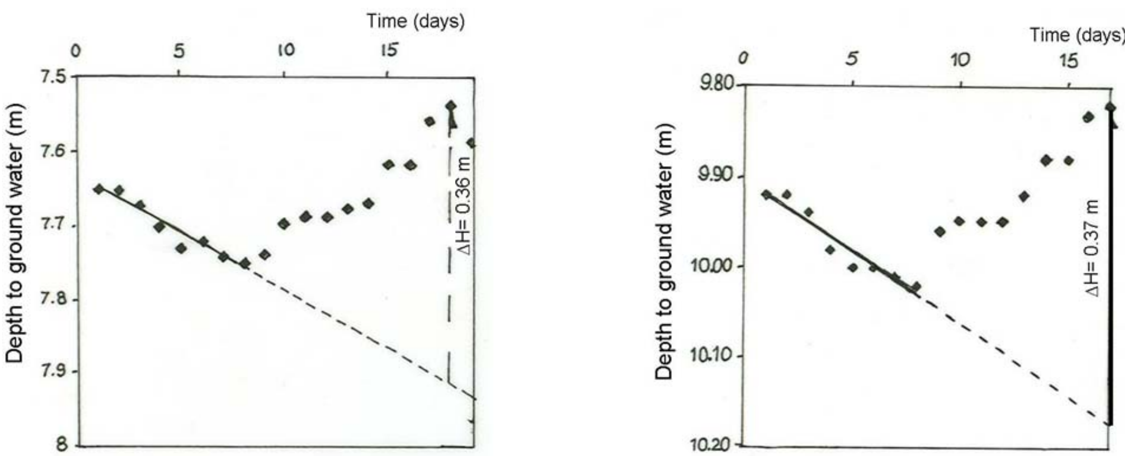

Figure 2: Estimation of rise of water table during October 2007 by extrapolation method

Relationship between MSMB and WTF

The actual recharge from WTF method and predicted recharge through ISMB in different field situation; permanent grass land, irrigated chilli and banana for 2007 was shown in Figure 3. Figure 4 shows the relationship between estimated recharge by ISMB model and to actual recharge estimated by WTF method for grass land. Groundwater recharge rates of the limestone aquifer range from 23% to 25% of annual rainfall as determined ISMB model for 2007 and 2008 whereas by WTF it was 19% and 27% of annual rainfall. The results indicate that assessment of the average annual recharge obtained with a ISMB and the WTF. There is an acceptable agreement in identified main period of recharge throughout the study period between recharge estimated from ISMB model and WTF method. Each method provides information about the temporal variation of recharge and correlation was very high (R^2 = 97%).

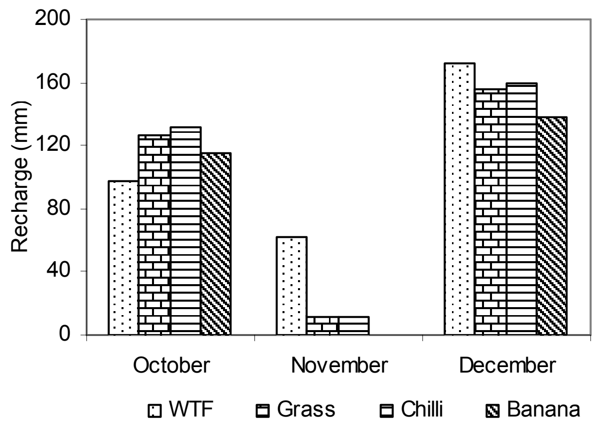

Figure 3: Predicted recharge pattern in different field and actual recharge by WTF

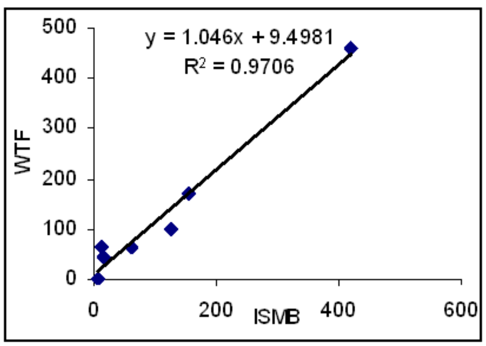

Figure 4: Correlation between recharge through ISMB and WTF

CONCLUSION

The WTF is a practical simple and generally reliable to estimate the recharge in relatively flat areas with a shallow water table. ISMB method could be used successfully to estimate recharge by modifying all parameters in different field situation. Since recharge estimation methods; ISMB and WTF are independent on each other, they can complement each other in terms of time and space. Also ISMB and WTF methods combinations are suitable since soil moisture balance represents the real field situation and water table fluctuation shows the real field situation. Hence combination of ISMB with corresponding analysis of WTF produce accurate estimate of recharge.

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Development of basins in a hard-rock terrain and their groundwater recharge potentialities: A case study

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ABSTRACT:

India is one of the fast developing countries of the world .Though a huge quantity of surface water (188 Mham) is available in our country, due to the topography, lithology and other controlling factors there is a limit of the amount of storage of water in the subsurface condition. The Kangsabati river basin of Purulia district, eastern India, bounded by latitude 23^0 05' to 23^0 30'N and longitude 86^0 to $86^0$20' E, has been chosen for a case study to evaluate the groundwater recharge potentiality in the drought prone hard-rock terrain and to understand the geomorphological evolution of the region.. The average annual rainfall of the region is 1180mm. Due to crystalline nature of the basement rock and uneven relief of the terrain, surface run-off is high . In the study area numerous basins mostly within Ajodhya plateau and in the adjoining area have been developed. There is a predominant structural and lithological control on the configuration of the basins. In many places planar surfaces act as water divide. Satellite image interpretation with selected field verifications and subsequently generated digital data base reveals that alluvial fans, planar surfaces with weathered residuum, palaeochannel, deep seated inter connected joints and fractures within the basins appear be potential rechargeable unsaturated zones. Overlay analysis involving several thematic layers like hydrogeomorphology, lineament number density and drainage density in the geographical information system (GIS) environment reveals the above mentioned findings. The digital terrain model (DTM) of the region reveals the generalized relief condition of the terrain . The presence of geomorphological features like stepped river terraces, inselberg, nick points (water falls), incised alluvial fans, parallel to sub-parallel hill slopes are all suggestive of the development of basins in multicyclic stages by the process of pediplanation aided by neotectonic activity.

KEY WORDS : Ajodhya plateau, rechargeable unsaturated zones, DTM

Introduction

The present study is concerned on the development of basins along with identification of groundwater recharge area and geomorphic features in and around Ajodhya hills (plateau), bounded by latitude 23°5' to 23°30° N; and longitude 86°0' to 86°20' E. The area has been evolved as a result of typical geomorphic processes operating along with the influence of tectonic activity. Geologically, the area is occupied by Precambrian granite gneissic complex, with general tectonic trend in East-West direction. Localized deviation due to secondary deformative forces acting from other direction(mostly North-South). Inspite of moderate precipitation (a.a.r 1140 mm), groundwater resource potential area is considered to be poor due to high amount of surface run off and evapotranspirational loss (50% Raghunath, 1982). Such loss has been ascribed to three major factors viz., (1) rocky nature of the terrain, (2) moderate to high gradient of slope, and (3) uneven distribution of rainfall in a year (90% of the precipitation takes place during rainy season). Here the summer month ranges between late March to early June, temperature 35°C to 46°C) and the winter is from late November to the end of February (temperature 10°C-13°C). South-West monsoon is the main source of rainfall. Average annual precipitation is about 1050 mm, bulk of precipitation takes place during July and August. This climatic feature is acceptable for the entire study area.

The high land of the study area is, mostly occupied by granite plutons (leuco granite) (Fig.1) with incipient gneissocity. The northern part of the area is relatively flat with isolated occurrence of bornhardt, inselbergs (Fig.1). The pediments in the area are often covered by loose debris and they are designated as mantled pediments. Presence of various planation surfaces at different elevations are probably due to gradual uplift and erosion The elevation in the different surfaces ranges between 520 metres to 620 metres in the southern sector of the study area while in the northern sectors of the study area the elevations vary, e.g. 500 metres to 300 metres. The low lying region, i.e.the Kangsabati R(KASAI N)(Figure 1) basin shows a transition in nature of sediments from colluvial (along the southern flanks of the Jabar hills) to latosol alluvium along Kangsabati R (KASAI N).

Methods:

The study is based on satellite data (IRS standard FCC data scale, 1:50.000), Survey of India topographical-sheet no. 73 1/3, 73 I/4, 73 1/7, (scale 1 :50,000) aided by field verifications. Third order and higher order basins have been studied in context of their large dimensions which facilitate recognition of lithology, structure and nature and extent of weathered zone. A detailed analysis of stream morphology and morphometry in relation to geology and lineaments has been carried out with the help of Survey of India (SOI) topographical sheet (scale 1:50,000) and geocoded satellite data (IRS-IB, Standard False Color Composite) (scale 1:50,000) aided by field verifications aiming towards the assessment of groundwater recharge potentiality of the study area. N) to colluvium and alluvium along the northern flank of Ajodhya hills(fig 2).3D terrain condition is represented by digital terrain model(DTM)(fig.3).

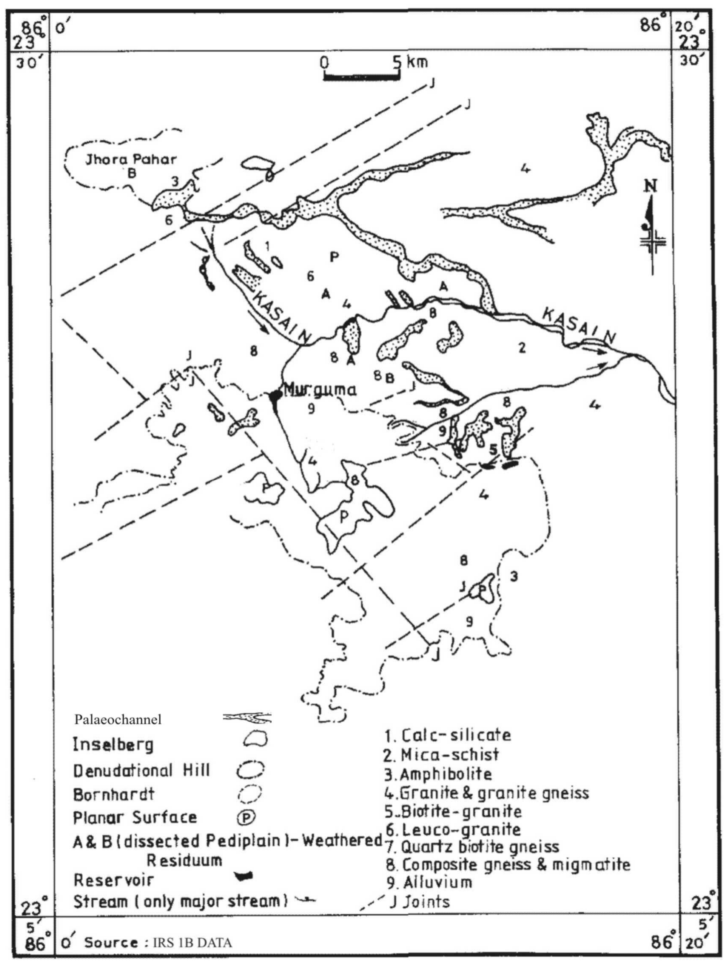

Figure 1 After Das2007

A generalized geological map of the area

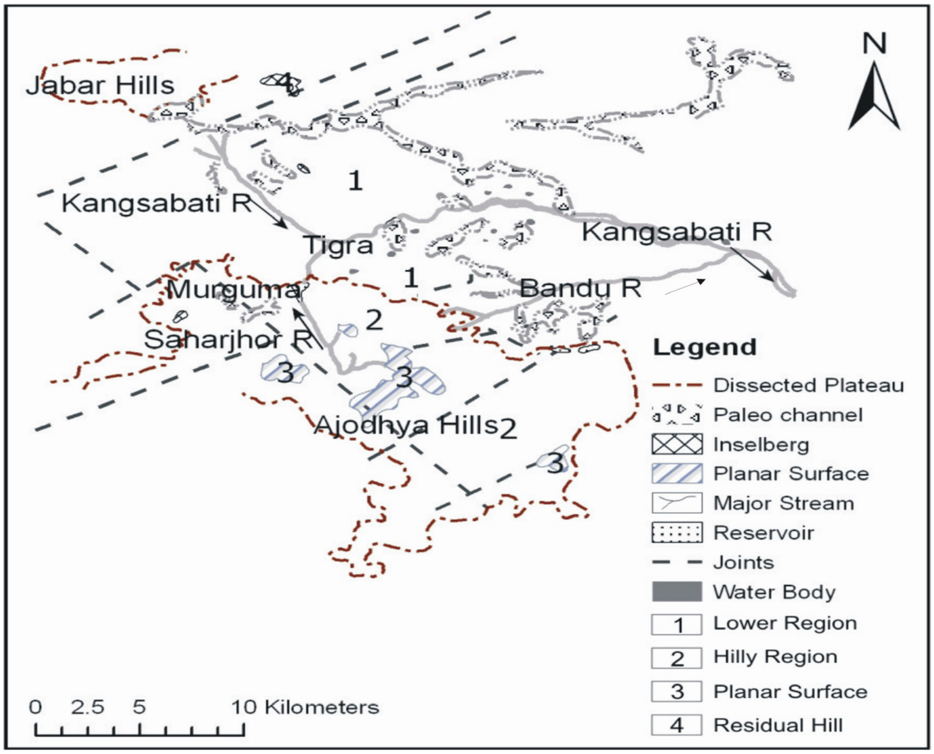

Figure 2

A generalized geomorphological and drainage map of the area

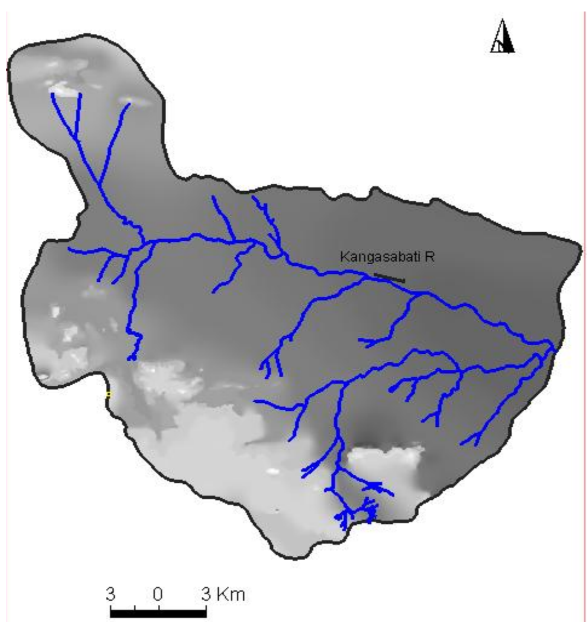

Figure 3.
DTM of the study area. Lighter tone depicts
higher elevation where as darker tone depicts lower elevation.

Observation and result:

In the study area numerous basins have been developed mostly within Ajodhya plateau and in the adjoining area have been developed. Third order and higher order basins nave been studied in context of their large dimensions which facilitate recognition of lithology, structure and nature of sediments in such basins. Here Kasai river covers a vast area in the northern part of the. study area. Three prominent trends of basins configuration have been recognized viz. EW, NW-SE, NE-SW. Since the area comprises mainly erosional landscape the development of basins is mainly due to slope retreat phenomena jointly with the action of structural and lithological control. The structural control is again governed by numerous master joints in the plateau (fig. 1). The major basin which comprises a part of the upper catchment area of Kangsabati river basin in the western part of Purulia district, eastern India is a drought prone area.

Here the proterozoic crystallines comprising mostly granitic rocks constitute the plateau and hills to the south and partly to the west of the study area. The southern range of the study area is Ajodhya plateau. Rocks of the area are traversed by several sets of joints among which two sets of intersecting master joints trending NNW-SSE and ENE-WSW and with steep dips (70°-85°) are quite noteworthy. Occurrences of some planar surfaces also been noted. In many places planar surface acts as water divide within the basins. Drainage network in the area is mostly rectangular being guided by master and associated minor joints except in the planar surfaces. Transverse profiles of major stream basins like Sahajor R, Bandu R(Fig.1), are narrow near their sources while those widen considerably near their mouth (fig.2). Longitudinal profiles of majority of the stream basins exhibit gentle gradient except Sahajori while some break in slope in its upper reach has been noticed. The average slope of the basins is 80m/Km.The lithological characteristic of the basins are granite gneiss with intercalated bands and lenses of calsilicate, amphibolites, mica hornblende schist. Some basins consist of a thin weathe profile while others with relatively fresh rock surface. Usually the shapes of such basins is almost triangular (leaf shaped). Major stream channels are mostly covered with gravelly sand and silt as well as by nearly rectangular boulders at places. Gorges, rapids, falls, incised meanders, nick points etc. sculptured mostly in the resistant rock masses within the plateau indicate rejuvenation of the area which was probably triggered as a result of the Himalayan Orogeny during late Tertiary period and this rejuvenation is still continuing (Singh, 1969) Here basin development has taken the advantages of the fracture and joint system which are opened up due to up warping of the plateau and its adjoining areas of neotectonic movement.

In such basins and interbasinal areas groundwater recharge potential areas are restricted to planar surface, valleys, fractured rocks as described below :

Planar surface : Planar surfaces are occupied mostly by regolith (5 - 10m. thick) which are veneered by a thin mantle (1-1.5m) of alluvial sediments in many places. Thickness of the saturated zone varies from 3-Mm. Aquifers are unconfined. Dugwells within such aquifers show poor yield (Dutta and Banerjee, 1979)

Valleys: Valley fill deposits (very light tones) are scanty in most of the major stream courses. Some isolated terraces of narrow width and thickness have been noted in the upper reach of Sahajhor as well as the lower reach of an easterly flowing streams. Several small coalescing fans on the opposite side of the terrace can be considered as a zone of groundwater recharge. Groundwater yield from the terraces and channel fill deposits appear to be low to moderate because of their limited spatial extent.

Fractured zones: Fractures in this zone comprise mainly the joint systems. Two intersecting master joint systems with steep dips have been conspicuously developed in the western part of the zone (fig. 1). Such joints have given rise to some major and minor stream courses. Most of the minor streams are ephemeral while major streams like Sahajor R, Bandu R, etc(fig.2). maintain a thin base flow during the summer months. From large extents of the major joints together with the associated minor joints, it is suggested that such fracture systems should be properly investigated to examine recharge potential.

The development of major basin has taken place in the synformal trough within two antiformal structures viz. Jabar in the north and Ajodhya hill in the south of the study area. The nature of sediments within this basin is rather sub-mature and is supposed to be the product of an older-fluvial cycle which can be traced in satellite imagery study (IRS IB) and field checks. The thickness of the sediments is of few metres and is underlain mainly by granitic rock. The average slope of the basin is 7-10 m/km. Field evidence shows that the Kangsabati river is an under fit river, amplitude of its meander is insignificant in comparison with the total width of the basin. Moreover, identification of older fluvial cycle with the help of remote sensing data and field checks suggests that the Kangsabati river (Kangsabati R or KASAI N) basin is probably a part of an older fluvial system. Occurrence of man made ponds probably have taken the advantage of the course of older fluvial system (fig.1).

Proper evaluation of groundwater recharge potential in a hard rock area needs an integrated approach involving geological, geophysical and hydrological studies.

Remote sensing technique can provide useful information on , the groundwater condition of such a terrain, e.g. for identifying various surficial features like water bodies, landforms, drainage density of stream network, localised gain or loss of stream flow. Combination of remote sensing and GIS technique has been proved to be very useful tool in groundwater studies (Krishnamurthy et.al 1996)

The north of zone of the figure 1, is a tract of sloping land comprising largely of coalescing alluvial fans (bajadas) formed by the major streams from the plateau region. Several distributary channels (mostly dry) could be discerned from the satellite imageries. The fan sediments comprise mostly silty sand in the lower reach while in the upper reach it is gravelly sand and silt. Drainage texture is coarse while its pattern is radial. Gradient of the fans range from 7 m/km in the west to 5.2 m/km in the east. The tract is extensively cultivated for paddy and sugarcane specially along its lower reach. Groundwater potentiality of the area is comparatively high as is evident from the presence of a good number of perennial tanks (in the lower reach), along the traces of paleochannel (fig.2) as increased flow along the major stream courses. A few strips of the badland topography along some minor tributaries in the zone have also been recognized in the field, this has been conspicuously developed near the source of Kumari river - eastern fringe of Ajodhya plateau(not present in the figures), here the alluvial fan has been incised at least upto a depth of about 20 metres, these along with the relatively steep scarp face of the plateau with some waterfalls indicate relative uplift of the area.

The pediplain zone lies to the north and is characterised by moderate undulatory topography but with an overall moderate gradient (average 5.25 m/km) towards south. Kangsabati R or KASAI N flows near the northern extremity of this zone. The area is mostly mantled by a weathered residuum (2-10 m thick) the country rock are mostly gneissose and schistose. Some abandoned major stream courses which are not cultivated for growing crops can be discerned from photo-interpretation. Such courses exhibit a NNW-SSE trend while trend of the tributaries varies from WNW to ESE. Drainage pattern is broadly rectangular with minor dendritic pattern developed at places where the tributaries have originated from comparatively thick weathered residuum in schistose rocks. Drainage texture is coarse. During the investigation drainage map has been draped on this composite map to show the spatial association of rechargeable areas with relevant sub-basins. Sub-basins with high drainage densities are favorable for surface run-off and do not encourage much natural recharge. However, intersections (fig.4) of sub-basins with higher drainage density values with weathered residuum/palaeochannels/buried pediments may encourage aquifer recharge when those intersections are bounded by water impounding structures(Das,2007).

Conclusion

The study area is an erosional landscape. The development of basins is controlled by structure and lithology apart from exogenous and endogenous erosional agencies. Basin development has taken the advantages of the fracture and joint system which are opened up due to upwarping of the plateau and its adjoining areas by neotectonic movement. Formation of comparatively large intermontane basin within a synformal trough viz. Kasai (Kangsabati R) basin reveals older fluvial system and the development of vast planation surface.

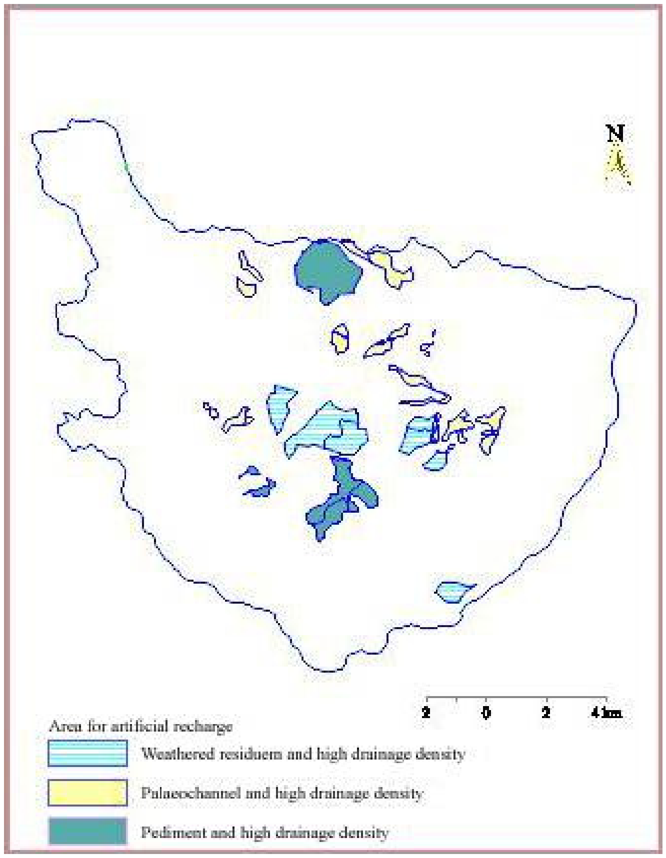

Figure 4 After Das, 2007
Potential recharge zones within the river basin.

Major fracture system mainly the intersecting master joints as described earlier are conspicuously developed in the plateau region, their traces can also be discerned in the pediplain region. Ajodhya plateau has been dissected into several large blocks by joints.Stepped topography, incised meanders, waterfalls etc. are the evidence, of rejuvenation. Major Hydrogeomorphic zones and their groundwater recharge potentialities as discerned mainly from remote sensing study need elaborate ground verification. Groundwater recharge potentiality for a major part of the area appears to be highly potential due to the presence of fractured rocks and drainage with high to moderate drainage density values.

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OPTIMISATION OF WATER RELEASE FROM A DAM TO RECHARGE A DOWNSTREAM UNCONFINED ALLUVIAL AQUIFER

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The Oanob Alluvial Aquifer (OA) is located at Rehoboth south of Windhoek in Namibia. The aquifer supplied the town till 1990. A dam constructed on the Oanob River, upstream of the aquifer, currently supplies the town and the aquifer is unused. Historically, seasonal flow in the Oanob River recharged the aquifer. Now recharge occurs when water from the dam is released or when water spillage occurs due to exceptional rain. The OA is narrow and elongated, made up of fluvial sediments laid down in an incised valley cut into impervious basement rocks. The aquifer is largely unconfined and elevated bedrock separates the aquifer into two compartments.

In order to reduce evaporation losses from the dam and utilise the additional storage available in the aquifer, and limit the negative affect of the dam on downstream vegetation, artificial recharge using the dam water is being considered. Recent groundwater level data unaffected by major recharge event was used to calibrate a steady state model to help understand the functioning of the aquifer. Evapotranspiration was identified as the dominant discharge process. Underground dams at the downstream end of the compartments were planned to reduce throughflow loss. The effects of such dams are also being assessed from the results of the groundwater model.

Keywords: managed aquifer recharge, evapotranspiration, Oanob aquifer, numerical modelling

INTRODUCTION

The Oanob alluvial aquifer (OA) is located in Rehoboth Town approximately 90kms south of the capital city of Windhoek, Namibia. The aquifer was used for water supply to Rehoboth till 1990. The Oanob Dam completed in 1990, upstream of the aquifer, has since made the aquifer redundant. The aquifer supplied Rehoboth Town from the 1950's to 1990. In 1988 the 17.28Mm^3 was supplied to Rehoboth. Currently, small-scale farmers use the aquifer for stock watering.

In a water scarce semi-arid region with high evaporation losses (2300mm/yr) the aquifer presents a unique opportunity to store water through managed aquifer recharge (MAR). This has been realised by the Namibia Department of Water and Forestry (DWAF) since a long time and had commissioned studies on the aquifer (Sweco, 2002 and DWAF, 2009). The total active storage capacity of the aquifer is estimated as 27Mm^3 (DWAF, 1989a). However high through-flow and evapotranspiration losses have been reported in past work and the viability of MAR will be determined by the volume of water that can be maintained through recharge of the aquifer with lower losses as compared to evaporation from the Oanob Dam.

The existing hydrogeological data on the aquifer (DWAF 1989a, DWAF 1989b, Sweco 2002, DWAF 2009) is used to reassess the losses from the aquifer with the aim of testing the viability of the proposed MAR. A preliminary steady state groundwater flow model was constructed using the MODFLOW-2000 code to improve the conceptual understanding of the aquifer and the findings are discussed here.

Oanob Aquifer

The Oanob Aquifer (OA) consists of riverbed sediments laid down in a narrow incised valley cut into the basement meta-sediments (Figure 1). The south-west flowing river carries sediments during ephemeral flood events from the elevated plateau area to the west. The sediment load is drops abruptly on reaching level ground immediately east of a regional north-south fault to the west of Rehoboth. The resulting sediments are poorly sorted and immature with interlayered sand, gravel, boulders and finer (clay and silt) sediments. A detailed description of the aquifer is given in DWAF (1989a).

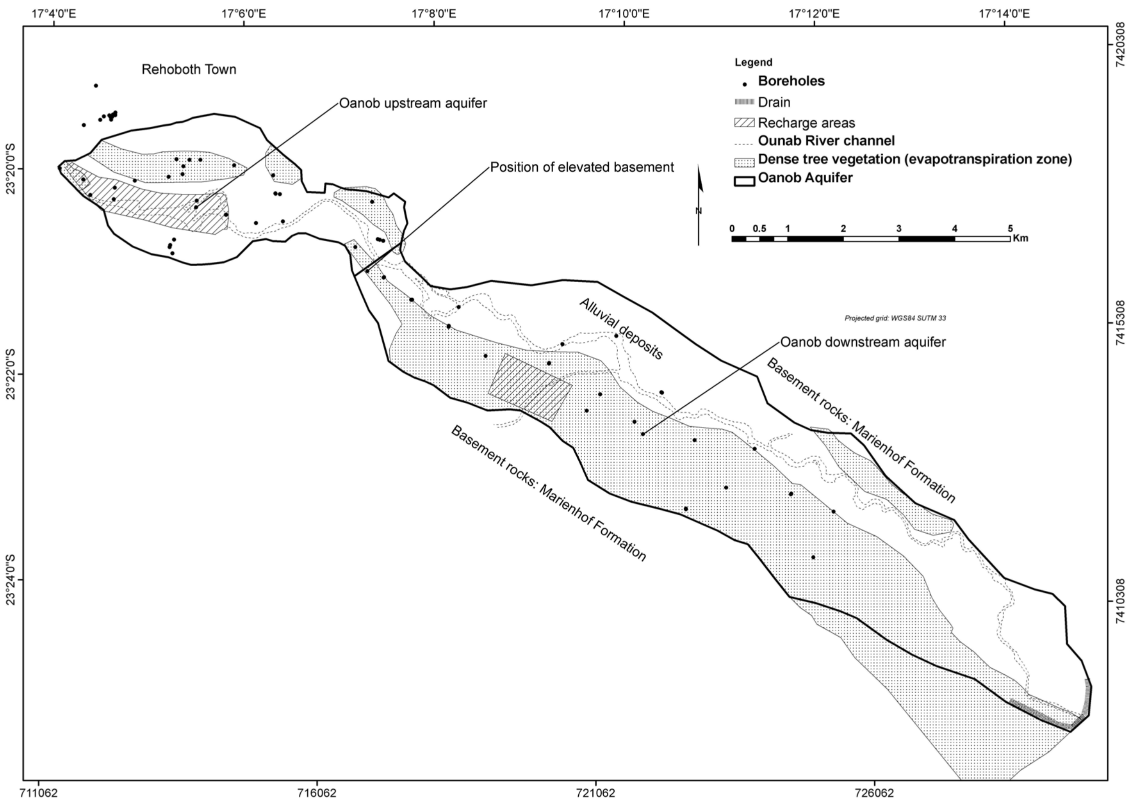

Figure 1: Oanob Aquifer and elements used in the flow model

The aquifer is elongated - 21.5km long and in average 2.5km wide. It is up to 38m thick but is thinner towards the south-eastern end. It is bounded to the north and south and underlain by meta-sedimentary basement rocks of the Marienhof Formation (Rehoboth Sequence). The bedrock has very low permeability and is considered impervious in the current study. The south-eastern boundary is not well defined though bedrock outcrop (granite) is seen at the end. For the purpose of the current study the aquifer boundary taken from DWAF (1989a) with minor modifications (Figure 1) is adopted.

Elevated bedrock at 5.8kms from the north-western end of the aquifer separates the OA into two compartments - upstream and downstream (DWAF, 1989a). Drilling information shows that the bedrock is at 11 metres below ground level (mbgl) at the divide located at the point indicated in Figure 1. Later stage erosion and superimposed alluvial fan deposits may have given rise to different sedimentary sequence and aquifer properties from approximately halfway towards the downstream direction in the downstream compartment. The properties of this part of the aquifer are not well known as no borehole information exists (Figure 1).

In the past the aquifer received recharge from seasonal flow on the ephemeral Oanob River. Recharge from rainfall is negligible. It was estimated that up to 33% of surface flow recharges the groundwater (DWAF, 1989a). Large evapotranspiration and throughflow losses were also recognised. Since the completion of the Oanob Dam in 1990, flow in the Oanob River has practically stopped. The only recharge events have resulted from release of water from the dam in 1992 and water spillage during years of exceptional rain (1996-1997, 1999-2000 &

2005-2006). It was agreed that 'surplus' water in the Oanob Dam would be released by the managing authority (Namwater) on a regular basis to sustain the vegetation downstream (DWAF, 1989b). This was discontinued after the 1992 release as it was observed that the released water is quickly lost downstream and does not benefit the vegetation as envisaged.

Hydraulic conductivity estimated from test pumping in the upstream compartment gave a value of 7m/day while in the downstream compartment values varied from 0.4 to 4.8m/day. Higher conductivities were noted in the section between the upstream and downstream compartments (22m/day). Mean specific yield estimate from three tests is 0.043.

Current Hydrogeological Conditions

During 2009, after a period of five years of no flow in the river, water levels were measured in the now unused supply boreholes and 13 monitoring boreholes maintained by Department of Water Affairs and Forestry (DWAF). It was estimated that the aquifer is 26% full (DWAF, 2009).

In the upstream compartment, shallow water levels are recorded closer to the northern extremity of the Aquifer, due to surface flow and probably underground leakage from the Oanob Dam. Water levels are shallower along the active river channel (from 3 to 9mbgl) while levels are deeper away from the river channel (more than 12mbgl). The flow in the river is restricted to the active channels and direct infiltration is therefore expected to be similarly constrained.

At the approximate location of the bedrock barrier separating the aquifers water level was measured at 9.7mbgl indicating that the throughflow to the downstream compartment is limited.

Water levels in the downstream compartment are deeper, ranging from 10.5 to 17mbgl. Data is sparse towards the southeastern part of the aquifer and are completely absent at the extremity. The contoured water table elevation show raised water levels at the confluence of a north flowing tributary of the Oanob River called Swartmodder that join the downstream section indicating inflow through the tributary alluvium.

Groundwater discharge can be via throughflow, evapotranspiration and abstraction. With the termination of the large-scale abstraction for Rehoboth water supply in 1990, loss by abstraction is limited to minor pumping by small-scale stock farmers with a total abstraction of 35 to 40m^3/day. Although some estimate of throughflow is available at the downstream end (DWAF, 1989a), there is no actual assessment of evapotranspiration loss. The relative importance of the processes is therefore unknown. It was observed during the current study that the thickness of the aquifer decrease rapidly from 16kms downstream and the effectiveness of the throughflow alone in discharging groundwater is doubtful (Figure 2).

An Aster satellite image dataset was used to calculate Normalized Difference Vegetation Index (NDVI) and highlight green vegetation. The image was acquired in 26 December 2008; two months into the rainy season (63.6mm rain in November and December 2008). The NDVI image was seen to highlight mainly tree vegetation when checked in the ground. The dominant species was acacia erioloba (camel thorn), known for its deep rooting and capacity to tap groundwater from depths up to 60m (Lubczynski, 2000). The acacia covered areas were mapped out as representing zones of higher evapotranspiration (Figure 1).

MANAGED AQUIFER RECHARGE AND NUMERICAL MODELLING

The understanding of the processes leading to losses from the OA is important in evaluating the potential of the aquifer for MAR. A steady state finite difference model was created using MODFLOW-2000 (Harbaugh et al., 2000). The following assumptions were made in creating the model:

1. The hydrogeological system is in equilibrium and all the elements used in the model are independent of time.

2. Recharge into the aquifer occurs through leakage from the Oanob Dam at the upstream end of the aquifer and at the confluence with a north flowing tributary (Swartmodder) that join the downstream section of the Oanob River.
3. Outflow occurs through evapotranspiration in areas of dense acacia vegetation and evapotranspiration from the grass covered and sandy areas are negligible.
4. Discharge by throughflow occurs at the downstream extremity of the aquifer.

The aquifer is unconfined and three zones of hydraulic conductivity can be identified: the upstream compartment, the narrow zone connecting the upstream and downstream compartments and the upper part of the downstream compartment, and the lower part of the downstream compartment.

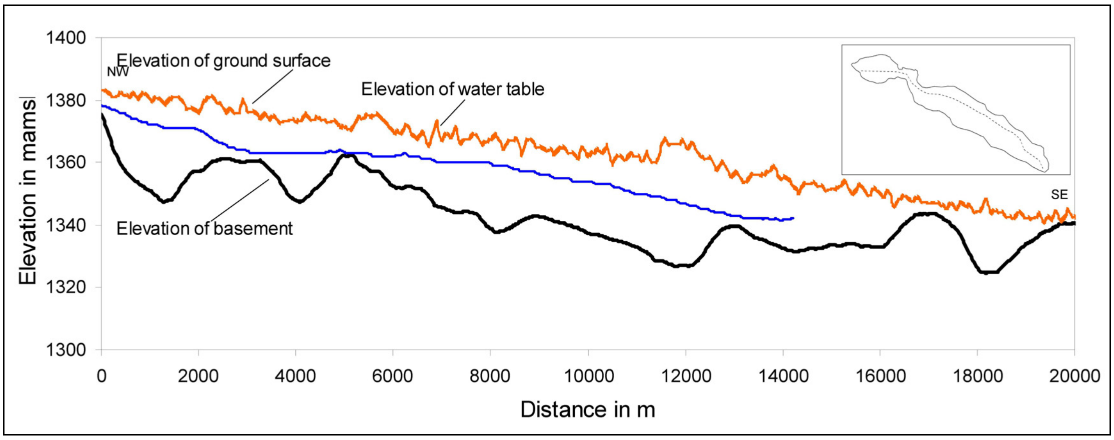

Figure 2: NW-SE cross-section along the length of the Oanob Aquifer

The model is set up as a single layer model using SRTM elevation data as the top surface. The bottom surface was estimated though interpolation using borehole information and vertical electrical sounding data from DWAF (1989a) and DWAF archives and calculated as depth from the top surface to the basement. The model was discretised into 200 by 200m cells. The modelled layer was assumed to be confined for the simulations as the saturated thickness of the final calibrated model is expected to reproduce the saturated thickness of the aquifer (Hill and Tiedeman, 2007). Also, the added effort required in parameter estimation under unconfined conditions with a varying saturated thickness is not likely to produce significantly different results when compared to simulation using a confined layer (Hill and Tiedeman, 2007).

The Modflow-2000 model was run using the observation, sensitivity and parameter estimation process mode taking the 2009 measured water level elevation as observations (Hill et al., 2000). The estimated parameters were hydraulic conductivity of the three zones (LPF_PAR1, LPF_PAR2, LPF_PAR3) and evapotranspiration flux in the upstream and downstream compartments (EVT_PAR1 and EVT_PAR2 respectively). The depth of extinction of evapotranspiration was assumed to be 10m below top of the model layer. A drain was introduced in one of the model tries at the south-eastern extremity of the model and the drain elevation was placed at the base of the aquifer. Drain hydraulic conductance was also estimated (DRN_PAR1). The simulations were run using the preconditioned-conjugate gradient solver (PCG2).

RESULTS AND DISCUSSION

Three conceptual models were tested on the basic numerical model setup described above:

Model 1: Model with recharge along the entire Oanob River (active channel) and discharge of groundwater through the drain at the end of the aquifer and evapotranspiration (Figure 1).

Model 2: Recharge limited to the upstream end of the upstream component (Figure 1) and at the confluence with a north flowing tributary (Figure 1). Initial inflow value used was estimated by using the Darcy equation and known hydraulic conductivity and cross-sectional data from the section of the aquifer. Discharge was modelled through evapotranspiration, drain and wells.

Model 3:Same as Model 2 above but discharge was allowed through evapotranspiration and wells.

Model 1 failed to reproduce the measured heads (while the model water balance was acceptable) and was discarded. The simulated heads approached the measured heads in reruns of Model 2 with increasing proportion of loss through evapotranspiration flux. The drain parameter was found to be insensitive in the model while the evapotranspiration parameters had the highest sensitivities. Model 3 converged with measured heads being adequately reproduced (Figure 3). The weighted residuals (Figure 4) show near random distribution with moderately higher variance in the simulated heads in the downstream aquifer. The deviation of the modelled heads from the observed in the lower downstream compartment is due to lack of sufficient head data on this section (Figure 5). The hydraulic conductivities estimated were considerably lower than the estimates from test pumping. This is probably because the tested boreholes were located in sediments with higher hydraulic conductivity and sediments with dominantly argillaceous material that are not included in the set of tested boreholes used to estimate hydraulic properties. The actual recharge therefore is less than initially calculated using conductivity values estimated by test pumping.

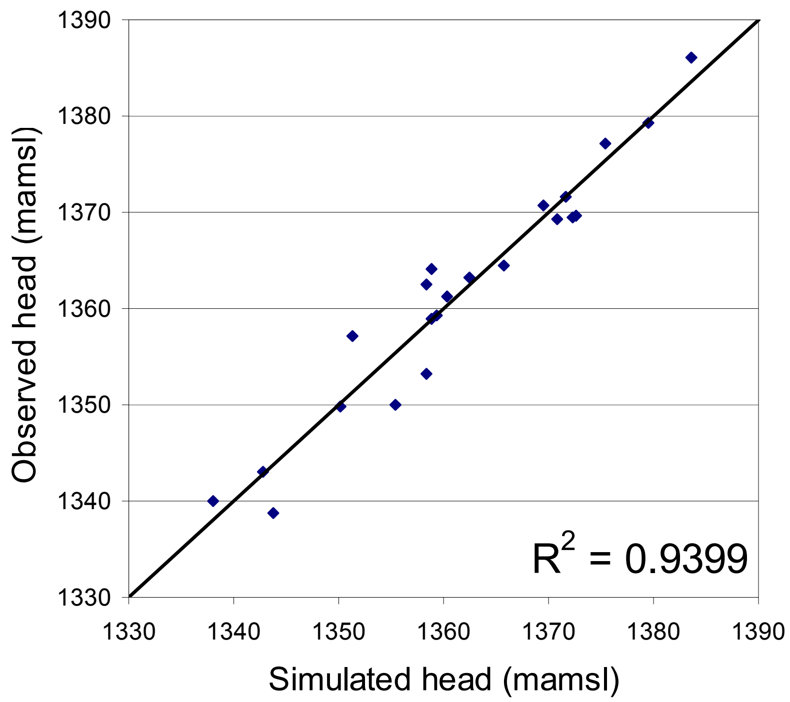

Figure 3: Observed and simulated heads, steady state Model 3

However the overall modelled heads were effectively reproduced and evapotranspiration accounted for 85% of the discharge for the system while the remaining 15% is discharged through wells. The modelling suggests that evapotranspiration is the dominant discharge mechanism of the aquifer. Throughflow may be only significant when water levels are higher after major flood events. This has implications on the viability of MAR as the aquifer evapotranspiration continues to be effective when the aquifer is partially saturated.

One of the ideas brought forward to limit throughflow loss during artificial recharge in the upstream aquifer, was an underground ‘barrier’ on the basement high between the upstream and downstream sections. This could minimise throughflow but evapotranspiration losses may still be significant. A similar basement high was identified at the end of the downstream compartment for a barrier at the downstream end. The aquifer is considerably thinner at the south-eastern end and barriers may not be effective in restricting losses. The deeper sections of the aquifer could be suitable for artificial recharge. For effective planning and design of the MAR in the upstream and downstream compartments the aquifer geometry, particularly the downstream boundary, has to be determined in more detail. Geophysics surveys and drilling specially at the south-eastern end of the aquifer will be necessary to estimate aquifer thickness (depth to bedrock), sample aquifer material and, determine effective hydraulic properties.

To simulate artificial recharge through a transient numerical flow model and to evaluate loss through evapotranspiration and throughflow would require verification and additional data on the aquifer properties (hydraulic conductivity and storage co-efficient) through test pumping in the upstream and down stream compartments of the aquifer.

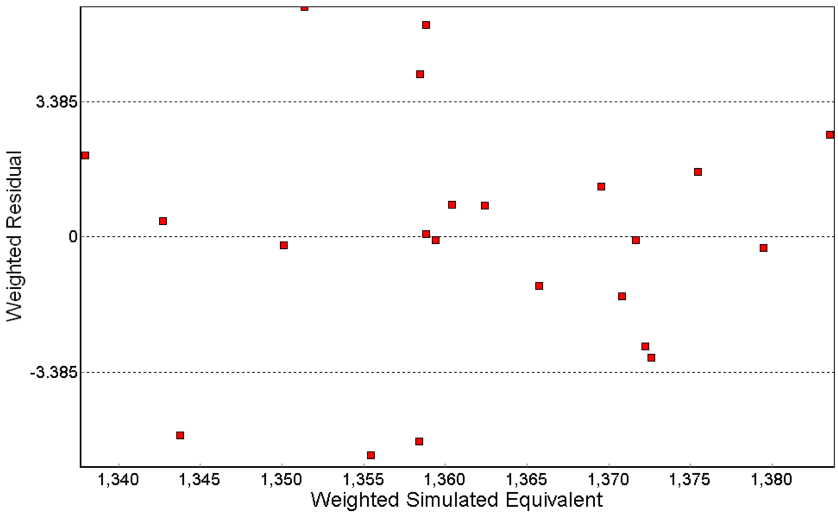

Figure 4: Weighted residual vs weighted simulated equivalent plot of Model 3

CONCLUSIONS

The steady state model suggests that the dominant process of groundwater discharge from the Oanob aquifer is evapotranspiration. Evapotranspiration and abstraction from wells can account for the total water loss from the system while the reproducing the observed heads. Throughflow may not be as effective in the discharge from the aquifer as previously thought particularly where the unconfined aquifer is thin and shallow.

The relative importance of the aquifer discharge processes when actively recharged by flow in the river or during artificial recharge cannot be assessed with the current knowledge of aquifer properties and the downstream boundary. To design managed aquifer recharge the effect of evapotranspiration will have to be further assessed in detail through transient simulation. For this, further investigations of aquifer geometry and hydraulic properties will be necessary.

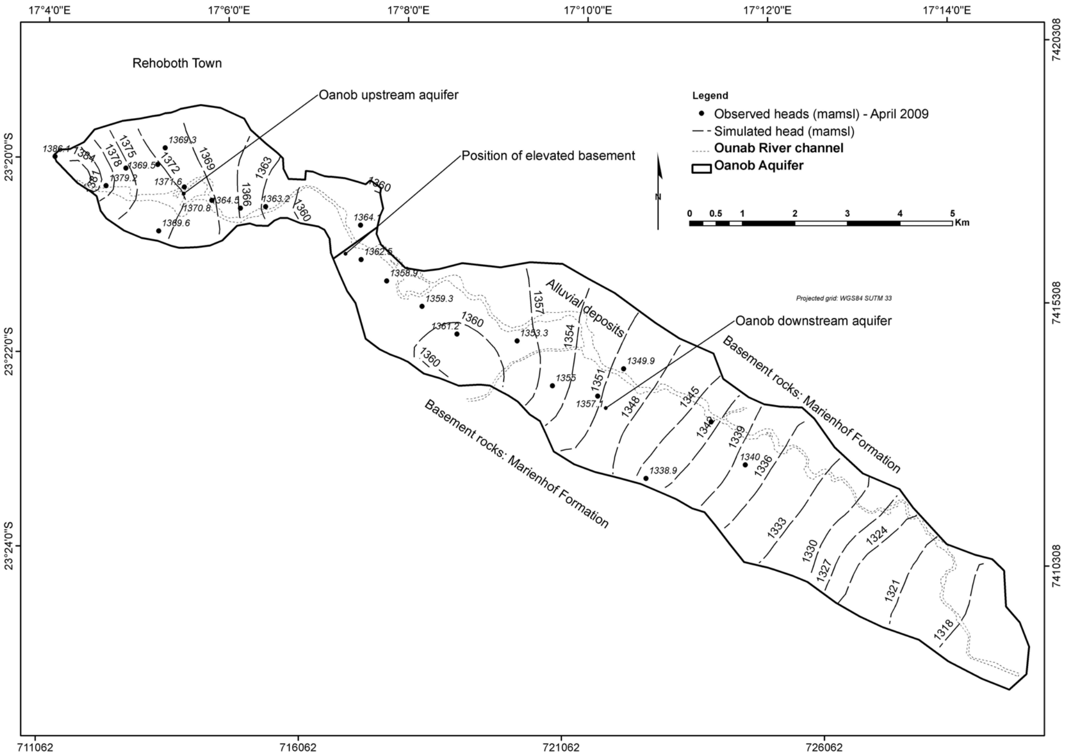

Figure 5: Model simulated contours and observed heads, Model 3

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Modeling of wellbore effects in Managed Aquifer Recharge monitoring

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Abstract
An adequate monitoring system is critical in any Managed Aquifer Recharge (MAR) field implementation and can target multiple objectives including operation performance, storage integrity and environmental impact assessment. In the case of fresh water injected into a saline aquifer, monitoring the freshwater plume location/distribution often includes repeated measurements or vertical profiles of the electrical conductivity (EC) in the available observation or pumping wells. The representativeness of such measurements in long-screened wells can be affected by wellbore effects like mixing and cross-flow inside the wells. In particular, cross-flows can be observed because of the vertical heterogeneity and vertical hydraulic gradients which exist in the geological formation. Such effects can not only significantly bias the monitoring results but can also increase the mixing effect and deteriorate the ASR recovery efficiency. A reservoir engineering simulator was used to model both the aquifer and well behavior in the framework of an ASR project in an arbitrary setting of a confined aquifer. Based on this illustration example, the potential impacts of cross-flow effects are detailed in terms of ASR monitoring and operations.

Keywords
Managed aquifer recharge, cross-flow effects, multilevel monitoring, modeling

INTRODUCTION

Monitoring in MAR/ASR operations

An adequate monitoring system is critical in any Managed Aquifer Recharge (MAR) field implementation and can target different objectives including measurement and evaluation of the following-up of:

1) Operation Performance, whereby the inputs/outputs of the system are controlled in order to optimize the injection/recovery scheme (often targeting the highest achievable flow rates under current design).
2) Storage Integrity in the case of Aquifer Storage and Recovery (ASR), whereby the pressure response and freshwater plume distribution are monitored to detect any issue of underground leakage (e.g. through the caprock), of unplanned displacement of the stored water due to hydrogeologic heterogeneity, or of storage contamination.
3) Environmental impact, to document that the hydrodynamic and geochemical effects of the MAR system continuously meet the local regulations.

The monitoring network is designed according to the monitoring objectives and ideally allows the acquisition of sufficient and unbiased monitoring data. The subsurface part of this monitoring system (as opposed to the surface facilities) generally consists of several observation wells spatially distributed around the recharge area (i.e. the ASR well(s) in the case of ASR projects) and where different types of repeated measurements can be made. Monitoring data typically includes water level measurements (or pressure measurements), electrical conductivity (EC), chemical and microbiological analysis from collected water samples (Pyne, 1995). For example, the areal distribution of the monitoring wells may be optimized in order to capture the breakthrough curve in concentrations and allow some mapping of the freshwater 'bubble'. The well completion often consists of long screened wells across the thickness of the ASR storage zone (Reese, 2002). The suitability of such long-screened wells is investigated in this paper.

Monitoring wells in multi-aquifer settings

The suitability of long-screen monitoring wells has been discussed in the literature for decades. The debate relates to the representativeness of measurements from long-screen wells because of the known occurrence of vertical flow of water in the wells. The theoretical basis of these cross-flow effects are, for instance, described in Papadopoulos (1966) in terms of hydraulics. Bennett (1982) also suggests that “ignoring the effects of multilayer wells in simulation will inevitably produce erroneous results”. The multi-aquifer well effects were thus described and equations were proposed to add those effects to numerical simulations. Development of dedicated simulation packages then followed in the next decades e.g: the MAW1 package for MODFLOW (McDonald, 1986) or the FWL4 for MODFLOW SURFACT (HydroGeoLogic, 2001). In terms of solute transport, Gibs (1993) concludes that contaminant concentrations derived from water samples from long-screened wells are not representative because of mixing effects. Reilly (1989) further shows through numerical modeling that vertical flow of contaminant in long-screen wells can cause contamination of previously uncontaminated aquifer zones. He also concludes that cross flow effects can be significant even when the vertical gradient is imperceptible. This is supported by Church (1996) from field investigation results in a relatively homogenous sand and gravel formation.

The analysis of monitoring data acquired from long-screen wells in the framework of MAR/ASR projects is not well detailed in the literature. This paper presents preliminary numerical modeling results of an ASR scenario including wellbore cross-flow effects in order to illustrate potential bias in monitoring well data, well-known in the world of contaminant hydrology.

MODELING RESULTS

Model description

The modeled scenario is that of a single ASR well completed in a 100 m-deep, 30 m-thick, two-layer confined aquifer. The model parameters are described on Figure 1.

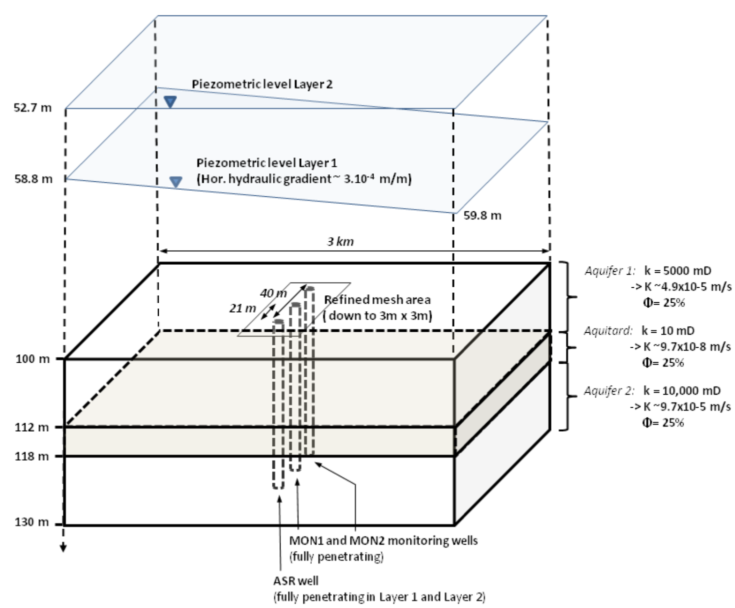


Figure 1: Model parameters

The model boundary conditions are chosen as follows:

- Layer 1: Constant head boundaries at the western and eastern edges imposing a regional horizontal gradient of 1 m over the 3 km extent of the model. No-flow boundaries at the top, at the northern or southern edges.
- Aquitard layer: No-flow boundaries on the outside edges.
- Layer 2: Constant head boundaries at the western and eastern edges. A zero horizontal hydraulic gradient is considered. No-flow boundaries at the bottom, at the northern or southern edges. The piezometric level in Layer 2 is purposely set significantly greater than that of Layer 1.

The ASR schedule arbitrarily consists of 4 ASR cycles with the following durations: injection during 110 days, storage during 60 days and recovery for a duration ranging from 29 days to 69 days based on the duration to reach a salinity threshold of 500 mg/l. Also, in order to observe any equilibration of the water levels in the wells, a period of 15 days without pumping is considered here prior to the first injection. The native groundwater salinity is 3,000 mg/l.

Flow and transport simulations were carried out using ECPLISE®, a reservoir engineering code that was originally developed for oil and gas applications but that has recently been adapted to also handle hydrogeological problems. Eclipse has been used to simulate ASR behavior as reported by Herrmann (2004, 2005). This simulation code can handle the density-dependent flow conditions that may be encountered in ASR projects as well as some advanced well modeling including cross-flow effects.

The numerical model consists of 20 layers each of 1.5 m thickness. Horizontally, a variable-size mesh discretization scheme has been adopted with cells refined to 3 m x 3 m in the area of interest around the wells (Figure 1).

Flow simulation results

As highlighted in Papadopoulos (1966), the composite hydraulic head measured in long screen monitoring wells is a weighted average affected by the difference in transmissivity and head in the two layers that are penetrated. Figure 2 show the hydraulic head response in well MON1 during the non-pumping period prior to the first injection. The time reference t_0 (t=0) corresponds to the start of the transient simulation.

Prior to t_0, the head in Layer 1 is steady and equal to -52.7 m. The head in Layer 2 is steady and equal to -59.3 m. At t_0 the well is drilled and the pressure in the two layers equilibrates at the wellbore. The resulting head is the composite head measured in the well. The fully-penetrating well does not allow for the detection of the effects of the vertical hydraulic heterogeneity. During injection, the simulated head in the well is slightly larger than that of neighboring blocks (i.e. in the formation). This can be attributed to block pressure averaging.

The pressure in the block has also been simulated without the cross-connection for comparison purpose. In practice, such results could be obtained from a wireline logging job or from sealed in-situ pressure measurements. When the cross-flows are removed from the simulations, the head in Layer 1 and Layer 2 remain separated. At t=15 days, the ASR injection causes a pressure increase in both layers. The different pressure increase in each layers is not be observed in the single fully-penetrating well.

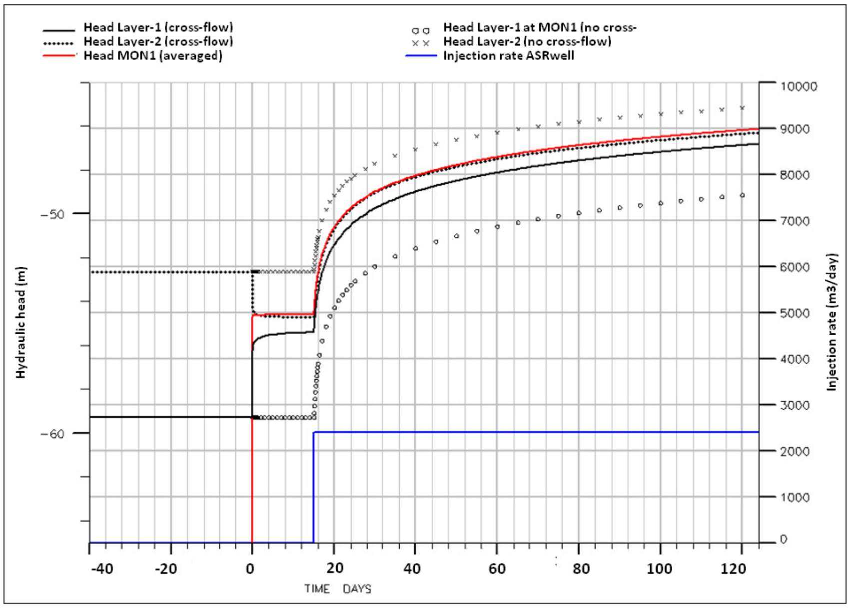


Figure 2: Simulated head in the MON1 monitoring well during the first injection phase.

The flow rate across each connected block is plotted in Figure 3. As expected, the well connections to Layer 2 have a positive flow rate illustrating that water is flowing from the formation to the well. The flow in Layer 1 is perfectly symmetrical i.e. water is flowing from the well to the formation. In this particular case, fully-penetrating wells create a hydraulic connection between different hydraulic regions of the system. This may have critical impact in terms of solute transport and ASR integrity and certainly handicaps one's ability to observe how the system is responding.

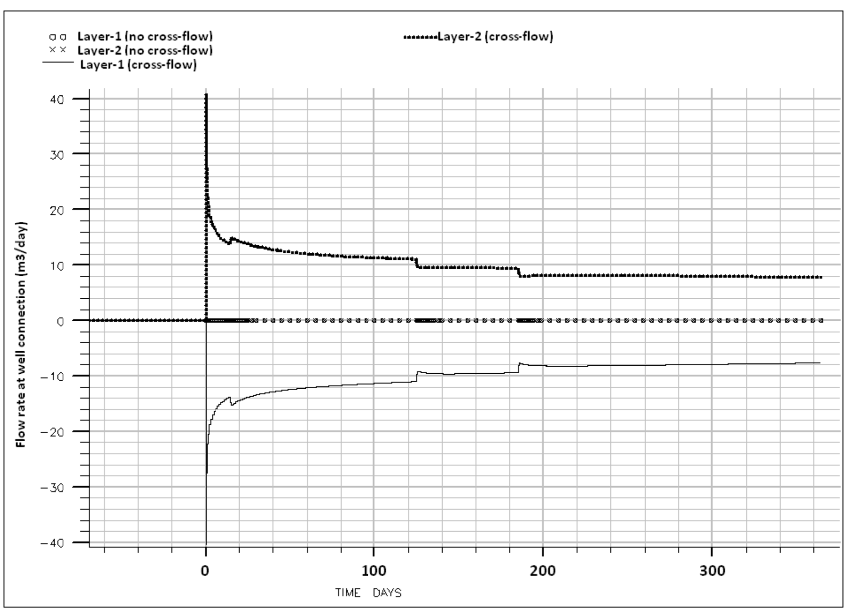
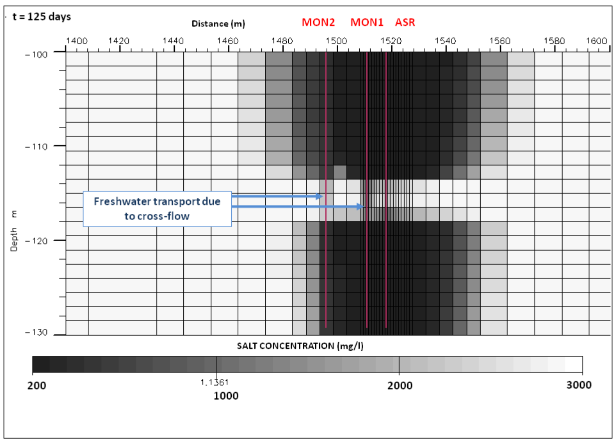


Figure 3: In-flow and out-flow at the well connections

Solute transport modeling results

The solute transport simulations results also show significant impact of the cross-flow effects on the monitoring results as illustrated on Figure 4. The salt concentration results are displayed in a North-South cross-section through the numerical grid. Figure 4a represents the simulated salt distribution at the end of the first injection phase. The freshwater injected in Layer 2 has flowed towards Layer 1 in the fully-penetrating monitoring wells MON1 and MON2. Similarly, the salt distribution at the end of the first recovery phase shows that the more brackish water from Layer 2 has moved upwards to Layer-1 (Figure 4b).

a)



b)

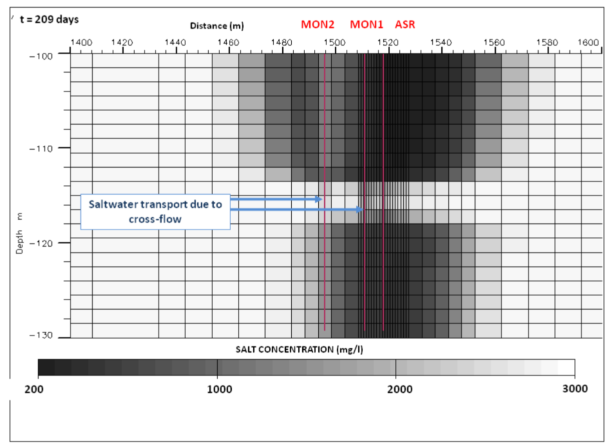


Figure 4: Simulated salt concentration at the end of the first injection phase (a) and first recovery phase (b)

The simulation results in the case of the fully-penetrating wells were compared to the results without the cross-connection effects. The salt concentrations are significantly different around the monitoring wells as illustrated in Figure 5 for well MON1. The salinity results in the middle blocks of Layer-1 and Layer-2 are plotted for the first ASR cycle. In this particular case, the fully-penetrating well causes significant bias in the monitoring data. For instance, one can observe, during the injection phase, that the difference in salinity between the upper and lower layers is less when cross-flow is allowed in the monitoring well. This can be attributed to the fact that the solute is transported from Layer 1 to Layer 2 and mixes. During the storage and recovery phases, the base case (without cross-flow) shows lower salinity.

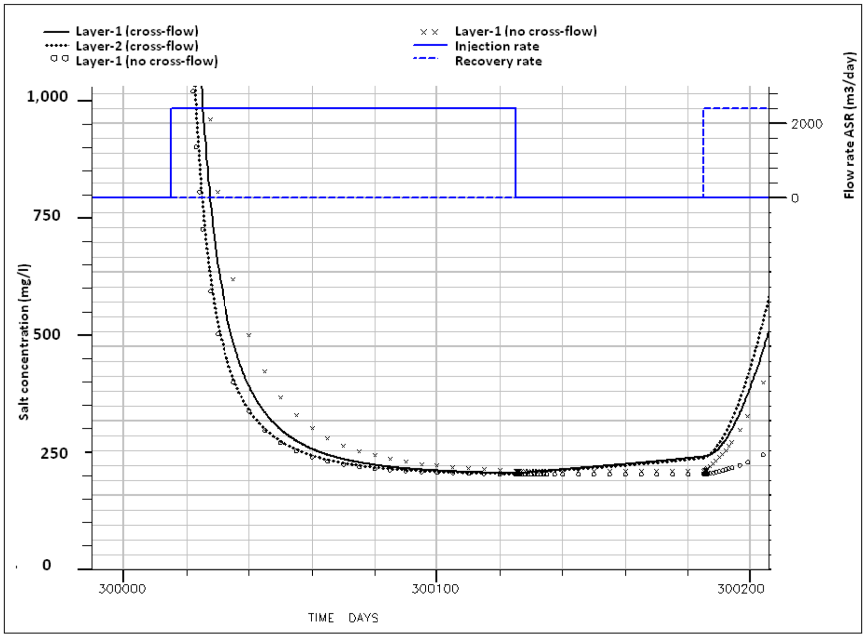


Figure 5: Simulated salt concentration in MON1 monitoring well

Besides the bias in observed concentration data, the produced water salinity in the ASR well is larger when cross-flow occurs, in the particular case considered here. This is illustrated in Figure 6, where the produced salt water is significantly fresher in the base case (no cross-connection).

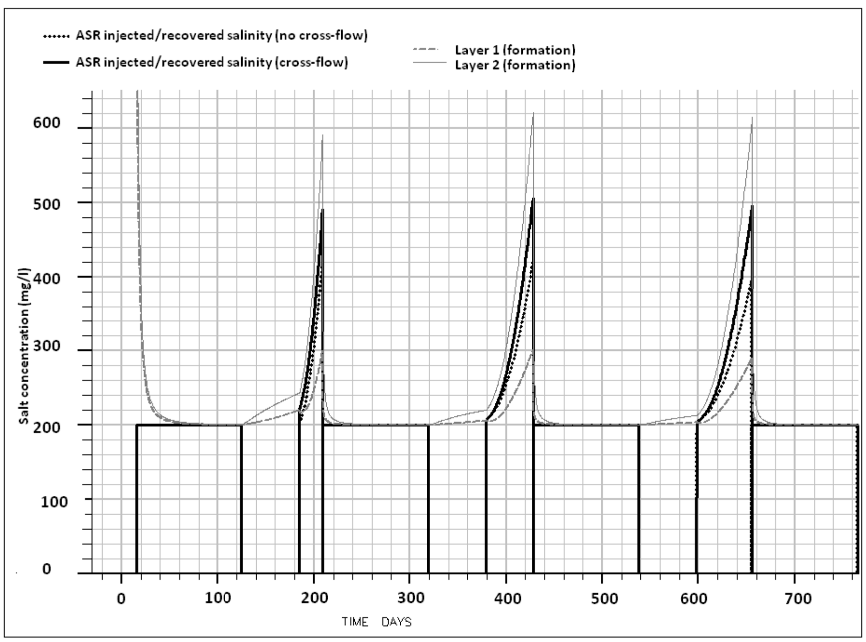


Figure 6: Simulated salt concentration in the ASR produced water during the first ASR cycle

CONCLUSION

For a particular case of multi- layer aquifer, the use of fully-penetrating monitoring wells may:

- Prevent a good characterization of the system as the multiple hydraulic regions may not be identified (as only one composite water level is collected);
- Bias the hydraulic head monitoring data because of the cross-flow effects;
- Bias the salinity data obtained in those wells through vertical profiling (such as EC logging) or sampling. The trends may be reversed during injection and recovery phases (under-estimating or over-estimating the concentration compared to its natural distribution);
- Create vertical pathways that can deteriorate the recovery efficiency of the ASR.

The chosen scenario might seem to be an extreme case with a large vertical hydraulic gradient of about 1 (m/m) between the two aquifers. Further work including sensitivity studies should be carried out to identify the limitations of such results. However, it should be noted that large vertical gradients can be found in natural settings as reported by Meyer (2007) and Hart (2007). Moreover, significant cross-flow effects have been observed and simulated in relatively homogeneous formations (Reilly, 1989). Therefore, the potential for such significant cross-flow effects must be taken into consideration in ASR operations monitoring unless the complete absence of vertical heterogeneity and vertical gradients can be demonstrated. The results presented here should then be seen as a particular case chosen to illustrate such effects.

It is shown that fully-penetrating observation wells generally do not provide an optimal way of monitoring ASR operations. The potential wellbore effects including cross-flows and mixing can bias the monitoring data and may even lead to deteriorating the ASR efficiency through cross-layer contamination and increased dispersion. The importance of prior characterization is critical in order to identify vertical heterogeneity in the formation in terms of lithology, permeability or even native water quality distribution. Adequate geophysical logging, hydraulic testing and hydrochemical analysis should thus be planned. During the ASR operations, monitoring through effective multi-level monitoring systems is recommended.

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The Turku Region Artificial Infiltration Project, Finland – Tools for Enhanced Aquifer Characterization

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Abstract

The city of Turku and its neighboring municipalities are located in Southwestern Finland. The area has a population of 300 000 inhabitants. The present water supply of the area relies on chemically treated river water. The surface water supply will be replaced with artificially infiltrated groundwater in year 2011. The project will be one of the largest managed aquifer recharge projects in Northern Europe.
The water to be infiltrated will be obtained from the River Kokemäenjoki, located 90 kilometers north from the Turku area. The artificial infiltration will take place in the Quaternary esker aquifer called the Virttaankangas aquifer.
Extensive geological, sedimentological, geochemical, and hydrogeological studies have been carried out in the Virttaankangas aquifer. However, the internal variety of the hydraulic conductivities in the coarsest part of the esker needed to be mapped in greater detail prior to the one year test pumping period with river water in 2010. The results will be used to update the existing groundwater flow model.
The studies conducted for detailed aquifer characterization included sedimentological modeling, GPR-soundings, drillings, and tracer tests. These studies were focused on the coarsest part of the aquifer, depicted as the *glaciofluvial coarse* unit in the 3D hydrogeological model. The structures of the 3D model units will be integrated with groundwater flow model. In short, the aim was to focus on the area where the infiltration and water intake takes place. The sedimentological and GPR studies provided detailed structures of the esker core, subaqueous fans, and morphologically undetectable kettle holes, whereas the tracer test verified the flow paths and residence times of the infiltrated groundwater within those structures. All these data will be used to explicitly describe the variation of hydraulic conductivities of the aquifer in groundwater flow model.

Keywords

Artificial recharge, fluorescent dyes, GPR-soundings, groundwater flow modeling, sedimentological modeling, tracer test.

INTRODUCTION

The Virttaankangas aquifer plays a key role in the Turku region artificial recharge project (Figure 1A, 1B). The aquifer will not be used merely to store the infiltrated river water, but also to enhance the quality of the water. The natural purification of the infiltrated water during the flow within the saturated zone of the aquifer is a crucial process for those artificial recharge plants in Finland that provide water for about 500 000 consumers at present. After its completion, the Virttaankangas managed aquifer recharge project will significantly increase the number of consumers using artificially recharged groundwater in Finland.

Previous studies conducted in the area have depicted the large scale hydrogeological structures and general flow conditions within the aquifer and its surroundings (Artimo et al. 2003a, 2003b). The Virttaankangas 3D hydrogeological model structures (Figure 1C) have already been integrated with the previous groundwater flow model versions. In addition, the extensive geodatabase (Artimo et al. 2008) has been used to manage different types of hydrogeological data.

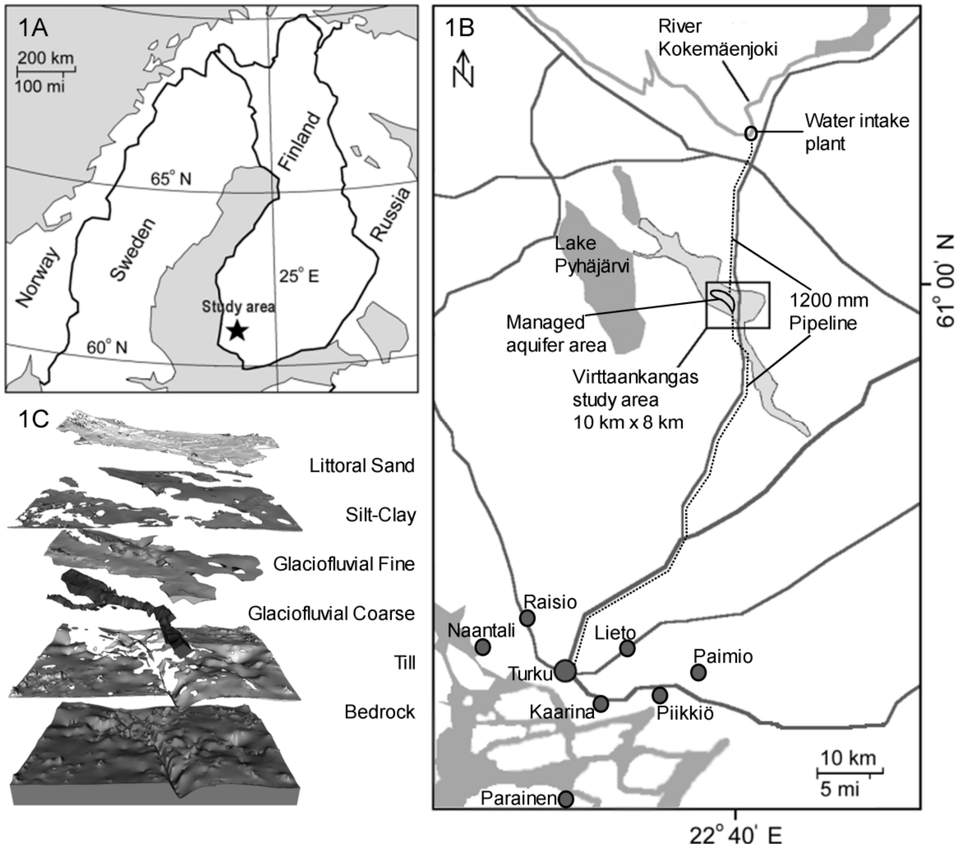

Figure 1. The location of the study area (A), the general map of the artificial infiltration project (B) and the hydrogeological units of the Virttaankangas 3D model (C).

This study was launched to obtain more detailed understanding of the internal varieties of hydraulic conductivities within the coarsest parts of the esker aquifer that will be used for infiltration and water intake. The task required means to identify the architectural units affecting the groundwater flow, and to define the precise flow paths and residence times of the infiltrated water.

The work was carried out by ground-penetrating radar soundings that were used to create a detailed sedimentological model of the area, and by conducting a tracer test to provide information of the groundwater flow field.

Albeit groundwater tracer applications are widely used in various geological settings, (Käss, 1998; Divine and McDonnell 2005), the applications of fluorescent tracers in glacigenic esker environment and in the scale of this study are uncommon in literature.

METHODS

Ground penetrating radar soundings

The ground-penetrating radar (GPR) survey was designed to record the large-scale sedimentary structures of the aquifer covering an area ca. 3 km long and 0.5 km wide. The GSSI SIR-3000 radar system with 100 MHz antenna, GPS-positioning, and topographic correction from national airborne laser scanning data (vertical accuracy 0.3 m) was used for the survey. Based on pre-survey field tests the measurement time was set to 400 ns and dielectric constant to value of 6 representing dry sandy and gravely glaciofluvial material. The penetration depth varied between 10-25 m depending on the local sediment characteristics, and corresponds to thickness of sediments above the groundwater level. The survey includes 126 profiles (mostly 100-200 m long) with total length of ca. 20 km. The survey line network (Figure 2A) was guided by location of forest roads, accessibility of the terrain, distribution of drill holes and earlier research results on the depositional pattern and related hydrogeological units of the esker (Artimo et al. 2003; Mäkinen 2003). In order to estimate paleocurrent directions and lateral changes in depositional units, the GPR lines were planned to cross so that line spacing was kept in 50-100 m, if possible. The GPR survey was conducted by motor sledge at the beginning of March when the area was still snow-covered. Reference data for the survey is provided by one large excavation within one of the esker fans (cf. Mäkinen 2003) and 45 drill hole logs that are connected by the GPR lines. Processing of the GPR data (by Geo Doctor software) was kept minimal and included background removal and application of gain. All the profiles (cut to penetration depth of 22 m) were combined in freely rotatable XYZ space of SurPac 6.1 software to facilitate 3D interpretation of the profiles and their integration with other data sources (Figure 2B).

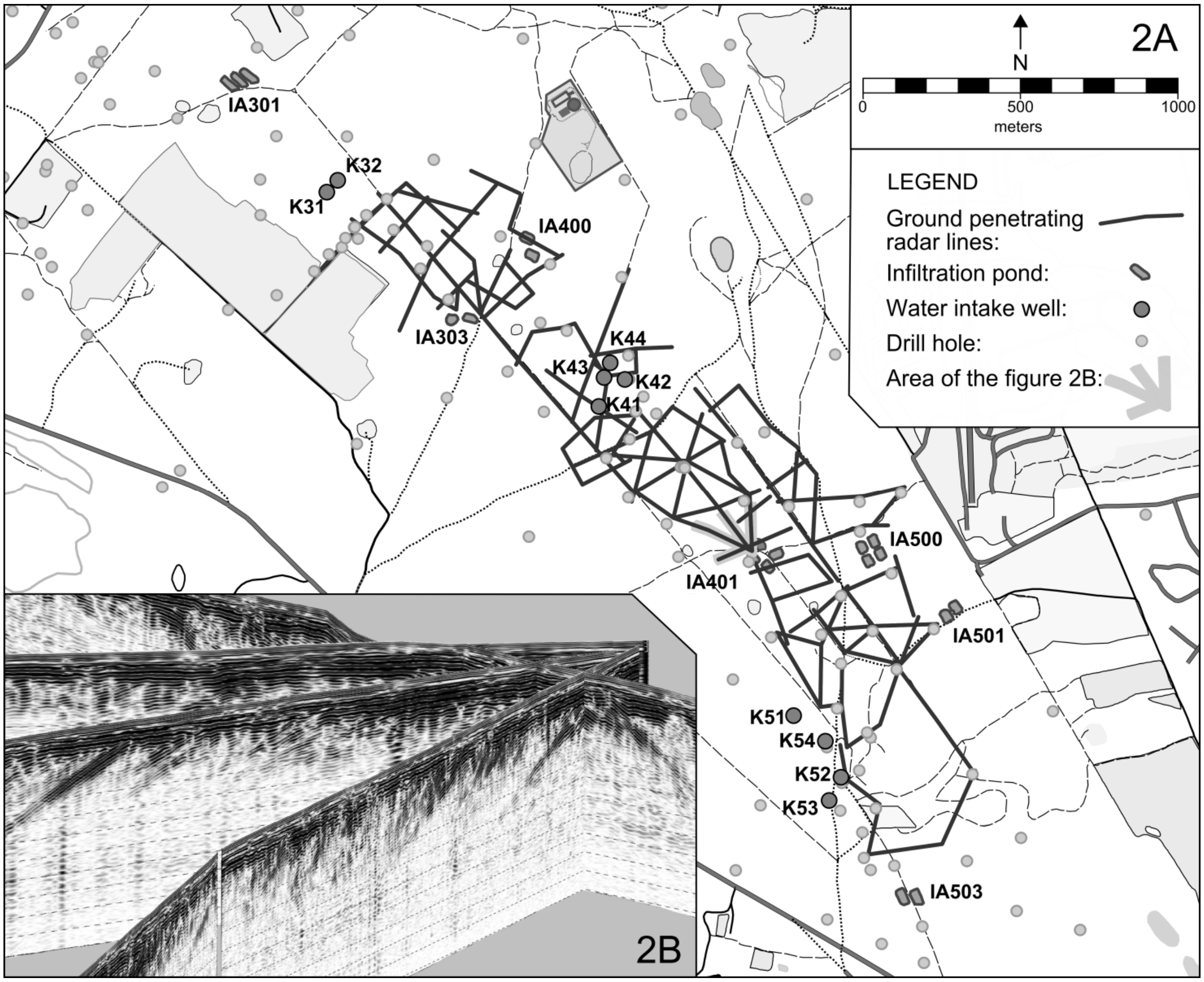

Figure 2. Locations of the GPR sounding lines, drill holes, water production wells and infiltration ponds (A). 3D representation of GPR lines showing the 10-12 m thick inclined fan foreset beds (B).

Sedimentological modeling

The GPR profiles in SurPac XYZ space were interpreted with data from drill hole logs, earlier GPR surveys, and interpolated bedrock topography in order to describe the major architectural and depositional units as well as related time-transgressive depositional stages of the esker. Firstly, the path of the uniform, gravely esker core with arched architecture was mapped to delineate the major conduit of glaciofluvial sediment delivery. Secondly, the location and direction of repeated, overlapping esker fans were determined to detect lateral changes from the coarse- to fine-grained deposits and the orientation of the associated fan foreset beds. Thirdly, the location of large-scale (up to 100-200 m wide and hundreds of meters long) deformation structures related to morphologically undetectable kettle holes (MUKH-structures) were mapped. These ice-contact structures are delineating the esker core and are crucial for artificial infiltration as well as for groundwater flow patterns within the coarse-grained unit. The original sediments on top of the esker (10-20 m) were intensively eroded by shore processes following the glacioisostatic land-uplift. Moreover, the eastern side of the esker is for a large part masked by extensive spit-platform deposits with large-scale foresets (Mäkinen and Räsänen 2003).

Due to limited penetration depth, only the upper part of the esker deposits was recorded by the GPR survey. However, the application the local depositional model of the esker including the influence of shore erosion (Artimo et al. 2003; Mäkinen, 2003), stratigraphical data from the drill hole logs in relation to the architectural units, and relatively detailed information on the bedrock topography made it possible to predict the sedimentary characteristics in the lower part of the esker deposits (below groundwater table).

Tracer tests

The tracer tests were conducted during a 6 month pumping and infiltration period. A total amount of 18 000 m^3/d of groundwater was pumped from 3 wells (K41, K42, K43) and was infiltrated to 4 different infiltration areas (IA303, IA400, IA401, IA500). Water production well K51 is located in the southern part of the study area and it withdraws 4950 m^3/d for consumption for almost 25 000 inhabitants. The locations of the production wells and infiltration areas are displayed in Figure 2.

Selection criteria for the tracers

Fluorescent dye tracers, uranine and eosin, were chosen for this test due to their many favorable qualities, such as good mobility, durability and low detection limits since the natural background levels of both tracers in the aquifer were zero. Because of the constant water production from the well K51, the most important property for the tracers was low toxicity for people and animals (Safety data sheets for uranine and eosin, Merck 2005, 2007) and that the tracers were geno- and ecotoxically safe (Behrens et al. 2001).

The tracer test area was extensive, including 59 monitoring locations, which were used to collect the tracer profiles to create a 3D picture of the tracer plumes. Therefore the possibility to measure *in situ* in the 52 mm diameter observation wells was significant. The field fluorometer measurements were immediately available to guide the subsequent measurements. This dynamic measuring schedule helped to save time, and more importantly, focus on the areas where the tracer breakthrough was next expected. In addition, field measurements were more inexpensive method to obtain data than analyzing the pumped samples in laboratory.

Testing and calibration of the field fluorometer

Prior to the actual tracer test, the fluorometer was tested in laboratory at the Department of Chemistry, University of Turku. The fluorometer was calibrated for measuring of both tracers using the groundwater from the Virttaankangas aquifer. Detection limits for both uranine and eosin were defined separately and with different uranine/eosin mixture ratios. Also the effect of turbidity at various dye concentrations was examined (Salo and Ala-Kleme, unpublished reports). Mixing of both dyes in the same sample caused some difficulty to the determination of the dye concentrations. Turbidity was found to affect eosin visibility more than the uranine visibility.

Tracer infiltration, measuring procedure and schedule

A total amount of 2 kg of uranine and 6 kg of eosin were introduced into the aquifer on 10th of June 2009. Uranine was infiltrated to area IA401, whereas eosin was infiltrated to infiltration areas IA303 and IA400. The used amounts were calculated on the basis of the regulations provided by the environmental authorities. The uranine concentration in the production well K51 was not allowed to exceed 2 μg/l, whereas the limit for eosin was 6 μg/l. These limitations were based on the theoretical visibility limits of both tracers.

Tracer concentration profiles were measured *in situ* using GGUN-FL24 –fluorometer (Schnegg and Bossy 2001; Schnegg, 2002). The field measurements were verified from pumped water samples that were analyzed with a spectrophotometer in laboratory. In addition, 55 groundwater monitoring wells and 4 water production wells were used for observations during the 12 month period, resulting more than 900 tracer concentration profiles. Three-dimensional (3D) coordinates for each measurement were recorded. Therefore the tracer profiles can easily be used together with the sedimentological and hydrogeological data of the Virttaankangas area. Even though the pumping and infiltration was ceased 6 months after the tracer injection, the tracer concentration measurements were continued to observe the aquifer recovery to normal flow conditions.

RESULTS AND DISCUSSION

Application of the sedimentological model

The number, arrangement, and spacing of the GPR-sounding lines proved to be adequate to record the depositional stages of the esker. The following 6 large-scale architectural and depositional units were determined for the sedimentological model: (1) esker core (subglacial tunnel deposits), (2) ice-marginal, proximal gravely apices of repeated subaqueous fans on top of the esker core, (3) subaqueous medial to distal esker fan lobes and related large-scale foreset beds, (4) morphologically undetectable kettle holes (MUKH-structures) with large-scale deformation, (5) fine-grained clayey bed supporting the perched groundwater table in beach deposits on the eastern side of the esker, and (6) uniform erosional unconformity and overlying beach deposits. The scale and size of the observed structures coincided quite well with the groundwater flow model cell size, which is 50 meters times 50 meters in horizontal direction, and 2 meters in vertical direction.

The sedimentological studies were found to be very useful in interpretation of the tracer test results. Especially this was the case close to the infiltration area IA401, from which the infiltrated water flowed almost entirely to north against the natural groundwater flow direction. The reason for this can be seen in Figure 3 that shows finer grained deposits related to the morphologically undetectable kettle hole structures acting as a flow barrier for the infiltrated water. In addition, the groundwater flow paths and flow velocities observed during the tracer test helped to extend the sedimentological interpretations below the depth range of the GPR-soundings.

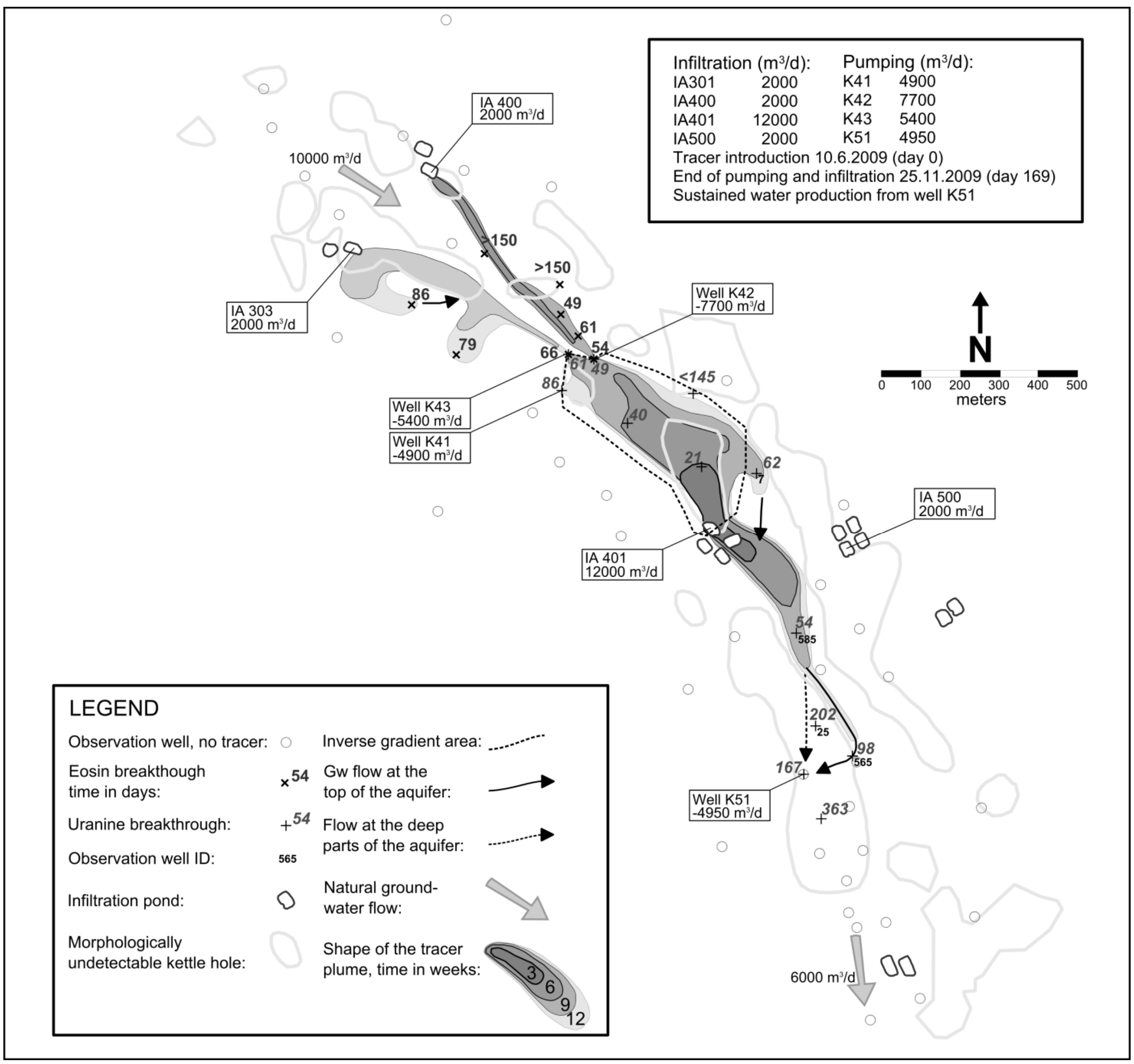

Figure 3. Tracer test arrangement, the measured breakthrough times in days, and the resulting interpretations of the tracer plume distributions at different times.

Development of reverse gradients and tracer breakthrough times at production wells

The development of reverse gradients is crucial for the operation of the artificial infiltration system. Prior to this study, the reverse gradients were only studied using groundwater flow model simulations. The results of this study confirmed the formation of the reverse gradients (Figure 3). In addition, the tracer breakthrough times at the wells K41, K42 and K43 were close to the flow model results. The chosen infiltration and pumping volumes resulted that the breakthrough times of eosin (natural gradient) and uranine (reversed gradient) at the wells were almost similar.

Eosin flow paths

Eosin plume from the infiltration area IA303 divided into two parts. Most of the infiltrated water flowed southeast following the coarsest esker core deposits. The tracer was discovered first from the well K43 66 days after the infiltration. Some minor volumes of dyed water also deviated south from the esker core following esker fan structures. The other plume from IA303 flowed south towards less conductive area resulting in a slower flow velocity.

Eosin from the infiltration area IA400 flowed mainly towards southeast. The flow focused into a very narrow zone following the coarsest parts of the esker core. The tracer was first found near the well area 49 days after the infiltration.

Uranine flow paths

Uranine was introduced into a single infiltration pond in the infiltration area IA401. The plume deviated into two parts. The main plume flowed north and followed the kettle hole structures. The first discoveries were made 21 and 40 days after the infiltration. The well breakthrough occurred at well K42 in day 49, and K43 in day 61, respectively. Further, the well K41 is located next to another kettle hole, which restrained the flow time to 86 days. Some parts of the northern uranine plume spread to the areas of normal gradient. The first discovery of that plume was made from observation well 7 in day 62. Interestingly, the first plume passed the observation well 7 completely during the six months of the pumping. When the pumping ceased and the gradients returned to their normal state, the second plume of uranine, originating from area near the well K43, was discovered in the observation well 7 in day 215 and it had passed the observation well completely in the day 321.

The rest of the uranine flowed southeast from IA401 following the coarsest parts of the esker. This plume was first discovered in the topmost 4 meters of the groundwater body in the observation well 585 54 days after the dye introduction. The other part of the plume was discovered in the same observation well 27 days later at the groundwater depth of 50 meters. During the next 10 days, the separation between the shallow and the deep plume disappeared, and the uranine covered the whole 50 meter depth of the aquifer. The shallow plume has been interpreted to have followed the esker core, and it was discovered again at the observation well 565 in day 98. However, most of the uranine followed the deeper flow route directly south of observation well 585. The deep plume was discovered in water production well K51 in day 167. The observation well 25 is located completely inside a kettle hole, and therefore the uranine was discovered 35 days after the breakthrough at the well K51. The uranine concentration is still slowly increasing in the well K51, being 0.2 µg/L at the moment.

Outcome of the study

Tracer test proved that both of the chosen fluorescent dyes were suitable for use in unconsolidated glacigenic deposits. Uranine, however, is not necessarily suitable in all glacigenic environments because it needs a slightly alkaline environment (pH>7) to avoid decrease in fluorescent intensity. An important benefit of the field fluorometer was that the tracer concentration profiles were immediately available, which enabled the following of the developing tracer plumes. Moreover, the extensive tracer monitoring would have been impossible to execute by sample pumping and laboratory analyses – the use of field analyzer saved time, effort and money.

The cross-analysis of the tracer plume shapes, GPR studies and sedimentological interpretations revealed the esker structures at a scale which is significant to groundwater flow. These results are directly applicable to control the future water production, and have also been used to create a high-detail groundwater flow model of the area (Figure 4). The results of the groundwater flow modeling, however, are beyond the scope of this paper.

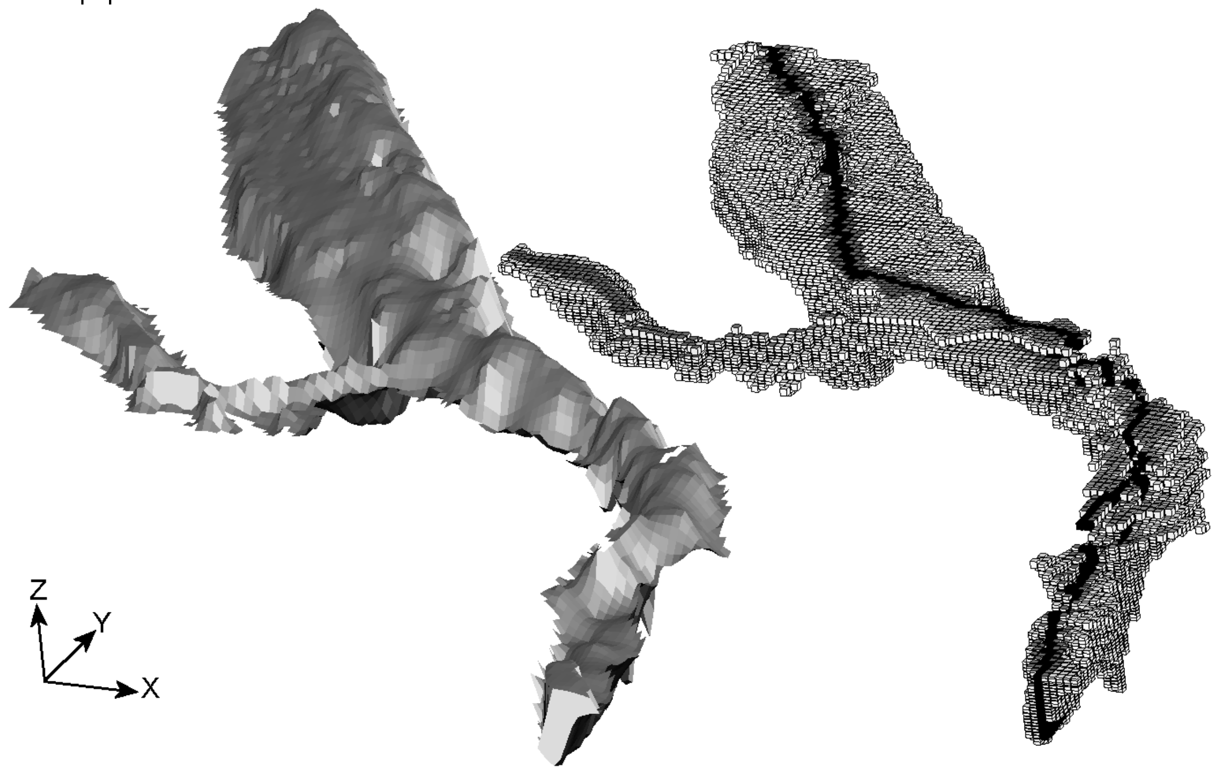

Figure 4. The results of this study have been used to add detail to the 3D groundwater flow model of the area. The glaciofluvial coarse unit of the 3D hydrogeological model (left) and the hydraulic conductivity distribution of the newly updated groundwater flow model (right).

Verification of the results and future monitoring

Twelve new drillings and groundwater monitoring well installations were conducted to verify the results of the sedimentological studies and tracer tests. In addition, these new monitoring wells will also act as additional monitoring locations for the artificially infiltrated river water. The tracer test showed that the flow paths of the infiltrated water can be quite narrow and restricted by the sedimentological structures. Therefore, the right location of the monitoring wells can have a crucial influence on the quality and representativeness of the obtained monitoring data.

CONCLUSIONS

The methods used in this paper proved to be highly useful in depicting the heterogeneities within an unconsolidated esker aquifer. The detailed sedimentological model used with the fluorescent tracers provided the tools to obtain a picture of groundwater flow in three-dimensions in a complex glaciofluvial environment.

The results of this study were used to update the 60-layer groundwater flow model of the Virttaankangas aquifer. The study area covered the coarsest part of the unconfined esker aquifer, in which the artificial infiltration also takes place. The interpreted and measured results described in this paper can be extended to cover the future changes in the aquifer during the full scale water production.

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NEAR WELL SEISMIC METHODS FOR AQUIFER RECHARGE PROJECTS; PERTH BASIN WESTERN AUSTRALIA

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Abstract

Acoustic methods have the potential to characterize the distribution of hydraulic and mechanical properties of rocks in the near well environment. Understanding these parameters can be crucial for design and operational management of any pumping or injecting well field. We investigate the application and potential of near well acoustic methods, such as Full Wave Form Sonic Wire-line Logging and Vertical Seismic Profiling for ASR projects. Acoustic methods provide information related to the propagation of stress and strain. However under certain circumstances acoustic methods can also be used to estimate hydraulic permeability. We consider data from multi-frequency full waveform sonic wire line logging and vertical seismic profiling (VSP) surveys completed at a trial aquifer recharge site in Perth Western Australia. We illustrate the seismic technologies deployed (e.g. new equipment) and how FWF sonic and VSP surveys can be incorporated in aquifer recharge projects at reasonable cost. The target injection interval for the trial ASR project site spans the Wanneroo member of the Leederville formation. It consists of several relatively thin highly permeability layers that are at most 10 m thick, separated by much lower permeability siltstone or claystone dominated layers. The aquifer zone is in the depth range from approximately 300 to 450 m below ground level. Of particular interest within the Wanneroo member are the very high permeability sandstone layers. We demonstrate how both high and low permeability layers are expressed in the multi-frequency full wave form sonic wire-line logging data. We demonstrate how small changes in velocity with frequency for a range of seismic wave propagation modes can be recovered from FWF sonic data and indicate how these measurements may be used to improve understanding of the hydraulic permeability distribution within sandstones.

Keywords (Full Wave Form Sonic Wire Line Logging, Stoneley Wave, Compressional Wave, Velcoity Dispersion, ASR, MAR)

INTRODUCTION

Seismic technologies for analysis of the near well environment are highly developed for a range of hydrocarbon exploration and production applications. Various complex arrangements of transmitters (capable of transmitting at a range of frequencies), and receivers allow an array of mechanical and hydraulic parameters to be estimated. Examples of the parameters recovered include: porosity, shear modulus, bulk modulus and yes even permeability. Although applied sparingly and viewed with some trepidations within hydrogeological circles, near well acoustic methods like FWF sonic wire-line logging and vertical seismic profiling are starting to gain traction as legitimate and useful tools in hydrogeology. In particular ASR and MAR require specific and high quality information concerning mechanical and hydraulic parameters in the near well environment. One impediment to progress in the application of near well acoustic methods is the sometimes confusing set of physical models and associated mathematical representations that are needed to explain the behavior of acoustic wave propagation (see, Barton 2006 and Pride 2005). Concerning this issue, Bourbie et el., (1987), writes that:

" In fact, the complexity of the porous medium (see chapter 1) may be such that it is totally unrealistic to try to construct a general model for porous media"

That is, a single universal model for acoustic wave propagation does not exist. Pride, 2004 makes and attempt to unify three models, while Gurevich (2007) was able to show how some models are only suitable over a restricted range of frequencies. At this point in history we need a range of models to explain acoustic wave propagation. These models depend on, wave propagation mode, propagation frequency, rock type and fluid type (see Biot 1956 or Mavko. et al 2009).

Despite the above, if the selected numerical model is suitably matched to physical state of the rock and fluid under investigation then there is no reason why important parameters like permeability cannot be derived, given that the required field measurements are made with sufficient accuracy. The evolution of multi-frequency, multi-separation, multi-orientation full wave-form sonic tools is bringing the technology closer to its potential. Of particular importance for hydrogeology are the newer slim-line FWF sonic tools that can transmit at dominate frequencies in the range 1 to 30 KHz and have up to 2 transmitter and 8 receivers (e.g. the Mount Sopris Instruments FWF sonic probe as used for this research).

The basic information to be derived from sonic logging is compressional or P wave velocity. However the modern trend is to attempt to obtain, attenuation and velocity for compressional, shear and tube (i.e. Stoneley) wave modes over a range propagation frequencies. Note that frequency dependence of amplitude attenuation (e.g. typically described by the reciprocal of the "Quality Factor", Q) can be related to velocity dispersion (Batzle 2005). Changes in compressional and Stoneley wave velocity with propagation frequency (i.e. velocity dispersion) is of particular interest as there may exist a link between dispersion and permeability (Pride 2004). Although there are a range of theories, the basic idea is that dispersion in sands/sandstones can be related to the relative movement of the rock framework (e.g. grains) and fluid (e.g. water) in a saturated or partially saturated rock (Berryman et al. 2003). That is, small changes in velocity with frequency can be related to fluid movement around grains and ultimately to permeability. This idea has relevance for both compressional and Stoneley modes of acoustic wave propagation.

The field site chosen for the FWF sonic wire line logging experiments is located along Gnangara Rd in Perth Western Australia at the M345 Trial ASR site. The research is being completed as part of a "Western Australian Water Foundation Grant" project designed to assess high volume aquifer storage and recharge (Li et al. 2006) in the confined high permeability Wanneroo Sandstone member of the Leederville Formation; Perth Basin; Western Australia. The Wanneroo Sandstone is confined above by the Pinjar member and below by the Marajiniup members of the Leederville formation. The project is a collaboration between, CSIRO land and water, Curtin University of Technology and the Water Corporation of Western Australia. The Grant is awarded by the Western Australian government and administered by the Department of Water, Western Australia.

Previous studies show that the propagation of Stoneley waves can be highly effected by the discontinuity in permeability. That is, they are effected by transition between high permeability sand and very low permeability shale layers (see, Zhao, Toksoz and Cheng 1994).

Of particular interest for this project is the highly heterogeneous nature of high and low permeability layering within the Injection interval. That is, the injection interval is approximately 100m thick and can be partitioned into several thin (typically less than 10m thick) high hydraulic conductivity (often greater 10 m/day) sandstones and extremely low hydraulic conductivity siltstone/shale layers. The transitions between high hydraulic conductivity and low hydraulic conductivity is often sharp. This characteristic of the Wanneroo member of the Leederville formation made it ideal for trialing near well acoustic methods for ASR.

NEAR WELL SEISMIC METHODS AT THE M345 TRIAL ASR SITE

For the drill hole M345-109 we consider the characteristics of full waveform sonic P wave velocity with frequency in the five key sand interval at the M345 site. Flow logs indicate that the hydraulic conductivity in these five sandstone dominated intervals may be up to 40 m/day. Given the high hydraulic conductivities it is likely that most of the water injected to or pumped from the M345-207 production well, 30m away from the monitoring well M345-209, will be constrained to flow dominantly in these layers. Figure 1 below shows the gamma and electrical resistivity logs for the injection interval at the M345 site. Cluster analysis (i.e. from cross plotting of the gamma and electrical resistivity) allow the five intervals to be correlated from the injection well (i.e. M345-207) to the monitoring well (i.e. M345-109).

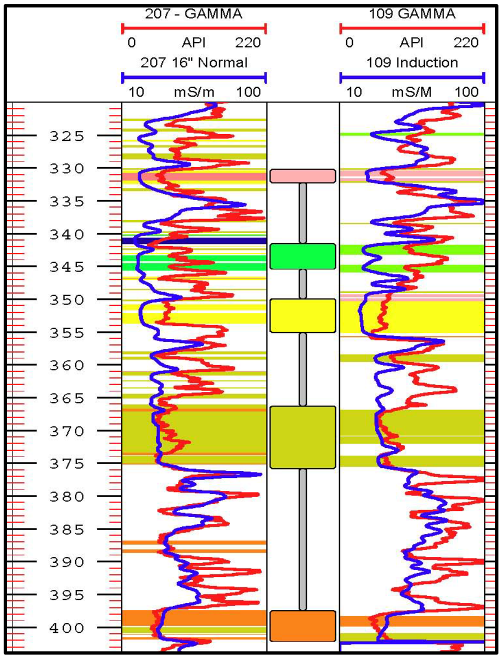

Figure 1, M345 sandstone intervals mapped out by cluster analysis of gamma and resistivity cross plots. The injection and monitoring well are separated by ~ 30m. Both induction (M345-109) and 16" normal resistivity logs are shown in mS/m (i.e. conductivity) and so are expected to track the gamma log. The five sandstone dominated interval are color coded from top to bottom, pink, green, yellow, yellow/green and orange.

Multi-frequency FWF sonic wire line logging was completed in the M345-109 drill hole after completion of drilling (i.e. before casing was installed). That is, at every 10 cm over the injection interval a full wave form sonic record is made. That is, a pulse with a specified dominate frequency is created at the Transmitter (Tx) and the full waveform is recorded at 4 receivers (Rx1, Rx2, Rx3 and Rx4). The receivers are spaced at 3, 4, 5, and 6 ft above the transmitter. The receiver separation divided by the difference in travel time for the first arriving pulse in any two receivers will provide an estimate of "group velocity" for the pulse traveling in the bore hole wall. However we can also recover small but measurable changes in "phase velocity" with frequency. It is these small changes in phase velocity with frequency that may provide information about formation permeability for sandstones.

We explain how these small change in "phase velocity" with frequency can be measured from full waveform sonic data.

The left hand image in Figure 2 shows a typical full wave form sonic record (i.e. one of many thousands). The right hand image shows the cross correlation of the first arriving pulse (see figure 3) for Rx 2 and Rx 3 for a selected dominant transmitter pulse frequency of 3KHz. It should be noted that although a dominant transmitted pusle frequency of 3KHz is selected, in reality the transmitter generated a pulse with measureable spectral energy in the range ~2 to ~25KHz. That is, selecting different dominate transmitted pulse frequencies simply shifts the distribution of spectral energy toward the selected frequency for the transmitted pulse. The peak (i.e. largest amplitude) of the cross correlation traces provides a very accurate measure of the first arriving pulses travel time between Rx2 and Rx3 for each frequency. There is a clear shift in the peak of the cross correlations traces with frequency. That is the travel time decreases and hence velocity increases with increasing frequency for the interval between Rx2 and Rx3. In short the image below indicates velocity dispersion of the order 5 -10 % at a transmitter depth of 353 m below ground level. Note that we should take care not give too much weight to values at the extreme higher or lower frequencies. That is, at the lower and high frequencies signal to noise ratio drops away. Also for the lower frequencies the receivers tend exist within the "near field" of the transmitter. Technicalities aside, we start to see convincing evidence for velocity dispersion. Notice that there are many methods to obtain phase velocity versus frequency from seismic data (see, McMechan et al. 1981, Park et. al. 2004 and Pun et al. 2010). These methods are not necessary suitable for FWF sonic data, which often has a limited number of offsets and is "contaminated" with a wide range of wave modes that trail the first arriving pulse (i.e. the wave modes can be difficult to separate). We have selected a method that uses narrow bandpass filters to separate out frequency content followed by cross correlation to accurately determine the travel time (i.e. velocity) between any two receivers.

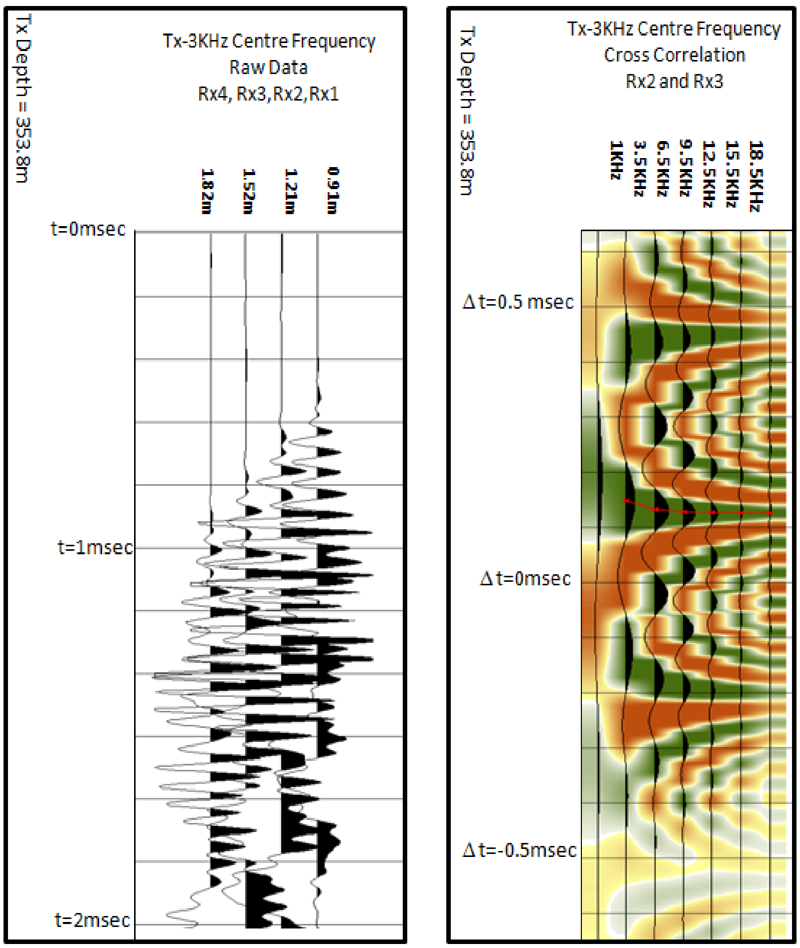

Figure 2. Example of full wave form sonic signal for dominate transmitter centre frequency set to 3 KHz (Left) and the cross correlation of traces for receivers 2 and 3 for a range of narrow frequency bands. Note the red line identifying the cross correlation peak amplitude, shows a small increase in travel time between Rx2 and Rx3 with decreasing frequency (i.e. velocity dispersion).

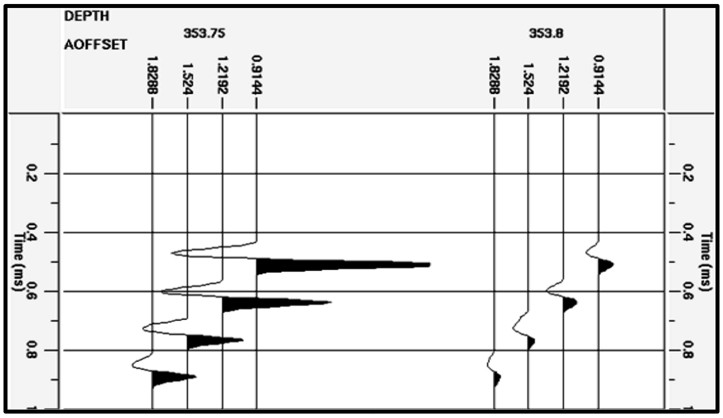

Figure 3. Comparison of first arrival pulse for dominate transmitter pulse centre frequency set to 15 KHz (left) and dominate transmitter pulse frequency set to 3KHz (right). Transmitter receiver separation is provided at absolute offset or "AOFFSET" on the image.

Note that in general the transmitted pulse contains measureable spectral energy over a broad range of frequencies. The result is that it is possible to recover an estimate of velocity dispersions between any two receivers (e.g. Rx1 to Rx2, Rx2 to Rx3, and Rx3 to Rx4 or Rx1 to Rx4 etc) for every dominant transmitter pulse centre frequency selected. That is, for a 100 m logged interval we could have 3000 independent estimates of velocity dispersion for each centre frequency selected if down hole sampling is set to 10cm.

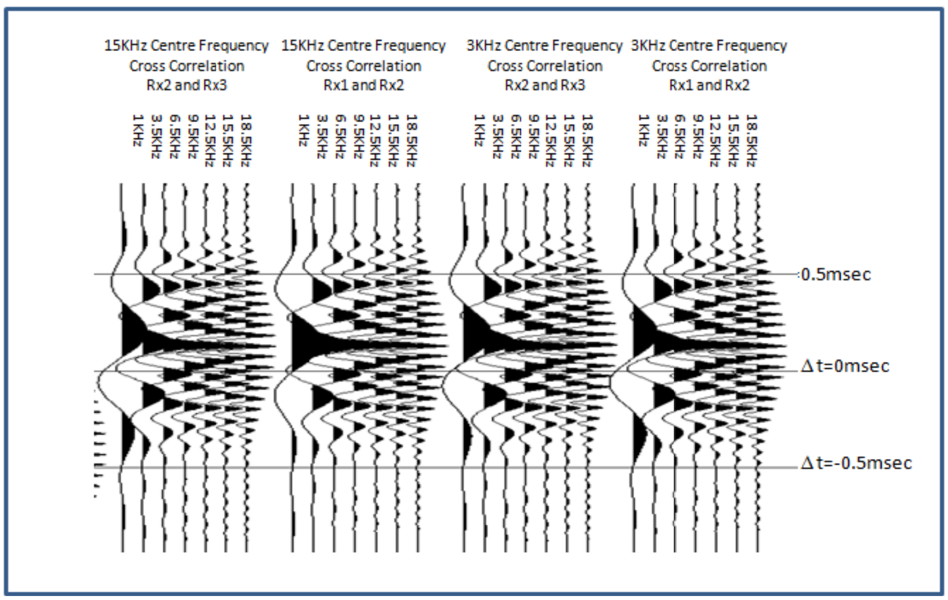

Figure 4. Example of our independent computations of travel time with frequency between the indicated receivers for dominate transmitter pulse centre frequency set to 15KHz and 3KHz at a depth 353 m below ground level. The Rx separation divided by travel time between the receivers is velocity. Note all curves show a slight increase in the maximum cross correlation amplitude with decreasing frequency (indicating dispersion). The cross correlation traces have been normalized, as the amplitudes at the extreme high and low frequencies tends to be very small in particular for 3KHz data. The reason for acquiring data at multiple transmitter pulse frequencies is to broaden the frequency range over which high signal to noise is achieved.

As a simple alternative method for comparison with the cross correlation technique above we apply a narrow bandpass filter to the raw FWF sonic records for Rx1, Rx2, Rx3 and Rx4 at all depths then apply semblance analysis to map slowness for the different wave modes. The narrow bandpass filters were focused at 25,16, 9, 6, and 1KHz. It's possible that 25KHz and 1 KHz are unreliable (i.e. outside the range where the raw data has sufficient spectral energy), however the

16KHz, 9KHz, and 6KHz should provide relatively robust results for the sandstone intervals. Figure 5 below shows the estimated slowness based on semblance using 4 receivers for the narrow frequency bands centers at 25, 16, 9, 6, and 1KHz. Sandstone and claystone layers are clearly located and are characterized by low gamma, low electrical conductivity and high acoustic velocity. For the sandstone dominated intervals, gamma is generally low and constant, while electrical conductivity increases with depth (i.e. increasing solute concentration) and velocity also increases with depth (with increasing confining pressure and compaction).

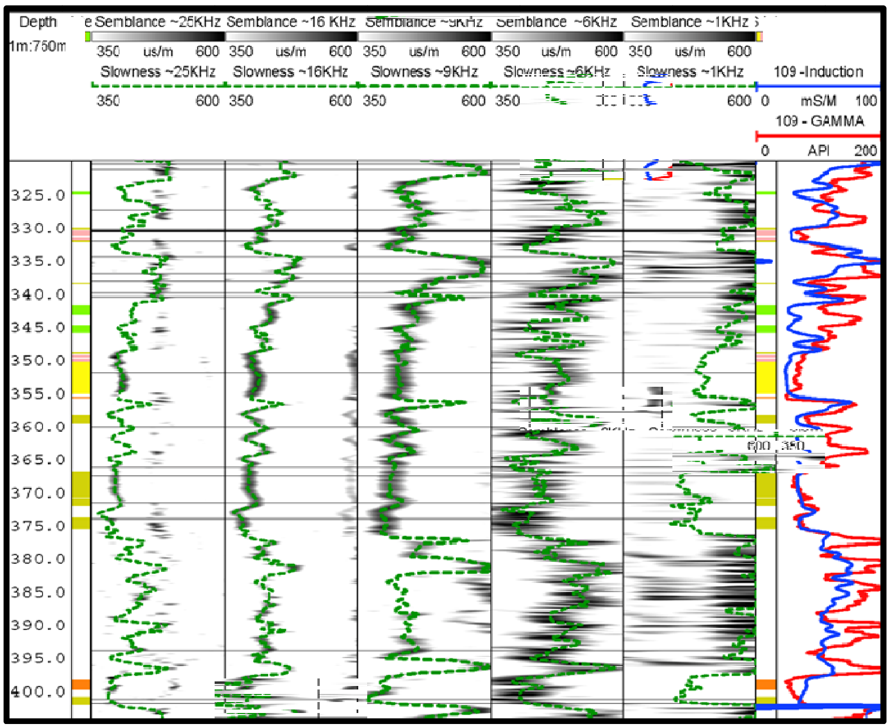

Figure 5. Slowness (the reciprocal of velocity) derived from semblance analysis including 4 receivers for 5 selected narrow frequency bands. Note that slowness tends to increase with decreasing frequency (i.e. velocity dispersion) however signal to noise ratio decreases with decreasing dominate frequency.

Notice that slowness in the marked sand intervals tends to decreasing depth. This highlights one of the difficulties for direct interpretation of compressional wave velocities in the very near surface. That is, acoustic velocities in the same formation tend to increase with increasing confining pressure (e.g. increasing depth of burial). Also notices that the picked velocity curve is more erratic for the lower frequencies bands. This is because of three main reasons: a) spectral energy is lower at lower frequencies, b) the semblance method relies on a semi-qualitative judgement as to which peak is connected to which wave propagation mode and c) at lower frequencies the receiver tends to move within the near field of the transmitter. Although semblance is common used for FWF sonic analysis (i.e. it is fast and simple to apply), it is certainly not the best methods for obtaining velocity dispersion. Velocity dispersion has been estimated at a single transmitter depth for each of the sandstone dominated layers from the FWF sonic data. The selected depths include, 331.5m, 342.5m,353m, 368, and 400m below ground level. The computed dispersion curves for the specific depths are provided on Figure 6 below. The dispersion curves are not completely defined by the FWF sonic data. However the basic shape of the dispersion curves is emerging from the data and may be sufficient to provide insight into the relative permeability of the Sandstone at each depth. That is, several wave propagation models suggest that as the centre of the velocity dispersion is located further towards the low frequencies, permeable should increase. Figure 6, below also shows approximate frequency ranges for FWF sonic data, surface seismic data, and ultrasonic measurement on core samples. That is, the measurable spectral energy for surface seismic is typically in the range from 0.01 to 0.2 KHz, for FWF sonic the range is 1 to 30000 KHz and for ultrasonic measurements on core the approximate range is 100 KHz to 10MHz. The range of frequencies captured by the FWF sonic measurements should be sufficient to characterize hydraulic permeability over several orders of magnitude. However Muller and Gurevich, 2005 explore highly plausible relationships between wave induced fluid flow, velocity dispersion and formation heterogeneity. The relationship are likely to be important at lower frequencies

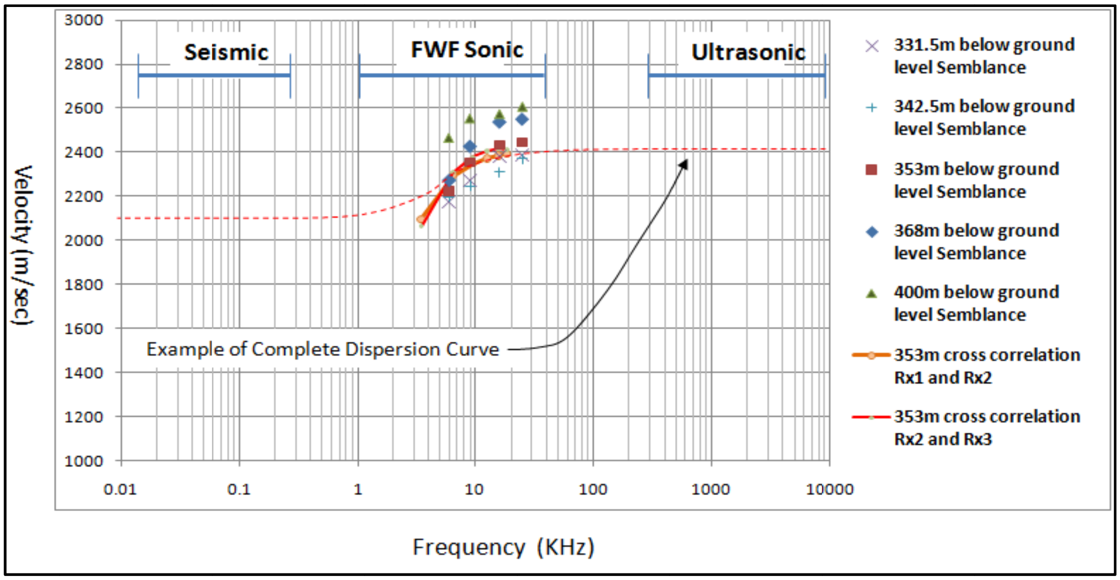

Figure 6 Example of partial dispersion curves computed from FWF sonic data acquired at five specific depth in sandstones intervals at the M345 Trial ASR site in Perth Western Australia. Notice that velocity increases with depth (i.e. increasing compaction and confining pressure) and that the high frequency velocity limit appears to occurs at between 10 and 20KHz. Although it cannot be directly demonstrated we would suspect that the dispersion curve at depth 400m (green triangles) would represent the most permeable sample as the onset of the velocity dispersion does not start until below 9 KHz.

A VSP survey was also acquired in monitoring well M345-109. The survey was completed with a hydrophone string and weight drop source (i.e ~1000 kg). The hydrophones interval was 10 m and hydrophones spanned the full injection interval. The large ten meter interval means that the transmitted pulse must travel through several sand and shale for any two receivers and can only provide large scale indication of velocity. While it's difficult to relate the low frequency VSP velocities (made over a 10m interval) to FWF sonic velocities (made over a 30cm interval), it is worth noting that velocities at approximately 353 m were close to 2000m/sec. This value of approximately to 2000 m/sec would not be unreasonable as the low frequency limit for the dispersion curve on Figure 6 for the sandstone at depth 353. Stoneley wave data has also been analysed for drill hole M345-109 with some notable correlation with expected zones (i.e. sandstones) of high permeability inferred from flow logging in the Injection Well M345-207 (i.e. 30 m away).

CONCLUSION

We have shown how partial velocity dispersion curves for both compressional and stoneley waves can be extracted from the field FWF sonic wire-line logging data and that the dispersion could potentially be interpreted to recover estimates of permeability distribution. Establishing a robust method from recovering and interpreting dispersion curves remains challenging. It is likely that VSP surveys can be used to recover larger scale near well hydro-stratigraphy. It is also likely that VSP may provide the lower frequency velocities needed to more fully complete velocity dispersion curves obtained from multi-frequency FWF sonic wire line logging data. The combination of multi-frequency FWF sonic wire-logging and VSP offers the potential to resolve hydraulic and mechanical parameters in the near well environment. A key point is that the full wave form sonic data could be acquired in less than 3 hours and the VSP took a similar amount of time. Our experience has been that most of the time required for both FWF sonic and VSP methods was in the processing and interpretation. In time and with continuing research processing and interpretation will become stream lined making detailed analysis more widely accessible in hydrogeology. In figure 6 we showed five partial velocity dispersion curves. It should be emphasized that there are similar curves for every 10 cm of the injection interval at multiple selected dominate transmitted pulse frequencies. That is, there are several thousand curves representing the hydraulic and mechanical properties of the formation. At this stage there are too few field examples to support definite interpretations (e.g. explicit values for permeability), however the conversion of such curves to hydraulic parameters will evolve with research programs like that being conducted at the M345 trial aquifer storage and recovery site; Perth Western Australia.

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CHARACTERIZATION OF WEATHERED BASEMENT AQUIFERS: IMPLICATION FOR GROUNDWATER RECHARGE

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ABSTRACT

Analyses and interpretation of aquifer tests of 21 boreholes in the weathered aquifers of Ibadan metropolis, SW-Nigeria were carried to characterize the influence of bedrock types (i.e. banded gneiss, augen gneiss, and quartz-schist) on the hydraulic properties and implications for groundwater recharge. The study involved evaluation of borehole data and pumping / sludge test data, on the basis of which hydraulic properties of the weathered regolith/fracture crystalline aquifer were estimated. The results revealed varied weathered regolith thickness of 21.6 – 60m in banded gneiss, 12.1 – 68m in augen gneiss and 60 – 87m in quartz-schist. The observed yield is generally low with average value of about 80.8m^3/d in all the three bedrock settings. The estimated average specific capacity (Sc), range from 5.2m^2/d in the banded gneiss and quartz-schist to 13.8m^2/d in the augen gneiss. Further evaluation of overall aquifer parameters show that the augen gneiss setting exhibits higher hydraulic potentials in terms of infiltration and aquifer recharge compared to banded gneiss and quartz-schist settings. Nonetheless, weak correlations (R =<0.1) of the yield and Sc with respect to saturated aquifer and total regolith thicknesses are indications of the interplay of the bedrock geology and hydraulic characteristics on the infiltration process and aquifer recharge.

Keywords: Weathered basement aquifer, Bedrock type, Hydraulic Characteristics, Ibadan, SW-Nigeria.

INTRODUCTION

Over the years, there has been increasing interest in groundwater resources throughout the world. This interest stemmed from a combination of increase in groundwater development for public and domestic uses as well as increase in water demands for agricultural and industrial activities with attendant negative impact on groundwater quality. Therefore, there is the need for detail assessment and evaluation of recharge potentials based on hydrogeologic characteristics of aquifers. Several hydrogeological studies had reported the complexity of the crystalline basement aquifers in terms of groundwater occurrences as well as in terms of hydraulic characterization (Chilton and Foster, 1995; Kellett, 2004). Despite extensive studies in the developed regions of the world, a direct generalization regarding hydraulic characteristics and groundwater potentials are not possible due to complex aquifer characteristics / geometry of the crystalline Basement Complex setting (Gustafson and Krásný, 1994). The weathered regolith can be a poor groundwater reservoir, especially where it is composed of clayey materials, while due to the inhomogeneous subsurface basement configuration, there is a serious challenges to hydraulic characterization of such complex subsurface aquiferous medium.

Many developing countries of sub-Sahara Africa, like Nigeria, are characterized by complex crystalline basement bedrocks and as such detail evaluation and understanding of the hydraulic characteristic of the associated aquifers are lacking. This is, no doubt, a major constraint on the sustainable water resources evaluation and aquifer management. However, most of the basement bedrocks have, over the past geologic periods, undergone several stages of deformation and denudation at varying degrees with resulting secondary porosity that determines availability, quality, and hydraulic characteristics of groundwater system. Consequently, adequate groundwater resources evaluation and management requires the understanding of impact of bedrock types on the hydraulic characteristic of this complex basement aquifer on one hand. On the other hand, there is the need to assess the implications of such varied hydraulic characteristics with respect to groundwater recharge. Based on the above background, this study focused on the weathered basement (regolith) aquifers of south-western Nigeria, using parts of Ibadan metropolis, SW-Nigeria, as case study while the scopes of the study are:

(a) Assessment of bedrock type and related geological control on the hydraulic characters of the weathered basement aquifer.
(b) Evaluation of impacts of the hydraulic characteristic on the infiltration and groundwater recharge potentials of the bedrocks units.

Study Area: The study area encompassed parts of Ibadan metropolis and is situated between longitude 03°49'00"E and 03°59'00"E and between latitude 07°22'00"N and 07°28'00"N with aerial coverage of about 168km^2 (Fig.1). The study area is characterized by tropical humid climate with two distinct seasons: the wet season (March to October) with an average annual rainfall of about 1,250mm and the dry season (November to February). Geologically, the study area lies within the Precambrian Basement Complex of South-western Nigeria. The major rocks are quartzites, gneisses (partly as augen gneiss) and migmatite and intruded by pegmatites, quartz veins, aplites and dolerite dykes in several places. The availability of groundwater in the basement terrain would depend on the presence and extent of the weathered overburden/regolith as well as the presence of joints and fractures system in the underlying bedrock (Foster, 1984).

In the study area, the hydrogeological setting and occurrence of groundwater are characterized by weathered regolith unit underlain by fractured or unfractured crystalline bedrock unit with varied thicknesses depending on the extent of fracturing. This is consistent with hydrogeological profiles from other similar basement terrains in tropical regions (Tijani 1994; Uma and Kehinde, 1994; Edet and Okereke, 2005). However, the nature and hydraulic characteristics of the weathered saprolite unit influences the infiltration and attendant groundwater recharge.

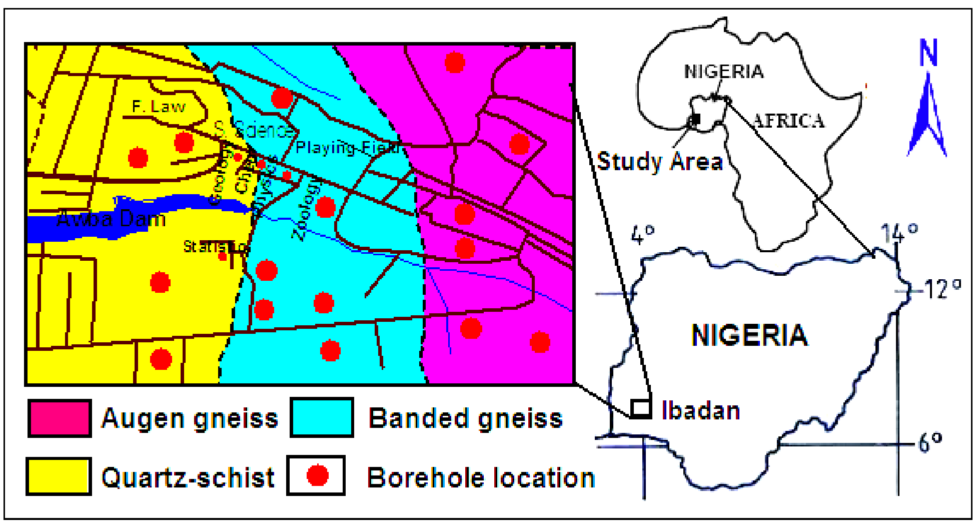


Figure 1: Location Map of the study area showing borehole locations

METHODS

In this study, single-well pumping tests and slug tests were carried out on boreholes and hand-dug wells respectively. A total of 21 private and institutional boreholes and wells distributed over three (3) crystalline bedrock types (6 in banded gneiss, 10 in augen gneiss and 5 in quartz-schist setting) were employed. For the borehole pumping data analyses, transmissivity and hydraulic conductivity were evaluated using software developed by Halford and Kuniansky (2002) for Cooper-Jacob (1946) method. Other parameters such as specific capacity, drawdown and yield were estimated from the pumping test data following standard procedure. For the slug test analyses, Bouwer and Rice (1976) and Hvorslev (1951) methods were employed. In addition statistical evaluation was used for correlation of aquifer and well hydraulic parameters in order to template possible influence of bedrock geology on the hydraulic characteristics of weathered regolith/fractured aquifers. Prior to pumping tests, data on well location, geology, depth and other relevant well inventory such as coordinates, elevation, depth to water, yield, regolith thickness and saturated thickness were also collated.

RESULTS AND DISCUSSION

Well inventories and Measured Parameters: The depth of wells within the different geologic settings range from 12.12 – 68m (mean 47.7m) for augen gneiss, 21.6 – 60m (mean 36.2m) for banded gneiss and 60 – 87m (mean 66.4m) for schistose quartzite setting (Table 1). This variability in well depths signifies the geologic control and differences in the extent of weathering. Nonetheless, the thicknesses are within the range that can be anticipated for shallow boreholes in a basement terrain aquifer (Tijani, 1994; Uma and Kehinde, 1994; Edet and Okereke, 2005). Depths to water level in the boreholes vary widely from <1m to more than 10m but generally less than 15m below ground surface in the different geologic settings with quartz-schist bedrock exhibiting relatively deeper water level. Such shallowness of water level is an indication of enhance infiltration and recharge of the weathered regolith aquifer. This apparently facilitates the development of hand-dug wells and shallow boreholes for domestic water supply in the study area. Saturated thickness in the boreholes shows a wide variation of 8.02m (in gneiss) to 81.58m (in the quartz schist setting). Higher saturated thickness in quartz-schist setting compared to banded gneiss and augen gneiss terrains, is an indication that the saturated thickness is dependent on the extent of weathered regolith and topographic disposition.

Table 1: Summary of well inventory and measure parameters of the weathered basement aquifer

Parameters	Banded gneiss (N = 6)			Augen gneiss (N = 13)			Quartzite (N = 5)		
	Min	Max	Mean	Min	Max	Mean	Min	Max	Mean
Well depth (m)	21.6	60.0	36.2	12.1	68.0	47.7	60.0	87.0	66.4
Depth to WL (m)	0.94	5.9	3.4	0.75	11.2	4.67	3.07	11.3	6.1
Drawdown (m)	2.2	26.4	13.7	3.3	29.4	11.3	15.1	34.7	28.9
Sat. thickness(m)	16.4	54.9	30.7	8.02	62.8	42.2	48.7	81.6	62.7
Yield (m^3/d)	31.0	88.6	55.9	30.0	138.2	88.7	62.8	86.4	75.4

The measured yields of the tested boreholes range generally between $30m^3/d$ – $138m^3/d$, and mean $80m^3/d$ (Table 1) with higher yields in the augen gneiss (range 30-138.2 m^3/d; mean $82.87m^3/d$). Banded gneiss sustained yields between 31-88.6 m^3/d; mean 55.9 m^3/d) while quartz-schist aquifer have yield of 62 – $86.4m^3/d$. High yields typify boreholes that intersect fractured bedrock with shallow weathered overburden, while low yield boreholes are associated with thick weathered regolith and no prominent fractured intersection. Nonetheless, the observed drawdowns were dependent on the pumping rate (yield) and transmissivity capacity of the aquifer which depend on the composition of the bedrock type.

Pumping Tests and Evaluated Hydraulic Parameters: In the study area, three basic types of curves were recurrent which represents the different ways in which the aquifers responded to pumping. The first scenario is typical of augen gneiss setting and represents a situation where discharge rate is proportional to in-flow into the well (Fig. 2). The second and third scenarios characterized all the 3 bedrock settings and typify recharge boundary; indicative of transmissive fracture system within the bedrock units on one hand and impermeable boundary condition; indicative of thick weathered regolith underlain by bedrock units with limited or fracturing on the other hand (Fig. 3).

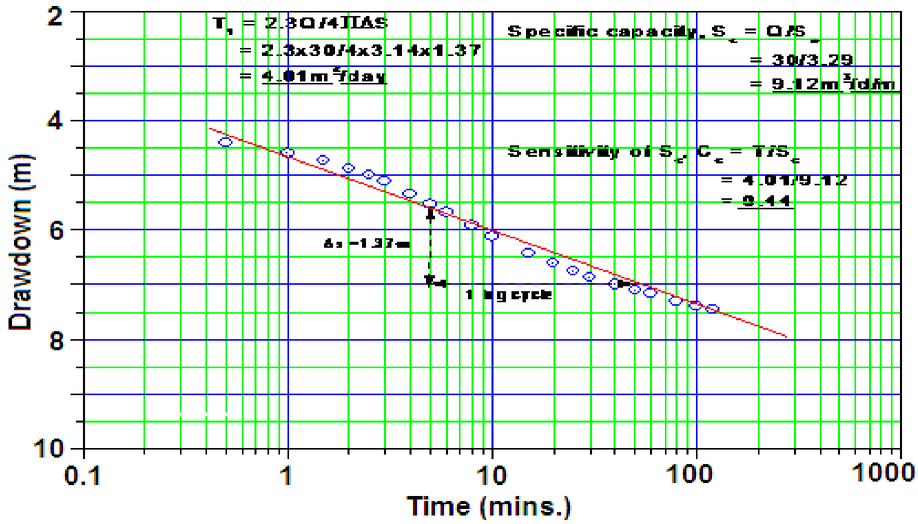


Figure 2: Representative time - drawdown plot indicating proportional discharge and inflow into well

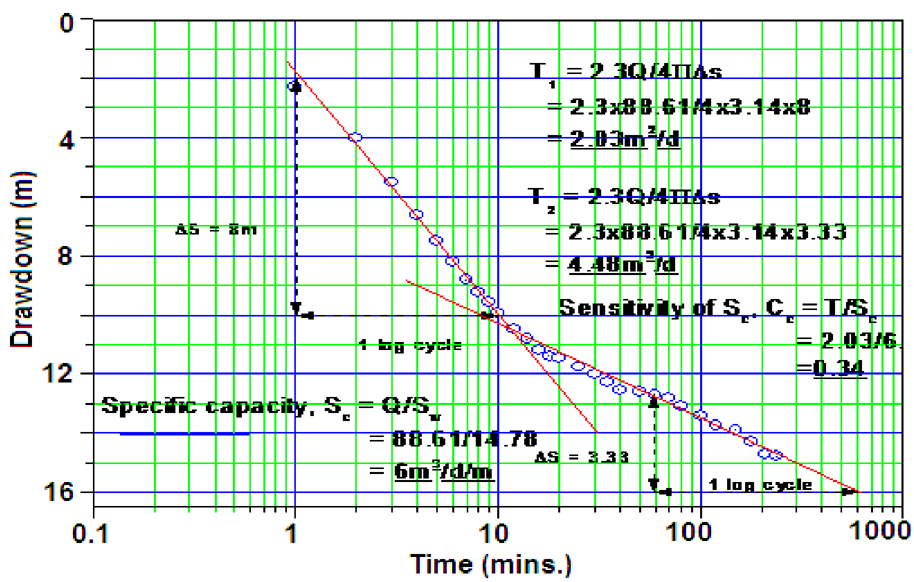


Figure 3: Representative time - drawdown plot indicating recharge boundary condition

Nonetheless, the observed boundary conditions are not bedrock dependent but can be attributed to structural heterogeneity which is characteristics of the crystalline basement rock setting.

Estimated hydraulic characteristics: The hydraulic conductivity was estimated from evaluated transmissivity and saturated thickness of the aquifer with values of 0.01 – 0.18m/d for banded gneiss and quartz-schist compared to 0.2-0.5m/d for augen gneiss (Table 2). These values cut across the various ranges of likely hydraulic conductivity for weathered granite and metamorphic rocks (Halford and Kuniansky, 2002) and are indicative of regolith aquifers with generally low permeability. Transmissivity in the three bedrock types range from 0.76-27.2 m^2/d (mean 7.3m^2/d) in augen gneiss, 1.1 – 4.2m^2/d (mean 2.8m^2/d) in banded gneiss and 0.41 – 10.6m^2/d (mean 2.7m^2/d) in the quartz-schist (Table 2). These imply that quartz-schist and banded gneiss settings show lower potential when compared to the augen gneiss; although low T values (<1 – 5m^2/d) are said to be more characteristic of the weathered basement aquifers (Offodile, 1983), except when there is a highly transmissive fractured zone.

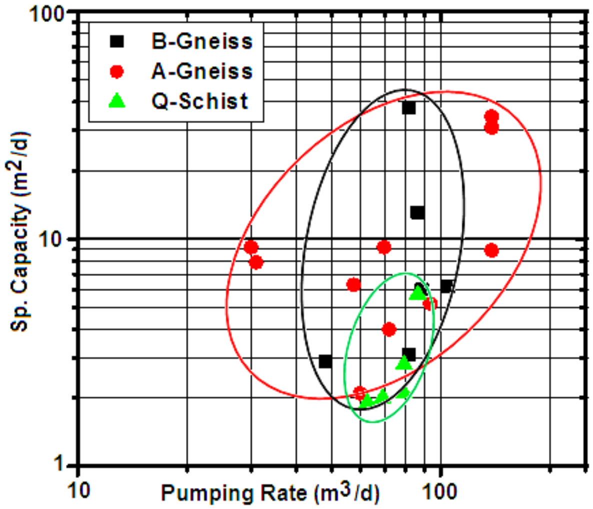
Specific capacity generally, gives a better indication of aquifer performance than yield since it also reflect aquifer transmissivity and saturated thickness (Mace, 2000; Uma and Kehinde, 1994). Specific capacity of the tested boreholes range from 1.3 – 37.8m^2/d; mean 13.0m^2/d (Table 2) for augen gneiss. Banded gneiss and quartz-schist have specific capacity of 2.9 – 7.9m^2/d (mean 5.6m^2/d) and 1.86 – 17.5m^2/d (mean 5.23m^2/d) respectively.

Table 2: Summary of evaluated hydraulic properties determined from pumping test

Parameters	Banded gneiss (N = 6)			Augen gneiss (N = 13)			Quartzite (N = 5)		
	Min	Max	Mean	Min	Max	Mean	Min	Max	Mean
Sp. Capacity (m^2/d)	2.92	7.9	5.6	1.25	37.8	13.0	1.86	17.5	5.23
Transmissivity (m^2/d)	1.10	4.16	2.8	0.76	27.2	7.25	0.41	10.6	2.71
Sensitivity, Cc	0.24	2.0	0.7	0.25	0.7	0.43	0.17	0.42	0.28
Hydr. Conduct.(m/d)	0.07	0.15	0.1	0.02	0.5	0.2	0.01	0.18	0.05

Furthermore, a plot of specific capacity (m^2/day) versus the borehole discharge/yield (Fig. 4a) revealed lower specific capacity for quartz schist setting with yield of 62-84m^3/day, suggesting influence of deep weathering. However, higher specific capacity of 1.3 - 37.8 m^2/day with average discharge of 88.6 m^3/day for augen gneiss can be attributed to the impacts of fracturing of the bedrock unit at depth.

However, a plot of specific yield as a function of well depth below the weathered layer (Fig. 4b) revealed a decrease with depth suggesting that the weathered overburden unit plays a significant role in terms of the permeability and recharge of the weathered basement aquifer especially for gneisses while the yield in quartz-schist is apparently fracture controlled through vertical drainage within the weathered regolith. Similar trend had been observed elsewhere by Cho et al., (2003). Furthermore, transmissivity is linearly related to specific capacity by a constant C_c called the sensitivity of Sp. Capacity i.e. T = Cc x SC. The values of the sensitivity of specific capacity (C_c) for the study area are presented in Table 2. The result reveals C_c values of 0.22 – 0.78 with a mean of 0.48 (i.e. T = 0.48SC) and median value of 0.38. This range of values correlate well with 0.23 and 0.34 determined by Adyalkar and Mani (1972) and as high as 0.44 for large-diameter wells in fracture aquifers.



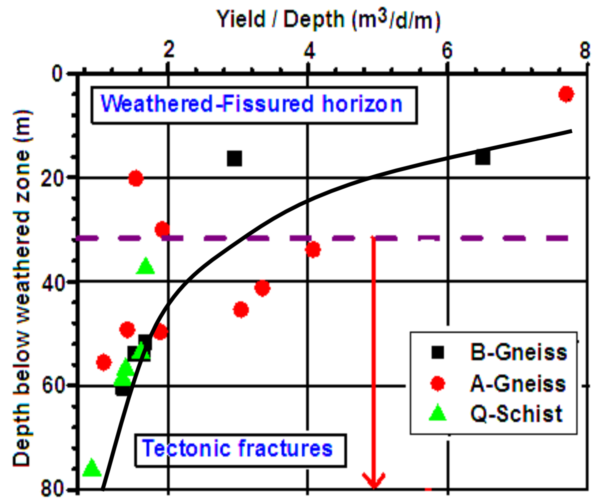


Figure 4: Plot of specific capacity (m^2/day) versus the borehole discharge/yield (a) and plot of vertical distribution of specific yield as a function of well depth below the weathered layer (b).

Slug test analysis result: The result of analysis of slug test carried out in four (4) 90cm-diameter hand-dug wells in the study area is presented Table 3. The hydraulic characteristics were analyzed using Bouwer and Rice (1976) and Hvorslev (1951) methods. As indicated, K ranges from 2.10 – 5.70m/d and 3.99 – 9.11m/d respectively while T ranges from 5.80 – 15.22m^2/d, and 6.34 – 34.98m^2/d respectively. Although, these values fall in the range of maximum for weathered granitic rocks as given by Halford and Kuniansky, 2002, these are however, inconsistent with values estimated from pumping test for the study area. Thus, since the slug tests were conducted in large diameter (90cm) shallow dug-wells, there is the need to correct for the effect of well diameter. The correction factor, Fc, for the result was derived by the author based on a simple mathematical relation from the general assumption for analyzing slug test ($F_c = (r_s/R_l)^2$).

Table 3: Summary of result of the analysis of slug test for the Bouwer and Rice (1976) and Hvorslev (1951) methods

Well ID	B	Bouwer Method				Hvorslev Method			
		K	T	Kcorr	Tcorr	K	T	Kcorr	Tcorr
UIB/A/WL1	1.6	3.7	5.8	0.1	0.2	4.0	6.3	0.1	0.2
UIB/A/WL2	2.7	5.7	15.2	0.2	0.5	8.5	22.6	0.3	0.7
UIB/A/WL3	3.4	2.1	7.1	0.1	0.2	4.4	15.0	0.1	0.5
UIB/A/WL5	3.8	3.2	12.3	0.1	0.4	9.1	35.0	0.3	1.1
Minimum	1.6	0.4	5.8	0.1	0.3	0.4	6.3	0.1	0.2
maximum	21.1	5.7	15.2	0.2	0.5	9.1	35.0	0.3	1.1
Mean	6.5	3.0	9.6	0.1	0.2	5.3	17.6	0.2	0.5
Median	3.4	3.2	7.6	0.1	0.2	4.4	15.0	0.2	0.5

B = saturated thickness; K_{corr} and T_{corr} = K and T corrected

For this study, test wells were corrected from 90cm to 150mm diameter as the standard borehole diameter in the study area using a correction factor (F_c) of 0.03. These resulted in a corrected K of 0.06 – 0.17m/d for Bouwer and Rice method and 0.12 – 0.27m/d for the Hvorslev method. Similarly, the corrected T ranges between 0.17 – 0.46m^2/d for the Bouwer and Rice method, and 0.19 – 1.05 for the Hvorslev method. These values are also similar to the values of the hydraulic parameters estimated for the aquifers from the pumping test analysis (cf. Table 2). This implies that the weathered regolith is of low transmissivity, and can improve with increasing surface area. The low values can be attributed to the high clay matrix in the weathered horizon with resultant impacts on infiltration and groundwater recharge.

CONCLUSION

The complexity of hydraulic characteristics of weathered basement aquifer is yet to be well understood as constraints to assessment of groundwater potentials. Consequently there is the general tendency to link aquifer potentials in terms infiltration and groundwater recharge performance to the bedrock mineralogy, structure features, nature / thickness of weathered regolith and saturated thickness. From the assessment of the degree of interrelationship among the aquifer parameters (transmissivity, specific capacity and yield versus the saturated thickness and the regolith thickness) revealed poor positive correlation between yield and regolith thickness (R=0.26); yield and saturated thickness (R=0.29). Also transmissivity exhibits a much weaker correlation with regolith (0.06) and saturated thicknesses (0.15) respectively. The implication is that increasing regolith and saturated thickness have minor effect on well recharge and yield of a weathered basement aquifer.

However, the overall assessment presented in this study has revealed that much as the assumptions are plausible, not all the parameters have noticeable impact on the recharge and productivity of regolith aquifers. It can be inferred that bedrock type and geologic structure influence regolith aquifer recharge and yield positively, while regolith thickness and saturated thickness are shown to have no significant influence. However, hydraulic indices (T, K, and specific capacity) are important controlling factors; adequate knowledge and characterization of which are necessary for effective and sustainable groundwater development and management.

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Hydrogeology of ASR - Lessons from Over 60 Years of Global Practice

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Abstract

Historic ASR experience demonstrates that system performance is highly dependent on local hydrogeological conditions, which may not be always locally favorable. Hydrogeological conditions must allow for the achievement of storage objectives, which differ between systems. ASR systems that have failed or underperformed often had early indicators of adverse hydrogeological conditions. Injected waters are often in chemical disequilibria with storage zone minerals. As a result, adverse fluid-rock interactions have occurred in some systems such as leaching of arsenic and metals.

Groundwater flow, solute-transport, and geochemical modeling can provide great value for evaluation of potential systems and the optimization of system design and operation. However, accurate modeling requires high-quality hydrogeological data, particularly for key variables that influence system performance, such as storage zone water quality, permeability, porosity types (matrix versus dual porosity), confining strata leakance, aquifer heterogeneity, injected and native groundwater chemistry, and storage-zone mineralogy. Greater value needs to be obtained from the hydrogeological data collected, often at a considerable expense. Workflow software can facilitate integration of diverse hydrogeological data into groundwater models. The critical lesson for the future is the importance of taking full advantage of past experiences and existing aquifer characterization and modeling technology to assess potential system performance in advance of construction.

Key words
ASR, hydrogeology, modeling, geochemistry, aquifer characterization

INTRODUCTION

A fundamental challenge in the management of water resources is that in many places, there is inadequate storage capacity. Available water supply may be adequate to meet all or most of the long-term average demands, but seasonal or longer-term imbalances occur between supply and demand. Aquifer storage and recovery (ASR) is increasingly being looked to as a means for providing large-capacity storage of water in order to improve overall water management. The advantages of ASR are compelling. ASR systems can store very large volumes of storage at a fraction of the economic and environmental costs of surface storage systems. ASR systems also avoid the evaporative losses associated with surface reservoirs and are less vulnerable to contamination.

The development of ASR has been evolutionary rather than revolutionary, and it is difficult to establish when the technology was discovered or invented. The first successful test of an actual ASR system appears to have been performed at Camp Peary, near Williamsburg, Virginia, in April and May 1946. The test involved storing fresh surface water in an aquifer containing brackish water (Cederstrom, 1947, 1957). The Cederstrom test successfully demonstrated that ASR could work, and it provides a historic understanding of some of the basic concepts and issues that are still important for the design and operation of ASR systems today. Cederstrom (1957), for example, described the basis for his 1946 experiment as follows:

> *The writer believed that, if fresh water was poured down a well reaching beds saturated with brackish water, a complete mixing of the fresh water with the brackish water would not necessarily result. In the first place, fresh water is less dense than brackish water and would have a tendency to “float” on the heavier water. Furthermore, since the movement of water through the interstices of sandy sediments is extremely slow and turbulence is lacking except in the immediate vicinity of the well screen, the recharge water, regardless of its*

specific gravity, might tend to push back the ground water, maintaining a rather narrow zone of diffusion between.

In the over 60 years since the Cederstrom test, much has been learned about the hydrogeological controls over the performance of ASR systems. ASR is a proven technology, but it does not work everywhere. The success of ASR systems, as far as providing additional water when needed at a quality suitable for its intended use, is dependent upon the occurrence of a variety of favorable hydrogeologic conditions, which are not present at all sites. There is an adage in engineering that one learns more from failures than successes. There is an unfortunate human tendency to want to bury one's mistakes or failures and trumpet one's successes, which results in valuable lessons being lost. Unsuccessful ASR systems tend to quietly fade away as neither the hydrogeologists, engineers, owners or operators want to draw any attention to them. However, science progressively builds upon the results, both positive and negative, of earlier research and experiences. The development of ASR is no exception. A key to improving future ASR implementation is to learn from and build upon the experiences of earlier systems.

ASR SYSTEM TYPES

ASR systems vary in how they store water and thus the hydrogeologic controls over their performance. There are three main system types, which are referred to as physical-storage, chemically bounded, and regulatory-storage systems (Maliva and Missimer, 2008, 2010a, 2010b). Physical-storage ASR systems achieve the useful storage of water (i.e., injection results in a net increase in available water), by actually increasing the volume of water present in the aquifer, as manifested by an increase in aquifer water levels or heads.

Chemically bounded ASR systems store freshwater by the lateral displacement of poorer quality water in the aquifer. The most common type of chemically bounded ASR systems use a storage zone that contains brackish groundwater. Injection of freshwater laterally displaces the brackish water. System performance is quantified based on the percentage of the injected water that can be recovered at a usable quality (i.e., recovery efficiency).

Regulatory storage ASR systems are essentially water banking systems. Injection of a given volume of freshwater into an aquifer confers the right to the system owner or operator to later withdraw water. Typically, regulatory-storage ASR systems store freshwater into a freshwater aquifer.

A fundamental issue for ASR is the importance of fully understanding the system type, the hydrogeologic issues with the system type, and the development of performance criteria and targets that are appropriate to the system type (Maliva and Missimer, 2010a). For example, recovery efficiency inherently must be defined using different criteria for physical-storage and chemically bounded ASR systems.

Hydrogeology of Physical-Storage ASR Systems

Injection of freshwater into a physical-storage ASR system must result in a <u>persistent</u> increase in aquifer water levels or heads for injection to have provided any water resources benefits. The word "persistent" is emphasized; an increase in water levels from injection must still be present at the time of recovery. A key issue is differentiating between the mounding of water during injection and a long-term increase in aquifer water levels for an increase in stored water volume (Figure 1). The former is a dynamic response to injection, analogous to the cone of depression (drawdown) that results from pumping a well. If there is no persistent increase in aquifer water levels, then no net increase in storage has occurred, and the injected water has either leaked out of the aquifer or has otherwise been lost to discharge or pumping.

The storage zone of physical-storage ASR systems must be effectively confined at its base and sides in order to store water. The underlying and horizontal bounding strata act as the "walls of the tank." The storage zone must also not have too large an areal extent relative to the volume of injected water, otherwise, the increase in stored water volume would not result in a discernable increase in aquifer water levels or heads. The theoretical increase in heads can be calculated from the aquifer area, storativity, and injected volume.

Surprisingly, there is very little discussion in the literature of water level increases and the recovery efficiency of physical-storage ASR systems even though this issue should be a paramount concern. Foxworthy and Bryant (1967) documented injection tests in the basalts of the Columbia River Group Aquifer at The Dalles, Oregon, and reported that when injection stopped, the groundwater mound in the vicinity of the well dissipated rapidly, and the water level in the well declined to pre-recharge static level within 56 minutes. The AWWA (2003) similarly reported that the one ASR project in Virginia was initially expected to help recover groundwater levels, but because of the time difference between the injection and withdrawal, the water levels had reached equilibrium, and there was no mound by the time the water was withdrawn, causing a cone of depression to form during recovery. The San Antonio Water System (SAWS) Twin Oaks ASR system stores freshwater from the Edwards Aquifer in the Carrizo-Wilcox (Carrizo) Aquifer, a regional aquifer that also contains freshwater. In the absence of significant local residual pressure buildups from injection, a cone of depression forms during recovery that adversely impacts other existing aquifer users. The Carrizo Aquifer Well Mitigation Program was implemented to address adverse impacts to existing aquifer users during recovery (Evergreen Underground Water Conservation District, 2006). Physical-storage ASR will, therefore, not work in a regional aquifer because the increase in aquifer water levels will be dissipated across the aquifer. In the absence of a local persistent increase in heads, the ASR system behaves as just another production well or wellfield at the time of recovery with its associated impacts.

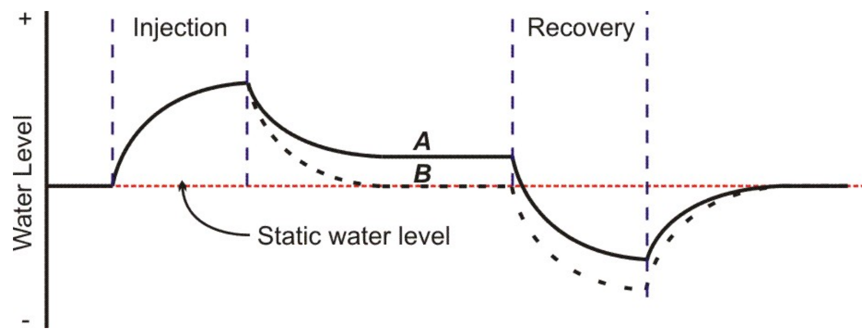

Figure 1. Conceptual diagram of water level response to injection and recovery for physical-storage ASR systems. Water levels must persist above static water level (track A) after injection in order for injection to have any benefit. In very large or leaky aquifers, water levels recovered to static water level after both injection and recovery (track B).

The largest and most successful physical-storage ASR system is the Las Vegas Valley Water District artificial recharge system. The storage zone, the Las Vegas valley basin, is a largely closed basin that formed primarily by a middle Miocene extensional event. The basin is filled with up to 1,520 m (5,000 ft) of mostly siliciclastic deposits that range in age from Miocene to Holocene. The bedrock of the basin consists of Precambrian metamorphic rock, Precambrian and Paleozoic carbonate rock, Permian to Jurassic siliciclastic rock, and Miocene igneous rock (Maxey and Jameson, 1948; Malmberg,1965; Plume, 1989; Zikmund, 1996). The Las Vegas valley has hydrogeologic conditions particularly favorable for a physical-storage ASR system. It is a closed basin, so stored water will not leak out of the basin to a significant degree. The use of confined storage zones provides protection of stored water from surficial contamination. Its overdrafted condition provides storage space for artificial recharge. Long-term hydrographs show increases in static water since the start of recharge of as much as 30 m (100 ft) in some wells (Las Vegas Valley Water District, 2008).

A lesson for physical-storage ASR systems is the importance of (1) evaluation of whether or not physical-storage goals (i.e., persistent increase in water levels at the system site) are theoretically possible, and (2) evaluation of whether local hydrogeologic conditions may allow for physical storage to practically occur. Fortunately, the latter can be readily evaluated through groundwater flow modeling. For example, initial groundwater modeling of the SAWS Twin Oaks ASR system by CH2M Hill (2000) indicated that for seasonal 60 million U.S. gallon per day (Mgd; 227,000 m^3/d) ASR operational cycles, water levels after both injection and recovery rapidly return to static conditions after aquifer stresses cease. Maliva and Missimer (2008b) demonstrated that the physical storage was not possible in the regional Floridan Aquifer System of South Florida, U.S.A. Once a basin is identified that has suitable dimensions and geometry for physical-storage ASR, the next step is evaluating the effectiveness of its confinement, which is technically more challenging and may require actual pilot system testing.

Hydrogeology of Chemically Bounded ASR Systems

The hydrogeological controls over the performance of chemically bounded ASR systems that use brackish and saline storage zones have, by far, received the most study of all ASR systems types. Reviews of the hydrogeology of chemically bounded ASR systems have been provided by Brown (2005), Maliva et al. (2005), and Maliva and Missimer (2006, 2010a). The recovery efficiency of chemically bounded ASR systems depends upon both the mixing of injected water and native groundwater and the migration of injected waters. Injected water may migrate upwards due to buoyancy differences (density stratification) and laterally under prevailing hydraulic gradients. The performance of chemically bounded ASR systems is thus highly dependent on local hydrogeologic conditions, particularly in the vicinity of the ASR wells.

One of the most important variables impacting the performance of chemically- bounded ASR systems is the salinity of the native groundwater in the storage zone. All other factors being equal, higher salinities result in poorer recovery efficiencies for two main reasons. First, as salinity increases, it takes progressively less native groundwater mixing with injected water to cause the salinity of the injected water to rise to unacceptable levels. Secondly, increasing native groundwater salinities result in a greater density difference between injected freshwater and native groundwater, which results in greater buoyancy-induced water migration. The lighter freshwater will tend to migrate upwards and outwards toward the top of the storage zone, and the denser native groundwater will move inwards at the bottom of the storage zone.

Aquifer heterogeneity can also profoundly impact ASR system performance. All aquifers are heterogeneous in that there are spatial variations in their hydraulic properties, particularly hydraulic conductivity. Two main types of heterogeneity are of concern for ASR systems; layered and dual-porosity systems. Layered heterogeneity is caused by variations in hydraulic conductivity between the beds of rock or sediment that constitute the aquifer. Dual-porosity systems are characterized by the presence of flow conduits, such as fractures or solution conduits, that have orders of magnitude greater hydraulic conductivity than the surrounding rock (matrix). The above-noted types of heterogeneity often coincide. For example, a layer in an aquifer may have a much greater hydraulic conductivity than the rest of the aquifer because of the presence of fracturing or dissolution.

During injection, water will enter aquifer strata at rates approximately proportional to their transmissivity (average hydraulic conductivity multiplied by layer thickness). In heterogeneous aquifers, most of the injected water will preferentially enter the most transmissive beds and also into fractures and other flow conduits that have the greatest hydraulic conductivity. Dual-porosity systems are particularly problematic because the flow conduits typically constitute a very small volumetric percentage of an aquifer, usually less than one percent. The injected water will necessarily have a great areal extent, perhaps orders of magnitude greater than would be predicted based on the assumption of cylindrical flow from the ASR well. Furthermore, according to Darcy's Law, flow velocity is proportional to hydraulic conductivity, which means that the rate of fluid migration in beds with high hydraulic conductivities and within flow cavities may be much greater than rates calculated using an average hydraulic conductivity value calculated by dividing aquifer transmissivity by thickness. Greater flow velocities also result in greater degrees of dispersive mixing.

Water injected in highly heterogeneous aquifers would thus expect to undergo greater degrees of mixing with native groundwaters and more rapid and unpredictable migration than would be occurring in less heterogeneous aquifers. ASR systems that have had very poor recovery efficiencies in Florida (Bonita Springs, Okeechobee, West Palm Beach, and Northwest Hillsborough County) were all highly heterogeneous. One means to evaluate the degree to which a dual-porosity system may exist is to compare the average hydraulic conductivity estimated from pumping tests with the likely matrix values of the aquifer rock. For the City of West Palm Beach ASR system, the average hydraulic conductivity of the upper flow zone located from 292 to 317 m (985 to 1040 feet) below land surface is 389 to 497 m/d (1,276 to 1,630 ft/d), assuming that interval provides 65% of the transmissivity, as indicated by flow meter logs (CH2M Hill, 1998). The hydraulic conductivity of the matrix of similar limestones in west-central Florida is 0.08 to 4.11 m/d (0.27 to 13.5 ft/d, 100 to 5,000 millidarcies; Budd and Vacher, 2004). The average hydraulic conductivity of the upper flow zone is on the order of 100 times greater than that of the limestone matrix, which indicates that virtually all of the flow is through fractures and/or other secondary pores.

Hydrogeology of Regulatory-Storage ASR Systems

Regulatory-storage ASR systems are very attractive to system owners and operators because recovery volumes are essentially a bookkeeping issue rather than a hydrogeologic issue. Injection confers the right to withdraw all or most of the injected water, usually irrespective of water quality and level issues. Regulatory-storage ASR is implemented in areas where groundwater usage is deemed to be at or near sustainable limits. If new groundwater withdrawals could still be permitted, then there would be no reason for ASR. In the absence of a local persistent increase in aquifer water levels, regulatory-storage ASR systems could potentially exacerbate water management problems. For example, if local groundwater withdrawals are being limited because of concerns over wetland water levels or spring or stream flows during dry periods, then recovery from ASR systems during these periods could actually make the situation worse. A key lesson for the implementation of regulatory storage is the evaluation of the impacts of operating the system during both injection and recovery periods.

Another key lesson is the importance of the integration of the ASR system into the overall groundwater management of the basin, which was illustrated by the Las Posas Basin ASR system in California. The Calleguas Municipal Water District (CMWD) and Metropolitan Water District of Southern California (MWD) developed an ASR project in the Las Posas Groundwater Basin located in southern Ventura County, California. The objective of the project was to store up to 370 X 10^6 m^3 (300,000 acre feet, (AF)) of excess surface water obtained from the MWD. The stored water would be retrieved and treated if the state supply of water is reduced or disrupted (Calleguas Municipal Water District, 2008). A key issue is that the annual injection into the Las Posas Groundwater Basin is a small fraction (≤ 6%) of the total water budget of the basin. Whether or not the system achieves its long-term storage goal, as evaluated by an increase in aquifer heads, will thus depend, to a large degree, on the extractions of other aquifer users. If the aquifer is otherwise in over-draft, then some or all of the injected water will be extracted by other aquifer users.

Water use in the Las Posas Groundwater Basin is regulated by the Fox Canyon Groundwater Management Agency (FCGMA). The FCGMA has a water credit system in which credits are issued for water recharged using an injection well or for allocated water not used (in-lieu recharge). The credits are issued on a one-for-one volumetric basis and can be used in future years to offset overuse of groundwater resources. The FCGMA experience reveals a critical limitation of the regulatory storage or water banking concept. The historic accumulation of credits within the FCGMA has been steadily increasing, approaching 678 X 10^6 m^3 (550,000 AF) in 2006 (Fox Canyon Groundwater Management Agency, 2006). The accumulated credits are over four times the annual extraction rate and greatly exceed the amount of water that could be extracted during a short-time period (e.g., major drought). The Fox Canyon Groundwater Management Agency (2006) noted that even a 5% use of the total amount of credits currently available would result in a net 24% increase in annual extraction, which could result in persistent depressions in groundwater elevations, land subsidence, and seawater intrusion (Fox Canyon Groundwater Management Agency, 2006).

Regulatory-storage ASR thus needs to be implemented and managed in a manner so that system users have reasonable expectations that can be met as to when and how much water can be withdrawn. An important lesson is that groundwater basins have a limited safe withdrawal capacity and need to be managed so that excessive credits are not accumulated. Limitations on withdrawals affect the cost-benefit ratio, and therefore, the economic viability of ASR projects need to be determined early in the development or regulatory-storage schemes.

GEOCHEMICAL LESSONS

The leaching of metals and metalloids, particularly arsenic, into water stored in ASR systems has become a major operational and regulatory issue in the United States. Elevated arsenic concentrations may make the recovery water unsuitable for potable use. Metals leaching can also be a violation of United States Environmental Protection Agency and state underground injection control rules if underground injection <u>causes</u> contamination of an underground source of drinking water. Contamination is defined as an exceedance of any applicable groundwater standards. The leaching of arsenic and some metals appears to be due to the introduction of oxygenated water and chlorine into aquifers containing reducing conditions and arsenic-bearing iron sulfide minerals (Arthur et al., 2001, 2002, 2007; Mirecki, 2004, 2006a, 2006b).

Arsenic leaching in stored water was a major surprise, despite the fact that it was well known in aquatic geochemistry literature that the introduction of oxygenated water into aquifers containing reducing conditions can result in the mobilization and concentration of metals present in reduced minerals. Uranium roll front deposits are a natural example of the process (e.g., Drever, 1997). In-situ uranium leach mining processes intentionally take advantage of the process to recover metals by injecting oxygenated freshwater into formations containing uranium present in reduced mineral phases. A key lesson is that a more sophisticated approach needs to be taken to assess potential adverse fluid-rock interactions in ASR systems and to learn how to cost-effectively manage them.

PROJECT DEVELOPMENT AND OPERATIONAL LESSONS

ASR Does Not Work Everywhere and Risk Needs to be Considered During Decision Making

Chemically bounded and physical-storage ASR systems require an envelope of favorable hydrogeologic conditions to be present in order to achieve acceptable recovery efficiencies. Perhaps the most important historic lesson concerning ASR is that the favorable hydrogeologic conditions necessary for successful implementation of ASR do not occur everywhere. ASR systems have failed and have been abandoned. Adverse hydrogeologic conditions may not be fully revealed until the start of operational testing. A difficult-to-quantify risk element is associated with ASR projects that needs to be incorporated into economic (cost-benefit) analyses. The risk element differentiates ASR systems from most other utility projects. For example, a utility owner or operator has a very reasonable expectation that an above ground storage tank will hold its design capacity of water, but no such guarantee is possible as far as the recovery efficiency of an ASR system.

Risk can be reduced by performing more thorough hydrogeologic investigations during the exploratory well phase of projects. Risk can also be reduced by minimizing investment in surface infrastructure, such as using temporary equipment, until it is demonstrated that an ASR system will likely be successful. Regrettably, the opposite course of action is very often taken, in which there is a rush to immediately design and construct the final wellhead and surface infrastructure including post-treatment facilities. If the ASR system subsequently fails, then the money invested in the wellhead and surface infrastructure is needlessly lost.

Recovery Efficiencies Are Often Overestimated

The recovery efficiency is of paramount importance for ASR systems because it is a storage technology. Water is injected into an aquifer with the expressed purpose of being able to recover that water at a later date. Achievement of high recovery efficiencies is an obvious goal of ASR systems with the perception that non-recoverable water is wasted. However, expectations as to recovery efficiencies should be realistic. The potential performance of chemically bounded ASR systems, particularly systems that store freshwater in brackish-water aquifers, has been marketed as approaching 100%. However, the actual historic operational experience indicates that a recovery efficiency of close to 100% will not occur unless the groundwater in the storage zone only marginally exceeds applicable groundwater standards. The mixing and migration of stored water will inevitably result in some of the stored water becoming unrecoverable.

An important, but often overlooked, study concerning the performance of ASR systems is the Bear and Jacob (1965) analytical evaluation of the movement of water injected into an aquifer with a uniform flow field. The key observation is that a water divide will occur between injected water and native water of the aquifer. For a given injection rate and ambient groundwater flow velocity, injected water can penetrate an unlimited distance in the down-gradient direction, but only a limited distance upgradient and laterally (cross gradient). Similarly, a water divide exists during pumping, beyond which water can never be recovered. Water can be captured in only a limited distance in the down-gradient direction. Only water that is in the part of the aquifer contained within both the injection and pumping water divides is recoverable. Density stratification will also limit recovery efficiency as denser, more saline water flows towards the ASR well near the base of the storage zone. There is thus an intrinsic limit on both recovery efficiency and the size of the “reservoir” or storage capacity of a single ASR well.

However, if the native water in the storage zone only marginally exceeds water quality standards for its intended use, then a considerable amount of mixing of the injected and native water can occur before water quality thresholds are exceeded. This allows for high recovery efficiencies, sometimes greater than 100%. The storage zone is being used, in essence, as a blending tank. However, for ASR systems storage using brackish-water aquifers containing greater than 1,000 mg/L of chloride, a more realistic target operational recovery efficiency is 70%. Even lower recovery efficiencies may be acceptable depending upon the value of the recovered water when needed compared to the lower cost of water when injected. The setting of unrealistically high expectations as to recovery efficiency can create the impression that an ASR system is underperforming or failing when it inevitably does not meet the expectations due to inherent hydrogeologic limitations. The success criteria for an ASR system should not be a fixed recovery efficiency value, but whether or not the system provides water when needed at an acceptable quality and at a lesser cost than other storage or supply options.

Well Rehabilitation is a Normal Part of ASR System Operations

Production wells require periodic rehabilitation to maintain specific capacity, although the frequency varies greatly between wells. ASR and other injection wells are more prone to loss of well performance because the flow of water is into the well and formation. The injected water may contain entrained suspended solids and is usually not in chemical equilibrium with aquifer rock and native water. The rehabilitation of ASR wells often involves periodic backflushing and less frequent major rehabilitation. Depending upon system-specific conditions, backflushing may have to be performed daily, weekly, or at less frequent intervals. Major well rehabilitation may involve acidification, high-level (shock) disinfection, jetting, swabbing, and/or sonic or liquid carbon dioxide treatment. The need for well rehabilitation should be considered a normal part of operations and maintenance and be incorporated into the initial system design and economic analysis. Wellheads should be designed to facilitate well rehabilitation activities. An adaptive management approach should be taken with respect to well rehabilitation as there is no one best method.

CONCLUSIONS

The main lesson learned from ASR systems that have failed to meet expectations is that ASR systems are hydrogeologically complex, and a more sophisticated approach needs to be taken to proactively identify problems early in projects, before the costs of constructing a full system are incurred. A risk element is associated with ASR projects that is not present in most other utility infrastructure projects and should be incorporated into cost-benefit analyses. However, the water resources and economic benefits of ASR are still compelling despite the risk element. ASR has a very large cost advantage compared to other potential options (e.g., tanks and surface reservoirs) for the storage of large volumes of water. Risk can be reduced, but never completely eliminated, by heeding the lessons from past ASR failures and successes.

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Determination of Infiltration Coefficient in Karstic Formations

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Abstract

Water balance calculations are the base of aquifer management especially in karstic aquifer of arid regions that have the main role in preparation of drink water demand. Estimation of the balance components of karstic aquifers is more difficult and sometimes impossible. So in karst researches many different amounts have been mentioned for the infiltration coefficient of karstic formations based on personal judgment. This parameter can be evaluated if all water balance components establish in a karstic watershed that infiltration coefficient will be the only unknown parameter. In this method deterministic parameters like precipitation and discharge were estimated and infiltration coefficient was calculated as an unknown parameter that can be extended to similar watershed.

In this research a comprehensive investigation carried out on three main anticlines of carbonatic formations in Zagros geological zone (southwest of Iran). Discharge rate of 35 karstic springs and hydrochemical properties of 10 representative springs were monitored monthly for one year. The results of this monitoring used for determination of catchment area for each spring and evaluation of hydrogeologic characteristic of the aquifers. According to the field survey and combination of geology, geomorphology, topography... maps by means of GIS software, the anticlines were divided into 3 categories (A, B and C) and the infiltration coefficient of them evaluated 90-95, 70-75 and 50-55 percent, respectively.

Keywords

Infiltration coefficient, Karstic aquifer, Water balance, carbonate formation, Hydrochemical properties

Introduction

The most important and complicated part of Groundwater Balance- equation is an estimation of effective precipitation which is penetrated to an aquifer. Particularly in karstic aquifer the estimation is more difficult because of heterogeneity, complex topography and nonuniformity of slop. Several studies on different areas show that the range of infiltration rate in karstic rock is variable and depends on climatology and lithology of rocks and it is essential to calibrate this rate for each area.

Method

The study area is included three anticlines which located between cold-dry region (Bakhtiyari) and warm-wet region (Khouzestan) and it is possible and logical to hypothesize these anticlines as the representative of the south-west part of Zagros due to extensiveness and varied geomorphologic conditions.

Minimum and maximum elevations of these anticlines are about 800 and 3600 meter from sea level, respectively. They are surrounded by Karoon River on the North side and Izzeh polje, Miyangaran Lake and marly impermeable formation on the South side. These anticlines are formed by carbonate rock (Sarvak, Asmari, Fahliyan-Dariyan formation) which separated by impermeable formations. Figure 1 demonstrates stratigraphy and geological condition of these anticlines.

Although the precise measurement of infiltration rate is impossible in lime stone formations, it is conceivable to estimate infiltration rate in lime stone formations in case of other parameters in groundwater balance calculation are definite. It should be mentioned this estimation is inexact because of heterogeneity in limestone and complicated geomorphic karst. But this inexact estimation could improve antithesis in this case.

A comprehensive investigation carried out on the three main anticlines (Kamestan, Piyon, Mongasht) which are spread in three provinces of Iran (Khouzestan, Chaharmahal, Kohkooliyeh).

These anticlines have been chosen for study based on following reasons:

In these anticlines two main formations of Zagros (Sarvak and Fahliyan-Dariyan formations) are exposed.

2- Impermeable thick layers prevent hydraulic connection from Asmari formation to Sarvak and Fahliyan-Dariyan formations.

3- These anticlines have vast extent, therefore they located in variable weather and climate, tectonic and geomorphology conditions.

4- There is not any hydraulic connection between limestone (anticline) and alluvial (plain). Geological profile confirms these anticlines are a single hydraulic unit.

5- None existence adjacent widespread plain reduces the error of outflow or inflow from alluvial plains in estimating groundwater balance.

On the north of study area, Kamestan Anticline is divided in two parts by Karoon River. The result of hydro chemical analysis, geological profile and springs situation confirm that these two parts are separated from each other and field study shows the west part of this anticline is not discharged to Karoon River.

The result of hydro chemical analysis especially Ca/Mg ratio is an evidence to prove non-existence hydraulic relation between the west part and the east part of Kamestan Anticline.

Three large springs are located on the south plunge of this anticline(Baghe shirin, Fabareh, Hoze agha Daniyal) which Sarvak formation's area is not enough to supply water discharge of these springs. Therefore it is possible that the west or east part of anticline supplies the water discharge of these springs.

For more investigation, the average of Ca/Mg ratio in water samples of these springs is calculated. This ratio is less than 3 whereas the same parameter at Sarvak formation's is more than 3 because of purity of limestone in this formation. As a result, it is probable that Asmari formation on the east part of Kamestan Anticline is the source of water supply of these springs.

As table 1 demonstrates, the sources of all springs with more than 10 l/s discharge is Sarvak and Fahliyan-Dariyan formations. Total discharge of study area is about 20336 l/s which is a combination of penetrating water in Sarvak and Fahliyan –Dariyan formations.

In this study the infiltration rate have been estimated based on following steps:

Initially, elevation-precipitation relation is evaluated based on the meteorology stations' data, after that the amount of total annual precipitation in study area are calculated with GIS tools.

The whole water resources on study area are identified and the average of spring's discharge is calculated.

All anticlines on study area are divided to several parts based on the satellite images, air photos, field studies and investigating impassable area by helicopter. These parts are introduced below:

A: Infiltration rate in this area is high because of Closed and low angle of slope basin, with high karst development area and plenty of sinkholes and snow downfall.

B: In this area, there are plenty of open grikes and joints because of fault operation.

C: This area has the least joints and grikes without any sinkholes and closed basin with high angle of slope in hillside therefore infiltration rate is low in contrast with A.

The average of infiltration rate for whole study area is calculated based on the average of springs' discharge and ground water balance equations.

The calculated average infiltration rate is modified for each area with GIS tools.

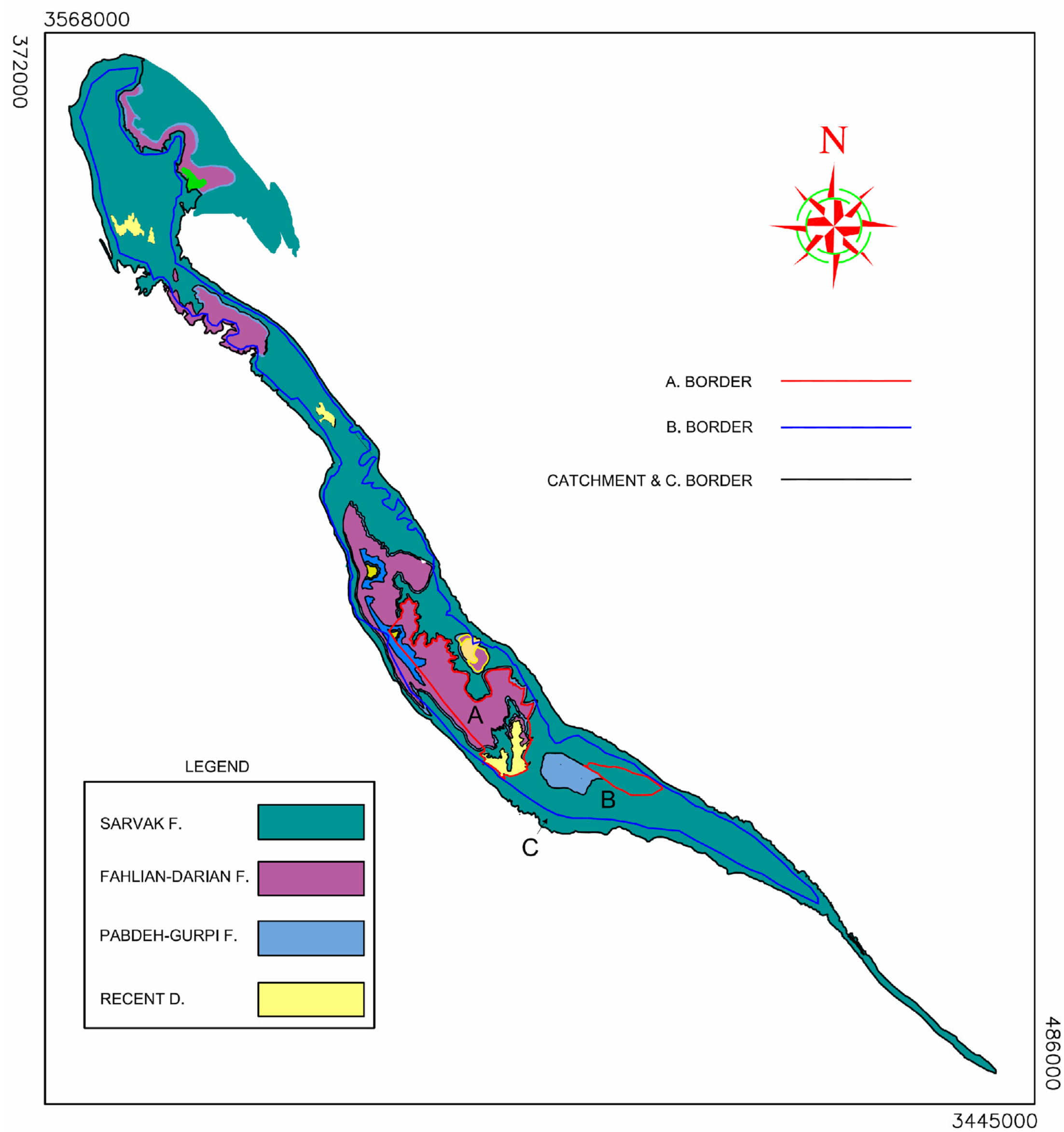

Fig 1: geological formations and area division to 3 parts

Table 1: Main springs characteristics in study area

No.	Spring Name	UTM		Average discharge *(lit/sec)*	Anticline Name	Formation Name
		X	Y			
1	Avajdan 1	388002	3547323	228	Piyon	Sarvak
2	Avajdan 2	388231	3547487			
3	Sarab	388363	3542063	420	Piyon	Sarvak
4	Gardan tagh	382236	3547371	17	Piyon	Sarvak
5	Torshak	389658	354351	50	Piyon	Sarvak
6	Shemi	378680	3543591	1095	Piyon	Sarvak
7	Siyahchal	389025	3542111	2125	Piyon	Sarvak
8	Absefid	430450	3490141	2054	Mongasht	Sarvak
9	Chalab	427373	3493640	1464	Mongasht	Sarvak
10	Roudkalou	422809	3490980	2935	Mongasht	Fahliyan-Dariyan
11	Mal agha	409517	3498799	2429	Mongasht	Sarvak
12	Shivand	420706	3496151	1613	Mongasht	Fahliyan-Dariyan
13	Lirab	446082	3478363	563	Mongasht	Sarvak
14	Gandomkar	437914	3478794	2740	Mongasht	Sarvak
15	Tang reis	489172	3440668	416	Mongasht	Sarvak
16	Dam tang kalou	426935	3497033	366	Mongasht	Sarvak
17	Simloun	426065	3495161	337	Mongasht	Sarvak
18	Tokhmdan	425612	3494877	237	Mongasht	Sarvak
19	Emamzade bala	427340	3497359	51	Mongasht	Sarvak
20	Emamzade payin	427334	3497412	56	Mongasht	Sarvak
21	Tang mourd	411161	3497563	337	Mongasht	Sarvak
22	Tihi	411572	3493396	225	Mongasht	Sarvak
23	Nidon	408803	3497015	146	Mongasht	Sarvak
24	Touf Spid	415635	3492542	22	Mongasht	Sarvak
25	Liton	470020	3457591	338	Mongasht	Sarvak
26	Damaneh jonoub	426993	3449615	11	Mongasht	Sarvak
27	Siyahchal	410290	3497023	61	Mongasht	Sarvak

Result and Discussion

There is not any groundwater out flow in study area by river, aqueduct, pumpage wells and also evapotranspiration because of high depth of the ground water surface. Only outflow way belongs to springs.

In addition there is not any hydraulic connection between river and Sarvak and Fahliyan-Dariyan formations due to impermeable formations existance and the height of springs.

Therefore the water balance equation at the study area is:

Outflow from springs= Area × Infiltration rate × Annual average precipitation

The total area of Sarvak and Fahliyan-Darian formations in study area is about 1060km^2 , the total outflow in the way of springs is nearby 20.34 m^3/sec , the total of average precipitation is about 850mm, yearly and the average of infiltration ratio is estimated 71 percent.

After dividing whole study area to A, B and C parts:

Area of part A: 120.96 km^2

Area of part B: 666.12 km^2

Area of part C: 272.65 km^2

And average annual precipitation estimation at each part:

Annual average precipitation at part A: 910 mm

Annual average precipitation at part B: 840 mm

Annual average precipitation at part C: 790 mm

As a result, based on the amount of out flow, precipitation, the area of each part and karst geomorphic conditions of each area with GIS tools infiltration rate (see below) is calculated for each part.

Infiltration rate in part A: 90-95 percent

Infiltration rate in part B: 70-75 percent

Infiltration rate in part C: 50-55 percent

Conclusions:

In this research, comprehensive investigation on three main anticlines shows that the infiltration coefficient in high developed karst area could reach up to 95 percent and at least the half of precipitation penetrates to karstic aquifer.

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MODELING OF THE WATER AND NITROGEN TRANSFERS IN UNSATURATED–SATURATED POROUS MEDIUM

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Water flow and nitrogen transport in a porous unsaturated-saturated environment constitute a very difficult problem to study because of the nature of the equations governing the transfers between the saturated zone and the unsaturated zone. The first difficulty comes owing to the fact that the water-table, which constitutes the lower condition of the unsaturated zone, is not known and depends on the exchanges between the two zones. The second difficulty is the reactive nature of nitrogen particularly in the unsaturated zone. In this study, a mathematical modeling tool was developed to simulate the water flow and nitrogen transfers in unsaturated–saturated porous medium. We used a mathematical formulation which consists in considering as a single continuum the saturated and the unsaturated zones. The model is based on the finite elements method using the Freefem++ code. This code was adapted to the equations used in this study, namely: Richards' equation to study the water flow in the unsaturated zone, the diffusivity equation for the groundwater flow and the transport equation of advection–dispersion type to study the nitrogen transfer. The advantage of the Freefem++ is that it generates self-adapting grids. The modeling tool was validated using experimental data measured on the small-scale physical model. A comparative study with the ADI method was carried out and showed that the use of the finite elements Freefem++ code presents an advantage and provides more accurate results.

Key words: modeling; nitrogen; unsaturated-saturated; self-adapting grids

1. INTRODUCTION

This paper is focused on the mathematical modelling of the problem related to water flow and nitrogen transport in the unsaturated and saturated zones of unconfined aquifers. Water and solute transfers in a porous unsaturated–saturated environment constitute a difficult problem to tackle because of the nature of the equations governing the transfers between the saturated zone and the unsaturated zone. The difficulty of solving the coupling of the vertical flow in the unsaturated zone and the horizontal flow within the groundwater is linked to the fact that the interface between these two fields (i.e. the water table) is not known *a priori*, and that it takes variable positions with time (Lemacha *et al*., 2007).
Study of water and solutes transfer in the unsaturated and saturated zones has quite often been carried out by using distinct mathematical models, which are applied separately in each zone (Dupuy *et al.*, 1997a,b). This kind of approach is based on a technique of artificial coupling connecting the two zones, which often generate numerical disturbances at the unsaturated–saturated interface. Diaw *et al.* (2001) performed a study of one-dimensional water flow through the unsaturated–saturated zone. Khanji (1975) developed a steady flow model through the unsaturated–saturated zone constituted by fine sands. In our approach, we developed a mathematical model based on a single flow equation which can be used for both unsaturated and saturated zones which are regarded as a single continuum. The diffusivity equation will be used in a nonlinear form and in a linearized form. Two numerical resolution methods (finite elements method and finite differences method) are used afterwards to solve the mathematical model and their results compared.
In this study, we will show the interest of modeling various chemical, physical and biological processes that influence the fate of nitrogen in the agricultural soils for evaluating the groundwater contamination by nitrates. The mathematical model presented in this paper is a deterministic-mechanistic simulation model which uses conceptual representations with a limited number of parameters. Soil vadose zone characterization is made worldwide since it is based on a physical approach (Saadi, 2003; Ibnoussina et al., 2006). The simulation model allows simplified calculations of water and nitrogen uptake by plant roots in water flow and nitrogen transport equations. The nitrogen transformations in the soil were simplified using parameters that can be calculated from easily measured chemical soil properties or obtained from the literature (Saâdi et al., 2003).

2. STUDY DOMAIN

Figure 1 shows the geometry of the study domain. The limits of the flow domain are:

- A horizontal upper surface (ground surface); to part of this surface an initial flow q_0 is applied, causing the infiltration. The infiltration strip can simulate either an irrigation canal, or an artificial recharge reservoir.
- Two tanks located at the same distance from the axis of the infiltration strip in which a water level is imposed, limit the groundwater. Each tank can simulate a drainage ditch.

The problem is thus a plan problem. We consider the system of axes XOZ, axis OX being assimilated with the ground surface, axis OZ being oriented positively towards the bottom and the origin O being in the centre of the ground section of width 2*L*. The problem is thus symmetrical compared to OZ. The transfer problem will accordingly be developed considering only one half of the domain.
On the basis of a horizontal unconfined groundwater with a thickness *E* (m) above an impermeable bottom and a depth $e_{0,}$ a constant infiltration flow q_0 (m/s) is applied to the ground surface on a strip of $2L_0$ (m) width and infinite length. It is supposed, moreover, that the infiltration strip is centred between the two parallel tanks distant of 2*L*, penetrating down to the impermeable layer. A constant piezometric level is maintained in these tanks at a constant depth.

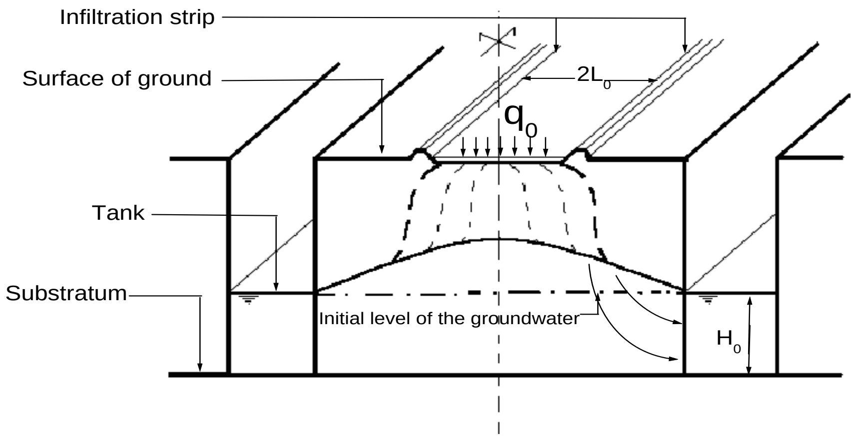

Fig. 1 Schematization of the study domain.

3. MATHEMATICAL MODEL

3.1 Water flow

The flow equation in the unsaturated zone is obtained by coupling the generalized form of Darcy's law which expresses the proportionality law of the flows to the continuity equation (Hillel, 1980). It is assumed that both unsaturated and saturated zones form a medium which can be regarded as only one continuum. The equation used for the two zones has the form of the Richards equation:

$$C\,\frac{\partial h}{\partial t} = div\,\left[Kgrad\quad(h - z)\right] \tag{1}$$

where *C* and *K* are two functions depending on the nature of the medium, *h* is the water pressure head (m), *z* is the vertical coordinate (m).

In the unsaturated zone, *C* = *C*(*h*) is the capillary capacity (m^{-1}) and *K* = *K*(*h*) is the unsaturated hydraulic conductivity (m/s). In this case, the flow equation is the classical Richards equation (Richards, 1931):

$$C(h)\frac{\partial h}{\partial t} = div\,\left[K(h)grad\;\;(h - z)\right] \tag{2}$$

In the saturated zone, $C = S_S$ is the specific storage coefficient (L^{-1}) and $K = K_s$ is the saturated hydraulic conductivity (m/s). One obtains the diffusivity equation governing the unsteady flow in a saturated medium. This diffusivity equation has a linear form:

$$S\frac{\partial h}{\partial t} = \left(\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2}\right) \qquad \text{(Where } S = \frac{S_S}{K_s}\text{)} \tag{3}$$

When the nonlinear diffusivity equation is assembled with the initial and boundary conditions of the study domain, the following system of equations is obtained:

$$\begin{cases} C(h)\dfrac{\partial h}{\partial t} = \dfrac{\partial}{\partial x}\left(K(h)\dfrac{\partial h}{\partial x}\right) + \dfrac{\partial}{\partial z}\left(K(h)\left(\dfrac{\partial h}{\partial z} - 1\right)\right) \\ S\dfrac{\partial h}{\partial t} = \left(\dfrac{\partial^2 h}{\partial x^2} + \dfrac{\partial^2 h}{\partial z^2}\right) \\ q_0 = K(h(x,0,t))\left(1 - \dfrac{\partial h}{\partial z}\right) \\ q_0 = 0 \\ q_0 = 0 \\ h(x,z,t) = z - Z_0 \\ q_0 = 0 \\ q_0 = 0 \end{cases} \tag{4}$$

Therefore, the formulated problem can be solved taking into account the boundary conditions imposed at the limits of the domain (Fig. 1) and the condition imposed at the water table, that is
$h = 0$.

In the numerical approximation, we only consider the first equation in the domain, where the functions K and C are such that:

$K=K(h)$ and $C=C(h)$ for $h<0$ (5)

$K=K_s$ and $C= S_S$ for $h>0$. (6)

3.2 Soil nitrogen transport and transformations

The transport of a chemical substance (i) in the unsaturated zone is commonly described by the general convection-dispersion equation:

$$\frac{\partial(\rho_d S_i)}{\partial t} + \frac{\partial(\theta C_i)}{\partial t} = \frac{\partial}{\partial x}\left(\theta D_{ap}\frac{\partial C_i}{\partial x}\right) - \frac{\partial(Q_x C_i)}{\partial x} + \frac{\partial}{\partial z}\left(\theta D_{ap}\frac{\partial C_i}{\partial z}\right) - \frac{\partial(Q_z C_i)}{\partial z} + \sum_{i,j} T_{i,j}(C_i, S_i) \tag{7}$$

where C_i ($M\ L^{-3}$) and S_i ($M\ M^{-1}$) are the concentrations of the substance (i) in liquid and solid phases respectively, ρ_d is the dry bulk density ($M\ L^{-3}$), $Q_{i,j}$ is the removal or supply rate ($M\ L^{-3}\ T^{-1}$) of the substance (i), q is the Darcian water flux ($L\ T^{-1}$) and D_i is the apparent diffusion-dispersion coefficient ($L^2\ T^{-1}$) of the element (i). The later parameter is given by the relation $D_i = \lambda_i|v| + D^*_{0,i}$, where λ_i (L) and $D^*_{0,i}$ ($L^2\ T^{-1}$) are respectively the soil dispersivity and the molecular diffusion of the species (i) in the soil, and $v = q/\theta$ is the pore water velocity ($L\ T^{-1}$). When dealing with nitrogen transport and transformations in an unsaturated soil, the chemical species (i) are the nitrate nitrogen (NO_3^--N) and ammonium nitrogen (NH_4^+-N).

Nitrification and soil organic nitrogen mineralization processes are modeled using first-order rate kinetics as described by Saâdi et al. (2003).

$$\frac{dN_{org}}{dt} = -k_{min}\left(N_{org} - N_{\infty}\right) \tag{8}$$

where N_{org} is the instantaneous soil organic nitrogen (M L^{-2}), N_{∞} the organic nitrogen.

The nitrification process is modeled as stated by:

$$\frac{dC_{NO_3^- - N}}{dt} = k_{nit} C_{NH_4^+ - N} \tag{9}$$

where k_{nit} is the first order rate conditional constant (T^{-1}),

The ammonium nitrogen NH_4^+-N adsorption by negatively charged clay particles and organic colloids in the soil is described by a linear equilibrium isotherm (van Genuchten et al., 1980):

$$S_{NH_4^+ - N} = K_D C_{NH_4^+ - N} \tag{10}$$

where K_D is the partitioning coefficient between soil solid et liquid phases (L^3 M^{-1}). As shown in Saâdi et al. (2003), one can obtain a good approximation for the K_D-value, using the cation exchange capacity (CEC) as the unique input parameter.

3.3 The transfer equation of nitrogen in the aquifer

For the transfer equation of nitrogen the aquifer is characterized by water content at saturation θ_S and saturated hydraulic conductivity K_S .The ions NO_3^- will behave as tracers and will be easily pulled towards the aquifer. Indeed in the unsaturated zone-saturated, the strong adsorption and the fast nitrification of the NH_4^+ ions in NO_3^- ions make their probability of reaching the aquifer very negligible. Considering all these circumstances, the transfer equation of the nitrogen nitrates in the aquifer is given by:

$$(\theta_S + \rho_d K_D)\frac{\partial (C_{NO_3^-})}{\partial t} = \frac{\partial}{\partial x}\left(\theta_S Dap \frac{\partial C_{NO_3^-}}{\partial x}\right) - \frac{\partial (Q_x C_{NO_3^-})}{\partial x} + \frac{\partial}{\partial z}\left(\theta_S Dap \frac{\partial C_{NO_3^-}}{\partial z}\right) - \frac{\partial (Q_z C_{NO_3^-})}{\partial z} + K_{nit}\theta_S C_{NH_4^+} \tag{11}$$

4. NUMERICAL MODELLING – FINITE ELEMENT METHOD

In the field of water flow in porous media, the application of the finite element method is rather recent. The advantage of the "concurrent" method of finite differences lies in a greater simplicity of the numerical schemes which leads to lower computing times. However, the finite elements methods show greater potential when dealing with the approximations of equations in the vicinity of curved borders on the one hand, and the expressions of numerical schemes of higher degree on the other hand, which represents two advantages over the finite difference methods (Raviart, 1981).
To solve the flow and transport equations by the finite element method, the FreeFem++ (Hecht *et al.*, 2003) software has been used. In order to run this software, a numerical code was developed. This code enables us to define the borders of the domain, to set the number of nodes on each border. The finite element grid is then automatically obtained. The spaces and the degrees of the polynomials are defined and the variational formulations of the equations are posed with the initial and boundary

conditions. A time loop and a loop to refine the triangulation at zones sensitive to flow and transport are introduced. Such sensitive zones are found under the infiltration strip and at the level of the capillary fringe.

5. RESULTS AND DISCUSSION

Concerning the numerical model, the infiltration code is adapted to the type of tests which one wants to simulate (infiltration under constant head or with constant flow). All the results obtained with this code (field of pressure head or of water content) can be used to obtain volumes, flows, etc. The pressure head field corresponding to the last step of calculation in the infiltration module is taken again as the initial condition of the recharge module. A treatment unit in this module also makes it possible to obtain the evolution of the water table, the volumes, etc. Thus the numerical model is able to compute, at any time, the pressure head and water content fields and the flows.
To validate the model, experimental data from Khanji (1975) were used. These data relate to fine sand. The parameters of this material and other data used in the model are as follows: X_{max} (length of the study domain) = 300 cm; Z_{max} (depth of the study domain) = 200 cm; q_0 (constant infiltration rate) = 15 cm/h; H_0 (imposed hydraulic head) = 135 cm; K_s (saturated hydraulic conductivity) = 35 cm/h; θ_s (saturated water content) = 0.3 cm^3 cm^{-3}; h_g (scale parameter of water pressure head) =30 cm; ω (effective porosity) = 29 10^{-2}; α (dispersivity) = 10 cm; C_1 (concentration) = 1 mol/L; S = 8.2 10^{-5} .
Taking into account the heterogeneity of the soil in place, there is at certain vertical positions, a slight difference between the values of the water contents, since it is assumed in our model that the soil is homogeneous and is characterized by the relation θ(h) which is the average of the suction curves measured on each vertical.

5.1 Water transfer

Figure 2 shows the temporal evolution of the water table profiles. We note that there is a rise of the water table in time. This rising is due primarily to the recharge of the groundwater from the unsaturated zone. We also note that the water table rise still continues after 4 hours. This reflects the flow regime, which is unsteady due to the fact that flow entering the domain by the infiltration strip is greater than that outgoing through the outlet. This will create a head gradient in the saturated zone between points located beneath the source zone and those close to the fixed head boundary.
Figure 2 indicates a good agreement between the positions of measured and calculated water table profiles during all the recharge periods.
The results obtained by the finite element method (FEM) are closer to the measured values than those obtained by the finite differences method (FDM). This is due primarily to the automatic refinement of the triangular grid at the level of the capillary fringe (Fig. 3) which increases the accuracy and which provides a better simulation of the water table.
The comparison between the numerical and experimental results shows that the response of the numerical model developed here is quite satisfactory. Indeed, the model manages to correctly represent the physical processes, if the initial and boundary conditions are expressed on known and

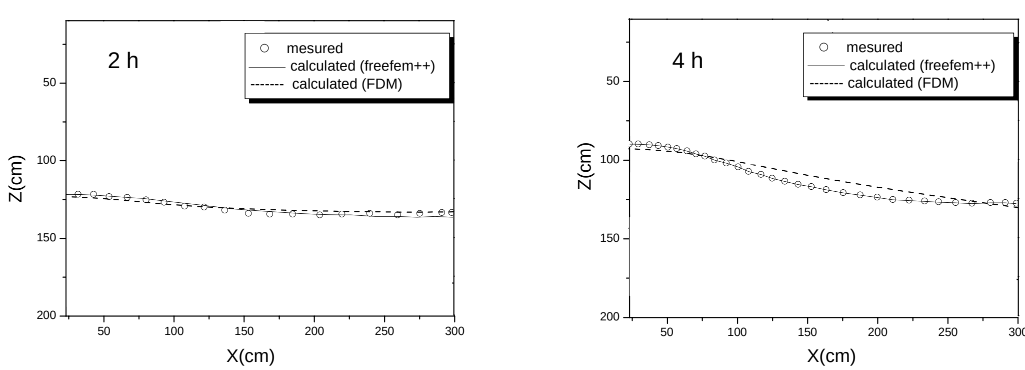

Fig. 2 Comparison between the water-table profiles simulated (finite elements and finite differences) and measured, at times t = 2h and 4h.

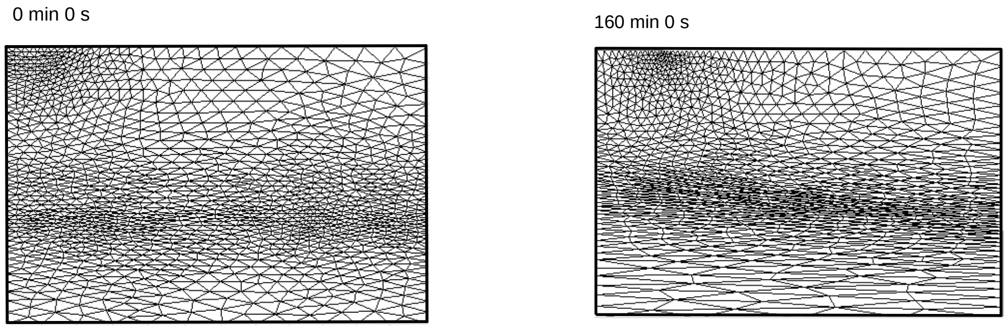

Fig. 3 Self-adapting finite element grid at the level of the wetting front and of the capillary fringe, at the initial state (t = 0) and at time t = 160 min.

fixed in time geometrical limits. The mesh adaptivity is performed by computing the space error indicators, refining the mesh where they are large than the mean value and possibly coarsening the mesh where they are much smaller. Mesh adaptivity based on *a posteriori* error estimation techniques have become an indispensable tool in large-scale scientific computation and much work has been done on finite element discretization of elliptic problems. The main results consist in exhibiting local error indicators which can be computed explicitly as a function of the discrete solution and the data. They are built from the residual of the strong equation and the jumps across the inter-element of the fluxes. Since they are local they provide a good representation of the error distribution, and they are so very efficient tools for mesh adaptivity.

5.2 Nitrogen transport

The study of the solute transport was performed by assuming that water coming from the infiltration strip contains a solute concentration of one mole per litre. It is also assumed that at initial state, the flow domain is characterized by a null concentration. During the recharge phase of the groundwater, a hydraulic gradient is generated between points located beneath the source zone and those close to the fixed head boundary. This involves a water flow towards the groundwater outlet and consequently the movement of the solute.
To study the effect of the self-adapting grid on the quality of the results obtained by the finite element method, Fig. 4 shows the solute concentration contours at various times. The curves illustrating the solute concentration distribution computed without refinement of the grid show some disturbances, whereas those obtained by adaptation of the grid have a very smooth form. This difference is due to the fact that refinement of the grid provides much more accurate results.

6. CONCLUSION

In this paper, we have proposed a mathematical model for the water flow and nitrogen transport in a porous unsaturated–saturated zone. The comparison between the numerical and experimental results shows the positive response of the numerical instationary model developed in this study to better represent the physical phenomena, when the initial and boundary conditions are expressed on known and fixed in time geometric borders. This model allows the simulation of water flow and solute transport in the unsaturated–saturated zone. The validity of the results simulated by this model is checked using experimental laboratory data. Moreover, the comparison between the finite difference scheme and the adaptive finite element approximation used confirms the robustness of the latter method.

The results presented here were obtained at the laboratory scale. It is essential to envisage their extension to the field scale. Passing to the land parcel scale, then to the catchment scale represents the logical extension to this research. Although the mathematical model was designed to allow its application to problems of larger space scales, the main impediment comes from the

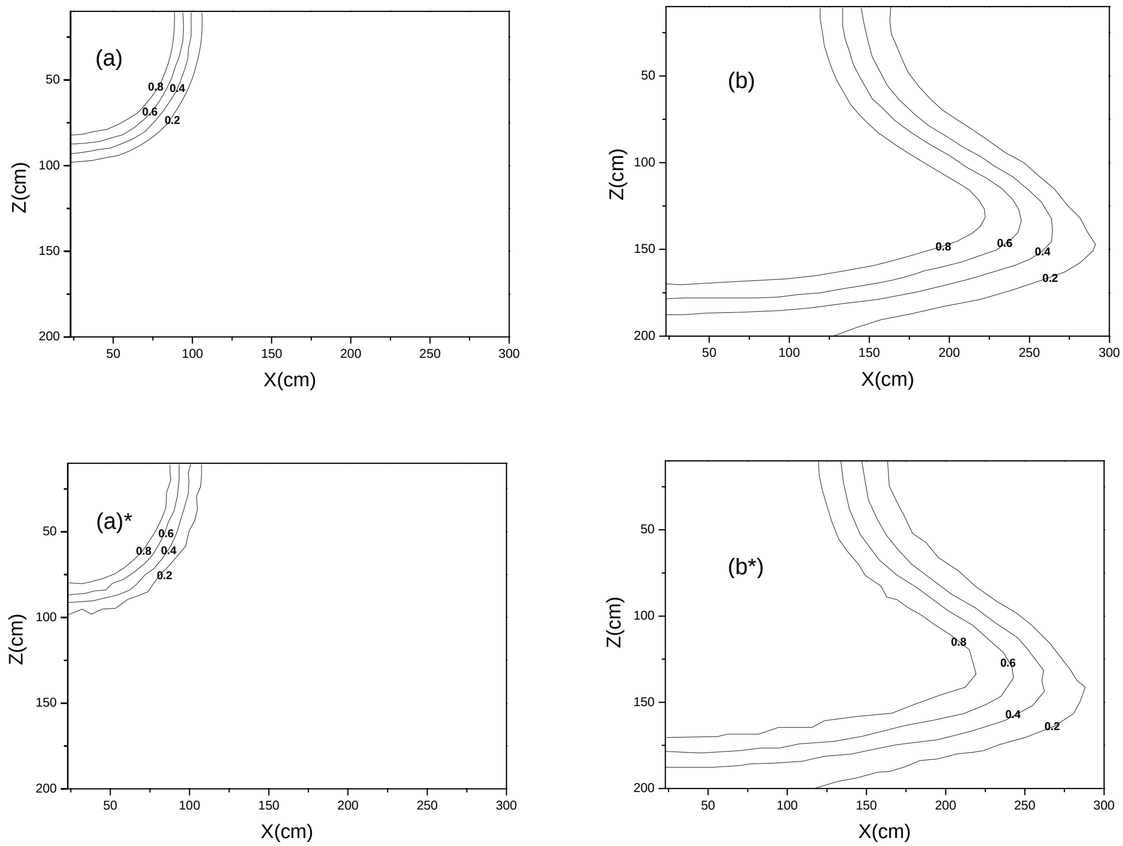

Fig. 4 Distribution in space and time of the solute concentration contours. (a) and (b): with self-adapting grid; (a*) and (b*): without adaptation of the grid.

difficulty in obtaining in the field the soil characteristic relations *K(h)* and $\theta(h)$ and their strong space variability.

Moreover, moving to the field very often implies the presence of significant heterogeneity, the existence of anisotropy and heat gradients, which indisputably restrict, in certain cases, the applications of the results obtained in laboratory. The presence of a more or less dense vegetation cover also modifies the flow transfer and affects the boundary conditions at the soil surface. These various phenomena will be taken into account in coming modelling work, based on their physical analysis.

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DYNAMICS OF SULFATE-REDUCTION IN THE FLORIDAN AQUIFER SYSTEM (FLORIDA, USA) DURING ASR CYCLE TESTING, WITH IMPLICATIONS FOR ARSENIC SEQUESTRATION AT RECLAIMED AND TREATED SURFACE WATER ASR SYSTEMS

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ABSTRACT

Aquifer redox environment is the primary control on the geochemical behavior of arsenic and other trace metals during cycle testing at many aquifer storage and recovery (ASR) systems. The source of trace metals (primarily arsenic and molybdenum) is euhedral pyrite, which occurs as syndepositional iron sulfide in marine limestones; or framboidal pyrite of microbial origin. . Pyrite oxidation occurs when oxic recharge water reacts with the limestone aquifer matrix. The subsequent fate of dissolved arsenic – whether it is stable in solution or as a secondary solid phase – has significant regulatory implications for ASR. Therefore, defining the geochemical conditions that favor arsenic sequestration in a stable solid phase, while maintaining aquifer permeability, are critical to successful ASR implementation. A cycle testing strategy that limits pyrite oxidation by simulating native Floridan Aquifer conditions is one such approach.

Most ASR systems in south and southwest Florida store water in permeable zones of the Floridan Aquifer System, which occur in Tertiary-age interlayered marine limestones and clastic deposits at depths greater than 100 meters. As groundwater travels from the central Florida recharge area, native water quality evolves to show increased total dissolved solids concentration and sulfate-reducing redox conditions. The Kissimmee River ASR pilot site (and also reclaimed water ASR systems) have recharge water-quality characteristics that enable the Floridan Aquifer to re-establish native sulfate-reducing redox conditions, favor iron sulfide as a stable solid, thus limiting arsenic transport in the aquifer. Optimum water quality characteristics for arsenic sequestration are defined using terminal electron acceptor species (ferric iron, sulfate, organic carbon). In addition, real-time monitoring of storage zone geochemical conditions (pH, ORP, temperature, specific conductance) using an oceanographic probe shows that the return of native redox conditions is rapid (1-2 days), implying rapid rates for sulfate-reduction.

KEYWORDS: redox geochemistry, sulfate reduction, cycle testing

INTRODUCTION

Mobilization of arsenic and other trace metals above regulatory criteria during aquifer storage recovery (ASR) cycle testing remains as a significant challenge for ASR implementation in Florida and elsewhere in the United States. Florida and federal regulations, and World Health Organization guidelines (FAC 62-550, Florida Safe Drinking Water Act; PL 93-523, Safe Drinking Water Act, (1974, and 1996 amendments); WHO, 2008) state that arsenic concentrations above 10 μg/L in the aquifer (the point of compliance) at any time during a cycle test are an endangerment to the resource and public health. Strict interpretation of these regulations has stalled ASR implementation in Florida, where application of these technologies could be most beneficial. Arsenic mobilization can be controlled by several methods: 1) deoxygenation of recharge water, 2) production well modification methods; and 3) operational strategies during cycle testing.

Feasibility testing of deoxygenation technologies is underway at several pilot sites in Florida, using either membrane systems or addition of chemical reductants to induce degassing of dissolved oxygen prior to recharge. The results of deoxygenation pilot tests likely will enable ASR systems to operate in complete regulatory compliance with regard to arsenic concentrations, because recharge water showing negative oxidation-reduction potentials (ORPs) will not oxidize pyrite in the aquifer matrix. Without pyrite oxidation, component trace metals are stable in the solid phase. However, the cost-effectiveness of deoxygenation for larger ASR systems has not been defined. Modification of existing wells for arsenic attenuation has been investigated by Halford et al. (2010), who demonstrated lower arsenic concentrations by selective grouting of arsenic-rich permeable zones in production wells. This approach has not been attempted yet in Floridan Aquifer wells, but could merit further investigation. Finally, optimizing cycle testing strategies for arsenic minimization may be a cost-effective means toward regulatory compliance at ASR systems, particularly if compliance could occur over time. Analysis of 5 successive cycles at the Tampa Rome Avenue Park potable water ASR system (CH2MHill, 2007; Mirecki, 2008;) showed that consistent, large volume (3,785 megaliters, ML) cycles showed a consistent, reproducible decline in arsenic concentrations with each of 5 successive cycles The likely mechanism for this declining pattern is source removal. Recharge with hyper-oxygenated (12 to 20 mg/L dissolved oxygen), ozone-disinfected water oxidized and removed the pyrite from permeable zones.

This paper demonstrates that geochemical reactions in the Floridan Aquifer during ASR cycle testing naturally and quickly tend toward conditions that favor sequestration of mobilized arsenic back into a solid sulfide phase That is, after recharge is complete, the aquifer quickly reverts to the native redox environment in which solid iron sulfide minerals are stable. At the Kissimmee River ASR pilot site (Mirecki and Verrastro, this volume), filtered, UV-disinfected Kissimmee River surface water is recharged into upper permeable zones of the Floridan Aquifer. Kissimmee River surface water characteristics include fairly high concentrations of total organic carbon, iron, dissolved oxygen, and positive values of ORP. These constituents are recharged into a sulfate-reducing aquifer, characterized by the presence of dissolved sulfide, sulfate, and ORP values more negative than – 200 millivolts (mV). In-situ measurements in the primary flow zone of the upper FAS show that aquifer redox conditions evolve quickly from suboxic (recharge-dominated) to reducing (native), within one day after recharge is complete. The guiding hypothesis of this investigation is that sulfate, iron, and dissolved organic carbon in recharge water stimulates native sulfate- and/or iron reducing bacteria in the aquifer. Colloidal (ferric) iron in recharge water reduces as the aquifer redox environment shifts from suboxic to sulfate-reducing, providing a ferrous iron source for subsequent iron sulfide precipitation. A simultaneous declining pattern of dissolved arsenic concentration throughout the wellfield during storage and recovery supports this hypothesis.

CYCLE TESTING AT KISSIMMEE RIVER ASR PILOT SYSTEM

Surface water from the Kissimmee River is recharged, stored, and recovered from permeable zones in the Ocala Limestone, which encompasses the upper zones of the Floridan Aquifer System (FAS). The storage zone occurs at depths between 172 and 268 m (565 and 880 ft) below land surface. Cycle 1 was conducted primarily to ensure consistent operation of all facility components, including a pressure media filtration to remove particulates, ultraviolet (UV) disinfection for coliform attenuation, and 300 horsepower single-drive pump delivering 19 megaliters/day (5 million gallons/day). Cycle 1 consisted of one month of recharge (total volume recharged was 486 megaliters or 128.5 million gallons) followed by one month of storage (no pumping), then recovery to native aquifer conditions (approximately 6 weeks, 143 percent of the recharge volume was recovered). Cycle 2 consisted of 4 months recharge (total volume recharged was 1278 megaliters or 373.5 million gallons) followed by 2.25 months of storage (no pumping), then recovery to native aquifer conditions (9 weeks, 98.3 percent of the recharge volume was recovered). Progress of each cycle is evaluated by sampling and monitoring at four storage zone monitor wells which have the same open interval as the ASR well. These monitor wells are located at 112 m east (MW-10), 381 m east (OKF-100 dual zone), 716 m southwest (MW-18), and 1280 m north (MW-19) of the ASR well. Geochemical data and interpretations presented here are based on data from the storage zone monitor well MW-10 only, because this well is instrumented to enable in-situ water-quality measurements in the storage zone, along with water-quality analyses from samples collected at the wellhead.

GEOCHEMICAL DATA COLLECTION PROGRAM AT THE KISSIMMEE RIVER ASR PILOT SYSTEM

The primary objective of the data collection effort during cycle testing is to demonstrate feasibility and regulatory compliance with particular attention to water-quality changes and percent recovery. Groundwater samples were collected weekly through cycle 1 at the ASR and storage zone monitor wells. Analytes included field parameters (pH, ORP, temperature, specific conductance, dissolved oxygen), major and trace dissolved inorganic constituents, total organic carbon, stable isotopes ($\delta^{18}O$, δD, $\delta^{13}C$, $\delta^{34}S$ of dissolved sulfide and dissolved sulfate), and selected radionuclides (gross alpha, ^{226}Ra + ^{228}Ra).

Selected parameters also were monitored at depth in storage zone monitor well MW-10, using a factory-calibrated multi-parameter probe (SEACAT Profiler SBE 19plusV2, Sea-Bird Electronics, Bellevue Washington), which was programmed to obtain hourly measurements of temperature, pH, ORP, dissolved oxygen, and pressure. The multi-parameter probe is suspended in the well bore, at the depth of the major producing zone (183 m) within the upper FAS. Interpretation of weekly water-quality data obtained at the wellhead plus simultaneous continuous monitoring at the storage zone enables a detailed characterization of aquifer redox environment throughout the cycle. Interpreting this data set has significant implications for the transport and fate of arsenic in the evolving FAS redox environment.

REDOX EVOLUTION OF THE AQUIFER STORAGE ZONE DURING CYCLE TESTING

Native Floridan Aquifer System Characteristics

The native redox environment of the upper permeable zones in the FAS are dominated by sulfate-reducing conditions (Rye et al., 1981; Sprinkle, 1989; Wicks and Herman, 1994; McMahon and Chapelle, 2008). Sulfate-reducing redox conditions favor pyrite as a stable solid phase, and arsenic is not mobilized (Jones and Pichler, 2007). The local (Okeechobee and Glades Counties; Mirecki, unpublished data) native FAS redox environment also is characterized by sulfate-reducing conditions. Native FAS water quality is characterized by slightly alkaline pH, low iron and nitrate concentrations, with the sulfate-total sulfide redox couple showing greatest mass (Table 1). Arsenic concentrations in the native FAS are always below the method detection limit of 1 μg/L.

Statistic	Iron, in ug/L	Sulfate, in mg/L	Sulfide, in mg/L	ORP, in MV	Nitrate, in mg/L	pH
Mean	199	411	2.6	-193	0.06	7.73
Std Dev	170	478	1.8	127	0.03	0.33
N	9	12	3	2	4	12

Table 1. Selected native Floridan Aquifer System water-quality constituents from wells in Okeechobee and Glades Counties, FL. Kissimmee River ASR Pilot site is located on the Okeechobee-Glades County line. N is number of samples.

Redox Evolution During ASR Cycle Testing

Cycle testing introduces water into the aquifer having high concentrations of iron (> 200 μg/L), dissolved organic carbon (15 mg/L), dissolved oxygen (4 to 8 mg/L), and oxic redox conditions (+264 +/- 49 mV; Mirecki and Verrastro, this volume). Introduction of recharge water at these fairly high pumping rates (18 megaliters/day or 4.8 million gallons per day) disequilibrates the aquifer redox environment temporarily. During recharge, dissolved oxygen concentration dissipates along storage interval flow zones, but can still be detected in well MW-10 located 112 m from the recharge. Oxic to suboxic redox conditions are indicated at well MW-10 during Cycle 1 recharge, as indicated by dissolved oxygen concentrations (1-2 mg/L; Figure 1) and ORP values (greater than +200 MV; Figure 2). Colloidal ferric iron (complexed to dissolved organic carbon) is transported in the aquifer as recharge continues. Arsenic is released during recharge by oxidation of pyrite in the aquifer matrix by dissolved oxygen, consistent with the proposed mechanisms (Moses and Herman, 1991; Jones and Pichler, 2007). Similar trends were observed in Australian reclaimed water ASR systems in carbonate aquifers (for example, Vanderzalm et al., 2010).

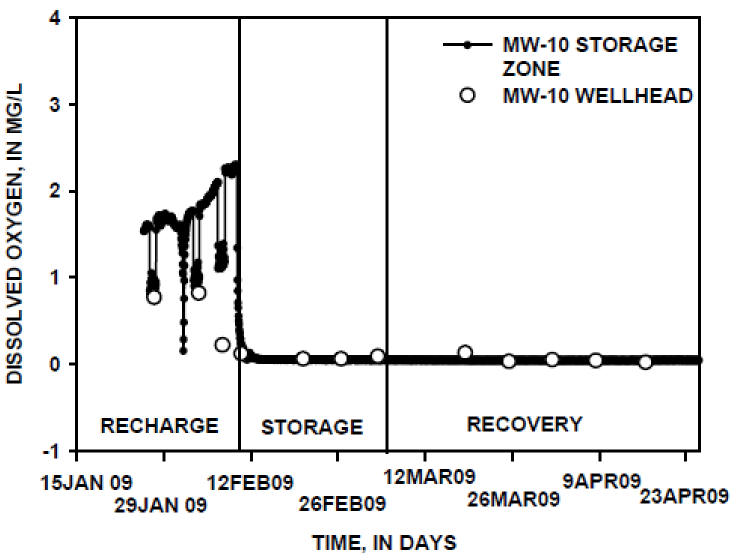

FIGURE 1. Dissolved oxygen concentrations measured during Cycle 1 in well MW-10 (112 m from the ASR well). Continuous measurements were obtained in the storage zone using the SeaCat multi-parameter probe; discrete measurements were analyzed weekly in wellhead samples.

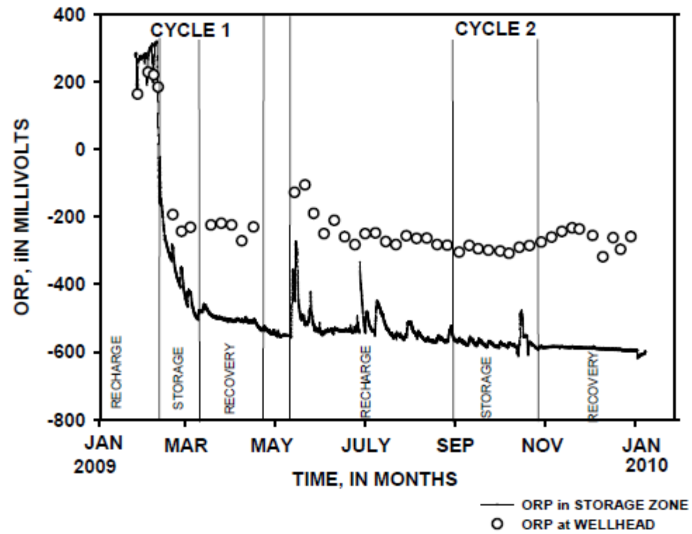

FIGURE 2. ORP measurements during Cycles 1 and 2 in well MW-10 (112 m from the ASR well). Continuous measurements were obtained in the storage zone were obtained using the SeaCat multi-parameter probe; discrete measurements were analyzed weekly at the wellhead.

After recharge ends, the storage zone redox environment quickly evolves back to conditions that approximate native conditions. Dissolved oxygen is depleted at the MW-10 location within 48 hours of pumping cessation during Cycle 1 (Figure 1). ORP values drop simultaneously, in both storage zone and at wellhead samples (Figure 2). The difference between wellhead and storage zone ORP values may be related to probe performance or a pressure effect on dissolved total sulfide equilibria. However, return to sulfate-reducing conditions requires more than just depletion of dissolved oxygen. Native sulfate-reducing microbes in the FAS apparently can tolerate temporary suboxic conditions. During storage and recovery, microbes couple sulfate reduction with organic matter oxidation, resulting in increased total dissolved sulfide and decreased dissolved organic carbon concentrations. In addition, under sulfate-reducing conditions, ferric iron will reduce to ferrous iron. Chapelle et al. (2009) propose using the ratio of ferrous iron to total dissolved sulfide (Fe^{2+}/H_2S) to distinguish sulfate- versus iron-reducing controls. Increasing total dissolved sulfide and decreasing ferrous iron suggest dominance by sulfate- (versus iron-) reducing microbes, and concomitant precipitation of amorphous iron sulfide. If amorphous iron sulfide is stable in the aquifer redox environment during cycle testing, then it is likely that arsenic will be incorporated into that precipitating solid phase. Therefore, recharge of surface water with colloidal iron and dissolved organic carbon can stimulate native microbes to attain redox conditions that favor arsenic sequestration in amorphous iron sulfides, because iron, organic carbon, and sulfide concentrations are sufficient. A geochemical model describing iron sulfide stability is shown in Mirecki and Verrastro (this volume).

Evolution in aquifer redox conditions through Cycles 1 and 2 was interpreted using the redox indicators defined by Chapelle et al., (2009; Figure 3). The ratio Fe^{2+}/H_2S declines as redox environment evolves from suboxic to sulfate-reducing conditions through storage and recovery of each cycle. Simultaneously, arsenic concentrations decline, as arsenic is incorporated into a stable amorphous iron sulfide, attaining the compliance criterion during storage and recovery (Figure 4).

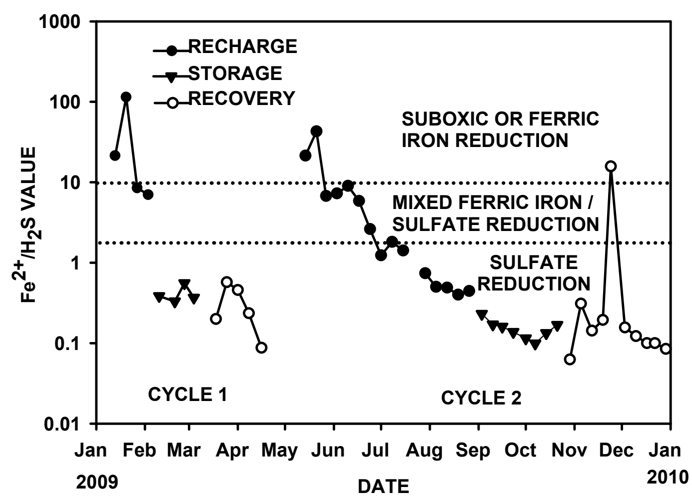

FIGURE 3. Evolution of redox environment in the FAS during Cycles 1 and 2 at MW-10, Kissimmee River ASR pilot system. Redox zones are defined following Chapelle et al. (2009).

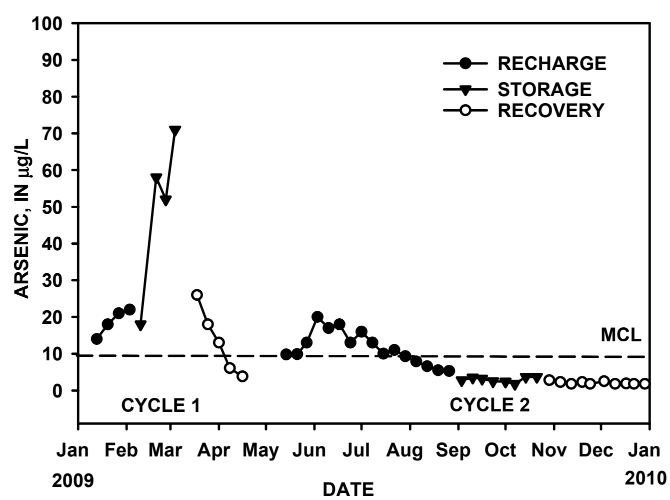

FIGURE 4. Arsenic concentrations in the FAS during Cycles 1 and 2 at MW-10, Kissimmee River ASR pilot system. The maxiumum contaminant level (MCL) is 10 µg/L.

CONCLUSIONS

A detailed characterization of FAS redox evolution during ASR cycle testing was developed at the Kissimmee River ASR pilot system during Cycles 1 and 2. Simultaneous measurements of redox parameters at the MW-10 wellhead and in the aquifer, supplemented with iron, sulfate, sulfide, dissolved organic carbon and other water-quality constituents in time series through each cycle shows arsenic sequestration that results during microbe-mediated iron sulfide precipitation. Interpretation of time-series data show progressive evolution of the redox environment from suboxic, to mixed iron- and sulfate-reducing, to fully sulfate-reducing conditions 112 m (350 ft) away from the ASR well. Simultaneous measurement of arsenic concentrations show consistent decline in the aquifer so that ground water comes into regulatory compliance prior to recovery, when water is distributed back to the environment.

ACKNOWLEDGEMENTS

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Identification of Geochemical Controls on Sediment Reactivity and Buffering Processes During Managed Aquifer Recharge in a Heterogeneous Aquifer: Laboratory Experiments and Inverse Kinetic Reaction Modelling

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ABSTRACT

Driven by the proposed injection of large quantities of aerobic, reverse osmosis treated recycled water into a deep anoxic heterogeneous pyritic aquifer, this study was undertaken to quantify the reactivity of aquifer sediments with respect to oxidant consumption and to characterize the variability of the reaction rates across different lithological units. Laboratory experiments were used to study the geochemical controls on sediment reactivity and buffering processes during the injection of oxygen (O_2) saturated waters into a deep anoxic heterogeneous siliclastic sedimentary aquifer. Detailed geochemical characterisation and sediment incubation experiments identified pyrite (20 – 100%), sedimentary organic matter (SOM; 3 – 56%), siderite (3 – 28%) and Fe(II)-aluminosilicates (8 – 55%) as the main O_2 reductants. Trace-levels of carbonate acted as a pH buffer, while a lower boundary pH of 3 indicated acid buffering by K-feldspar dissolution. The processes identified were used to formulate a kinetic reaction modelling framework that was able to reproduce (i) the observed, transient O_2 consumption and CO_2 production (ii) the major ion composition at completion, and (iii) the observed trace metal releases in each of the treatments. The experiments showed that the approach was useful for identifying and quantifying key geochemical reactions that affect the water quality evolution in artificially recharged anoxic aquifers. The calibrated model provided reaction rate estimates and a parameterized model for all major reductive processes and mineral buffering reactions identified in the main lithologies of the aquifer sections targeted for MAR.

Keywords: ASR, geochemical model, Leederville aquifer, mineral kinetics, pyrite oxidation.

INTRODUCTION

During managed aquifer recharge (MAR) water-sediment interactions strongly affect the water quality evolution of the target aquifer. These interactions will vary in space and time as a result of the physical and geochemical heterogeneity of aquifers (Lowry and Anderson, 2006). Water quality changes will be affected by geochemical reactions within the most permeable parts of the aquifer, where the bulk of the injectant will penetrate, and by reactions occurring within the finer-grained, less permeable but potentially more reactive media. Degradation of the injected water quality, for example by mobilisation of trace metals, may impinge on the economic and technical feasibility of MAR operations (Arthur et al., 2002; Pyne, 1995). Therefore a thorough understanding of the geochemical processes and coupled transport within the aquifer under MAR conditions is fundamental for the viability of large-scale MAR operations which must be solved *a priori* to predict solute transport and water quality changes.

Typical reductants in targeted aquifers are sedimentary organic matter (SOM), pyrite, Fe(II)-silicates, Fe(II)-carbonates and exchangeable ferrous iron (Appelo and Postma, 2005; Prommer and Stuyfzand, 2005). The reduction capacities of these sediment components are a function of their abundance and availability, and, equally important, of their reactivity, i.e., the kinetics of the oxidation reactions and the resulting competition for oxygen (Barcelona and Holm, 1991; Hartog et al., 2002). Besides these oxidation reactions, secondary reactions such as pH-buffering and trace metal release typically become relevant, particularly in response to the proton release by the oxidation of pyrite.

In order to identify and quantify the geochemical controls on the sediment reactivity and its spatial variability in complex heterogeneous systems, detailed geochemical characterisations and incubation experiments have been carried out during this study. The combined geochemical characterisation and incubation experiments were used as a basis to (i) quantify sediment reactivity towards oxygen, (ii) efficiently characterise the spatial and temporal variability of reaction rates in different lithological units, and (iii) formulate and constrain a kinetic reaction modelling framework which incorporated the identified water sediment reactions for assisting the interpretation of the ongoing laboratory-scale column experiments and MAR field trials.

METHODS

ASR field site

The ASR operation is located 15-20 km north of Perth's city centre in Western Australia, and is targeting the Wanneroo Member of the Leederville Formation for the injection and later recovery of drinking water. The formation thickness is around 350 m, and the confined Wanneroo Member lies between 250 and 510 m below ground level (mbgl). The aquifer sediments were deposited during alternating periods of continental, paralic and marine deposition during the early Cretaceous. This alternation in depositional environments have created a complex sedimentary sequence of interbedded sands, silts and shales (Davidson, 1995). The ambient groundwater in the Leederville aquifer is anoxic, NO_3-depleted and slightly brackish, dominated by Na-Cl. Moderate to highly reducing condition prevail, with Eh values between -100 to -170 mV. Groundwater composition is presented in Table 1.

Table 1. Groundwater composition at the study site.

Aqueous component	Concentration (in mg L^{-1}, unless otherwise stated)
pH (-)	6.4
EC (mS m^{-1})	190
TDS[a]	1040
Ca^{2+}	12
Na^{+}	308
Cl^{-}	490
K^{+}	20
Mg^{2+}	24
Mn^{2+}	0.2
SO_4^{2-}	80
Fe	10
HCO_3^{-}	40
NO_3^{-}	<0.05
Temp (°C)	25
DOC	0.4

[a] Total dissolved solids

Sampling and sediment characterisation

Sediment samples were collected from the Leederville Formation, 190-530 metres below ground level (mbgl) from two planned MAR sites, Perth..Recovered drill cores were split in a specifically designed anoxic N_2-positive pressure chamber installed next to the drilling rig. Samples were stored in sealed Al containers under N_2 atmosphere between 0 and 4 °C to prevent oxidation prior to commencing the batch and column experiments. Sediment subsamples were analysed for quantitative mineral content by x-ray diffraction (XRD; PANalytical X'Pert Pro Multi-purpose Diffractometer) and elemental composition by x-ray fluorescence (XRF; PANalytical AXios Advanced instrument). Chromium reducible sulfur (S_{Cr}) was measured according to Sullivan *et al.* (2000); S_{Cr} was assumed to be dominated by pyritic sulfur and thus an indication of pyrite content. Total carbon (TC) was measured by combustion at 1350°C in an oxygen-rich atmosphere and infrared detection (Labfit CS-2000), of the produced CO_2 . Total organic carbon (TOC) was determined after treatment with sulfurous acid (5–6%) and an overnight stand before combustion. Total inorganic carbon (TIC) was determined as the difference between TC and TOC. Detailed analytical results are presented in Descourvieres et al., (2010). Average sediment composition for each lithology is shown in Table 2.

Table 2. Average composition of sands, silts and clays of the Leederville Formation.

Lithology/ sample	Quartz w/w%	K-feldspar w/w%	Albite w/w%	Pyrite w/w%	Kaolinite w/w%	Calcite w/w%	Siderite w/w%	Ankerite w/w%	TOC w/w%	CEC meq/100g
Sand (*n*=124)	64.1±10	26.6±5.8	2.05±2.3	0.60±0.9	5.85±4.8	0.01±0.05	0.12±0.6	0.06±0.6	0.52±0.8	2.62±2.1
Silt (*n*=43)	39.6±11	29.4±5.6	2.67±1.9	1.54±1.4	24.2±10	0.02±0.01	0.77±2.3	*nd*	1.63±1.2	7.60±4.7
Clay (*n*=25)	17.8±13	20.4±5.4	0.87±0.9	2.45±2.2	54.3±14	.006±.007	0.87±1.5	*nd*	3.86±3.5	11.8±6.1

nd: not determined.

Incubation experiments

Forty-five samples from the two deep boreholes were selected to represent the three lithology types (sand, silt and clay). The samples were split into duplicates and the sediment (22.5 g) was mixed in 250 mL Duran bottles with ultra-pure anoxic water (up to 150 mL) under a N_2 atmosphere, leaving a constant headspace. Samples were incubated at 24.2 ± 1°C and connected to an indirect closed circuit respirometer (Micro-Oxymax, Columbus Instruments) for 50 days, during which O_2 consumption and CO_2 production in the headspace were measured every 2.7 hours. During incubation, the sediment-water slurries were stirred by a magnetic stirrer, to ensure a homogeneous chemical system and to enhance gas exchange between the head space to the water phase. At completion of the incubation, slurries were centrifuged at 10,000 rpm and the supernatant was filtered (0.45 µm) prior to complete chemical analysis following the methodologies of APHA (2005).

One sediment sample was divided into ten subsamples and one reference subsample prepared in the same way as previously described and connected to the respirometer. The subsamples were sacrificed by removing them from the respirometer after 0, 1, 4, 7, 11, 14, 18, 23, 28, 32 and the reference sample was disconnected after 37 days. At each sampling time, 2.5 mL was collected from the reference sample and analysed for Na^+, K^+, Ca^{2+}, Mg^{2+}, Cl^-, SO_4^{2-}, Br^-, NO_3^-, NO_2^- by ion-chromatography, as well as pH and Eh. The 2.5 mL of sample was replaced by anoxic distilled water to keep a constant headspace in the reference bottle. For all other samples, supernatants were analysed as *per* the incubation experiments described above.

Kinetic reaction modelling

Based on the selection of the major geochemical processes identified by Descourvieres et al., (2010), a reaction network consisting of a mixture of equilibrium and kinetic reactions was defined using PHREEQC-2 (Parkhurst and Appelo, 1999). The kinetic reaction modelling considered the hydrochemical evolution of the supernatants and the partitioning between the gas and aqueous phases. The developed numerical model combined measured initial sediment composition of each incubation with reaction rate expressions that were compiled from the literature. The major processes accounted pyrite to be the main sink of O_2, whilst the subsequent proton release was the driver for mineral buffering. The representing sequence of major reactions identified and included in the numerical model is presented in Table 3.

Table 3. Oxidation and mineral buffering reaction in the Leederville aquifer sediments and the reaction rate expression sources in the literature. (a) pyrite oxidation, (b) SOM mineralisation (represented as carbohydrate CH_2O), (c) calcite dissolution, (d) dolomite dissolution, and (e) K-feldspar weathering.

Reaction	Kinetic rate expression
(a) $FeS_2 + 3¾\ O_2 + 3½\ H_2O \rightarrow Fe(OH)_3 + 2SO_4^{2-} + 4H^+$	(Eckert and Appelo, 2002; Williamson and Rimstidt, 1994)
(b) $CH_2O + O_2 \rightarrow CO_2 + H_2O$	(Parkhurst and Appelo, 1999)
(c) $CaCO_3 + 2H^+ \rightarrow Ca^{2+} + CO_2 + H_2O$	(Appelo et al., 1998; Plummer et al., 1978)
(d) $CaMg(CO_3)_2 + 4H^+ \rightarrow Ca^{2+} + Mg^{2+} + 2H_2O + 2CO_2$	(Chou et al., 1989; Plummer et al., 1978)
(e) $KAlSi_3O_8 + 7H_2O + H^+ \rightarrow K^+ + Al(OH)_3 + 3H_4SiO_4$	(Sverdrup and Warfvinge, 1995)

RESULTS AND DISCUSSION

Incubation modelling

After the calibration of reaction rate parameters the model simulations were able to closely replicate the O_2 consumption by reductants and the CO_2 production by dissolution of carbonates and SOM mineralization measured during the incubations performed by Descourvieres et al. (2010) (data not shown), and the sacrificial incubations (Figure 1). Generally O_2 consumption and CO_2 production were lower for sands than clays, and O_2 consumption was up to 1-order of magnitude higher than CO_2 production (Descourvières et al., 2010). This low CO_2 production reflected the low carbonate content of the sample, as well as the small buffering capacity and low SOM reactivity. Analytical results suggested the presence of <0.06% highly reactive carbonates which was in agreement with model-based estimates of < 0.1 wt% (Descourvières et al., 2010). The simulated major-ion concentrations and supernatant pHs showed overall an excellent agreement with the measured data for the non-sacrificial incubations and the temporal evolution observed in the sacrificial incubation (Figure 1). Observed sulfate concentrations correlated negatively with pH (R^2=0.87) in the non-sacrificial experiments, indicating that the acidification triggered by pyrite oxidation was the key process driving mineral dissolution (Descourvières et al., 2010). The rapid depletion of the carbonates due to the very limited buffering capacity contained in samples is also indicated by the stalled Ca^{2+} and Mg^{2+} concentrations during the aqueous evolution of the sacrificial experiments (Figure 1). Following the carbonate depletion, feldspar and minor plagioclase dissolution continued to buffer pH at lower levels (~3), accompanied by an increase in aqueous K^+, Al and Si (Figure 1). The model indicates how feldspar buffering started to occur when the aqueous solution reached a pH of 3 and when carbonate buffering (shown mainly by Ca^{2+} and Mg^{2+}) was effectively halted. The observed initial increase in Na^+ concentrations was mainly attributed to the initial re-equilibration of the exchanger composition, while simulations indicated that dissolved Si was derived from aluminosilicate dissolution.

The model simulations illustrated that within a pH of 4-6, Fe and Al concentrations during incubations were indeed controlled by the solubility of $Fe(OH)_3$ and $Al(OH)_3$. Aqueous Fe and Al concentrations increased linearly due to the (re-)dissolution of $Al(OH)_3$ and $Fe(OH)_3$ when the pH became lower as 4 (Descourvières et al., 2010).. Supernatant concentrations of selected trace metals increased with decreasing pH and increasing SO_4^{2-} concentrations in both the sacrificial, and non-sacrificial incubations (Figure 1). A sharp increase in concentrations occurred when pH decreased below 4, which coincided with the depletion in alkalinity and sub-saturation conditions for ferric oxyhydroxides. The model closely described the observed concentrations by including a stoichiometric release of trace metals, coupled to pyrite oxidation. The stoichiometric ratio of the metals relative to the sedimentary pyrite content was found by a regression analysis that assumed pyrite to be the major metal source (Descourvières et al., 2010).

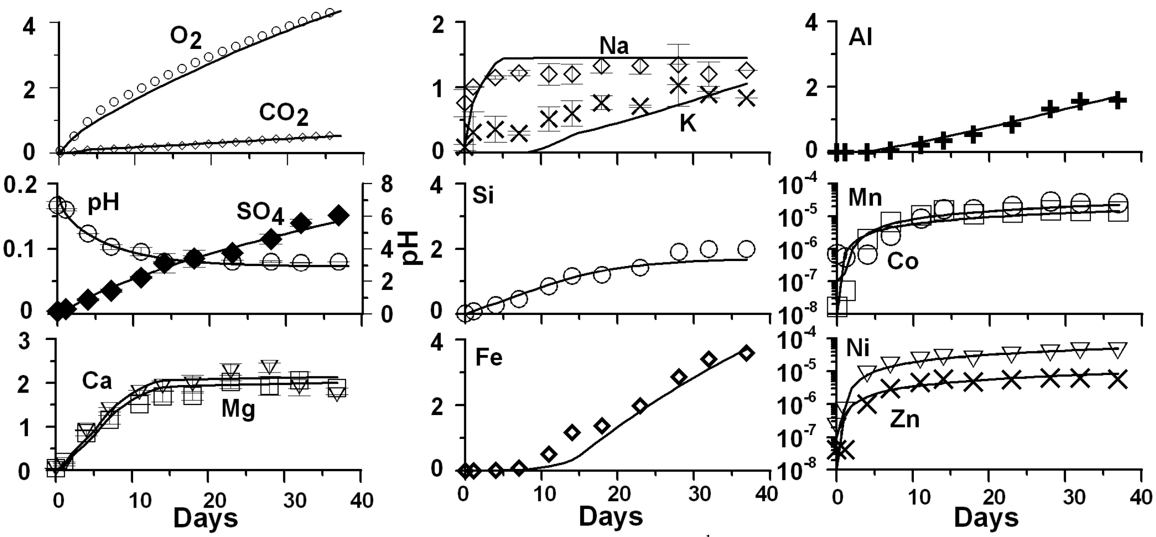

Figure 1. Average measured concentrations (symbols; mmol L^{-1}) and simulated (solid lines) supernatant ion concentrations and pH over 37 days in the sacrificial experiment. Error bars represent the variation between sacrificed and reference incubation batches.

Kinetic rate parameters

The average concentration-normalized pyrite weathering rates obtained from the kinetic reaction model are presented in Table 4. The obtained values ranged within previously reported pyrite (monomineralic) oxidation rates determined in the laboratory at circumneutral pH and 25°C (Andersen et al., 2001; Moses and Herman, 1991; Nicholson et al., 1988; Williamson and Rimstidt, 1994). The similar ranges in concentration-normalised weathering rates across lithologies indicated that reaction rates were overall proportional to the mineral concentration/surface area. The concentration-normalized aluminosilicate and carbonate weathering rates were one order of magnitude higher than for pure mineral phases reported in the literature. This was likely due to the fact that the rate of pyrite oxidation and consequent acid production actively controlled the subsequent mineral dissolution rate. Despite the discrepancy in absolute values, the model-derived rates were consistent with reported relative differences in weathering estimates; feldspars rates between K-feldspar and plagioclase were similar (Knauss and Wolery, 1986; White and Brantley, 1995), and calcite weathering rates were faster than dolomite/ankerite rates by at least one order-of-magnitude (Chou et al., 1989). Overall, the weathering rates for the sacrificial experiments were within the range obtained for the non-sacrificial experiments, indicating the ability to extrapolate the approach.

Table 4. Average weathering rates (mol $g^{-1}_{mineral}$ s^{-1}) and standard deviation obtained for kinetic mineral reactions from incubation experiments simulations. Literature weathering rates compiled from references mentioned in the text.

Lithology	Pyrite	K-feldspar	Albite	Calcite	Ankerite
Sand	$1.1\pm1.0\text{x}10^{-10}$	$2.1\pm1.7\text{x}10^{-12}$	$1.7\pm1.2\text{x}10^{-11}$	$7.1\pm4.6\text{x}10^{-10}$	$1.6\pm3.3\text{x}10^{-11}$
Silt	$1.1\pm0.9\text{x}10^{-10}$	$1.2\pm0.5\text{x}10^{-12}$	$5.3\pm3.4\text{x}10^{-12}$	$7.0\pm4.5\text{x}10^{-10}$	$6.0\pm4.7\text{x}10^{-12}$
Clay	$1.8\pm0.9\text{x}10^{-10}$	$3.8\pm0.9\text{x}10^{-12}$	$7.1\pm0.5\text{x}10^{-12}$	$4.3\pm1.5\text{x}10^{-10}$	$1.2\pm0.6\text{x}10^{-11}$
Literature	$5\text{x}10^{-8}$ - $4\text{x}10^{-11}$	$1\text{x}10^{-13}$ - 10^{-11}	$1\text{x}10^{-13}$ - 10^{-12}	10^{-10} - 10^{-12}	10^{-12} - 10^{-13}

CONCLUSIONS

The presented kinetic modeling approach was found to provide a robust method for interpreting reductant behavior, and to identify and quantify reaction rates and their variability for competing redox and buffering reactions in different lithologies. The overall assessment was constrained by an extensive dataset including gas-phase evolution and aqueous compositional changes that account for competing reductant reactivity and their relative contributions to oxidant consumption. This type of results can be used to underpin the construction of conceptual and numerical models for reactive transport in hydrogeological and geochemical heterogeneous aquifers, where differences in the reactivity of individual lithological units can strongly affect water quality evolution.

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Numerical evaluation of arsenic behaviour during deep well injection

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ABSTRACT

Deepwell injection is a frequently used managed aquifer recharge (MAR) technique to replenish aquifers in conjunction with water quality improvements. However, where oxygenated water is recharged into anoxic aquifers the mobilization of trace metals, including arsenic (As) can occur and potentially lead to elevated metal(loid) concentrations in the recovered water. Well-documented examples of As mobilization during MAR were reported from the Netherlands, from west-central and southwest Florida and from Australia. While conceptual models for the fate of arsenic under such circumstances exist, they are generally not rigorously tested through incorporation into reactive transport models and subsequent application to field data sets.

In this study, we use data from a case of arsenic mobilization that occurred during a deepwell injection experiment in the Netherlands for the development and evaluation of conceptual and numerical models.

Pyrite oxidation and the precipitation and dissolution of amorphous iron-oxides were shown to be key chemical processes to control the fate of arsenic during the injection experiment. In the model that best reproduced field observations, the release of arsenic was modeled by oxidative dissolution of As-pyrite and the subsequent attenuation by surface complexation reactions of predominantly As(V) to ferrihydrite.

INTRODUCTION

Deepwell injection is a frequently used managed aquifer recharge (MAR) technique to replenish aquifers and to achieve water quality improvements. However, where oxygenated water is recharged into anoxic aquifers the mobilization of trace metals, including arsenic (As) can occur and potentially lead to elevated metal(loid) concentrations in the recovered water. Well-documented examples of As mobilization during MAR were reported from the Netherlands (Stuyfzand and Timmer, 1999), from west-central and southwest Florida (Arthur et al., 2002) and Australia (Vanderzalm et al., 2007). While preliminary conceptual models for the fate of arsenic under such circumstances exist, they were generally not rigorously tested through incorporation into reactive transport models and application to MAR field sites. Coupled flow and reactive transport simulations have previously been shown to provide a useful framework for advancing the understanding of field-scale data and processes. Conceptual models can be formulated and their applicability tested.

In the present study, geochemical data from a storage, transfer and recovery (ASTR) system were used to develop, evaluate and constrain conceptual and numerical models of arsenic mobilization and attenuation under field-scale conditions.

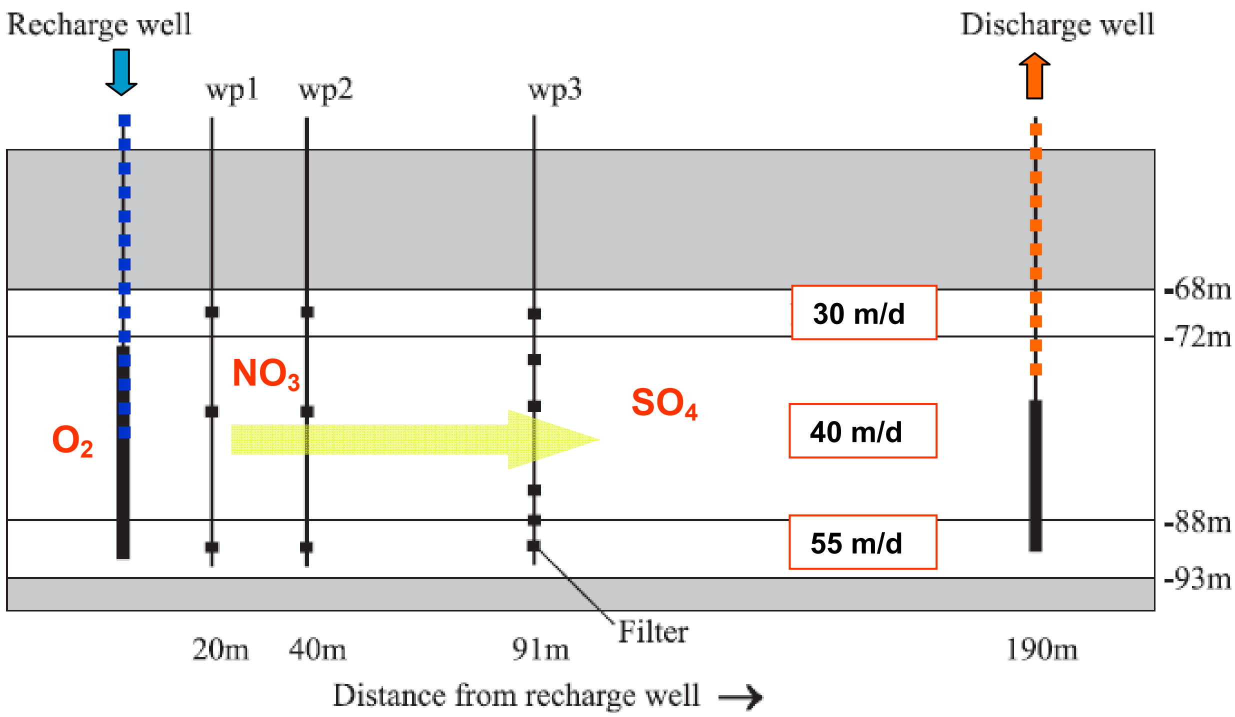

Figure 1 Schematic diagram of the ASTR operation at Langerak, Netherlands

FIELD SITE

The technical feasibility for deep well injection of pretreated river Rhine water was tested as a possible option to meet growing water demands in the region. A trial site was built in Langerak along the river Lek, a tributary of the Rhine River in South-West Netherlands. The target aquifer is composed of permeable fluvial sands at depths ranging between 68 m and 95 m b.g.l. underlain and overlain by clayey aquitards that confine the aquifer. The MAR system consists of one recharge and one recovery well (distance 190 m), as well as three monitoring wells emplaced along the flow direction (Figure 1). Injection at a rate of $35m^3\ hr^{-1}$ and recovery at a rate of $60m^3\ hr^{-1}$ proceeded without significant interruptions for almost 600 days. The injection trial was accompanied by a detailed monitoring program (Stuyfzand and Timmer, 1999).

REACTION NETWORK

The reaction network developed for this study consisted of a mixture of equilibrium-controlled and kinetic reactions of aqueous species, mineral phases, ion exchanger sites and surface complexes. It was adapted to the site-specific conditions and incorporated into the database of the reaction database of the reactive transport model PHT3D (Prommer et al., 2003, Prommer and Post, 2010). At the study site pyrite and sediment-bound organic matter were the most prominent potential reductants under the anoxic ambient conditions. Total dissolved arsenic concentrations were low or below detection limit prior to the start of injection. Mineralogical and geochemical analyses of the fluvial sands (Stuyfzand and Timmer, 1999) showed arsenic to be associated with pyrite.

The injection of oxygenated water into the aquifer leads to the oxidation of pyrite and the formation of amorphous iron-oxides (Stuyfzand and Timmer, 1999). Associated with pyrite oxidation was the mobilization of As while its adsorption to neo-precipitated amorphous iron oxides (e.g., to ferrihydrite) was thought to control aqueous As concentrations after mobilization.

Pyrite oxidation by oxygen and nitrate was included in the reaction network on the base of previously proposed and applied reaction rate expressions (Williamson and Rimstidt (1994), Eckert and Appelo (2002), Prommer and Stuyfzand (2005)):

$$r_{pyr} = \left[\left(C_{O_2}{}^{0.5} + f_2 C_{NO_3^-}{}^{0.5} \right) C_{H^+}{}^{-0.11} \left(10^{-10.19} \frac{A_{pyr}}{V} \right) \left(\frac{C}{C_0} \right)_{pyr}^{0.67} \right] \tag{1}$$

where r_{pyr} is the specific oxidation rate for pyrite, C_{O_2}, $C_{NO_3^-}$ and C_{H^+} are the oxygen, nitrate and proton groundwater concentrations, A_{pyr}/V is the ratio of mineral surface area to solution volume and (C/C_o) is a factor that accounts for changes in A_{pyr} resulting from the progressing reaction. f_2 is a constant, which was assumed to be unity, as in previous work (Eckert and Appelo, 2002, Prommer and Stuyfzand, 2005).

Arsenic mobilization was simulated through a release rate that was directly linked to the computed pyrite oxidation rate following an experimentally determined stoichiometric ratio. Sorption of arsenic was assumed to occur as a surface complexation reaction with neo-precipitated ferrihydrite inside the oxidized zone of the aquifer. The generalized two-layer surface complexation model of Dzombak and Morel (1990) for sorption on ferrihydrite was employed, extended by reactions for Fe^{2+} and HCO_3^- (Appelo et al. 2002). The surface complexation approach allowed for the simulation of competitive sorption between arsenic and other ions for a finite number of sorption sites. Increasing sorption capacity under the successively more oxidising conditions was modeled by coupling the moles of the surface complex to the (increasing) concentrations of ferrihydrite.

SIMULATED EVOLUTION OF THE REDOX ZONATION

The numerical reactive transport simulations successfully reproduced the spatial and temporal hydrochemical changes that occurred during the deep-well injection experiment. Oxidation of pyrite exerts the strongest influence on the redox chemistry, as clearly illustrated by diminishing electron acceptor concentrations (O_2 and NO_3^-) and increasing SO_4^{2-} concentrations. A comparison of nonreactive and reactive simulation results (Figure 2) shows that the breakthrough of nitrate was strongly affected by rapid denitrification. This resulted in a complete nitrate removal within short distances from the injection well. Similarly, the comparison indicates the extent to which SO_4^{2-} concentrations were impacted by pyrite oxidation. Full details are reported in Wallis et al. (2010).

SIMULATED FATE OF ARSENIC

Initial model simulations that assumed redox equilibrium between arsenic and other species failed to reproduce the measured As data. The comparison between simulated and observed data suggested that slow redox transformations of the freshly released As would potentially explain the high mobility of As at the start of the injection trial. Deviation from equilibrium controlled arsenic speciation is commonly observed in natural systems and the rate of oxidation of As(III) in groundwater is considered to be slow (Smedley and Kinniburgh, 2002). This hypothesis was tested by decoupling arsenic from the overall redox equilibrium and by modelling the transformation/oxidation of As(III) to As(V) as a slow, kinetically controlled reaction.

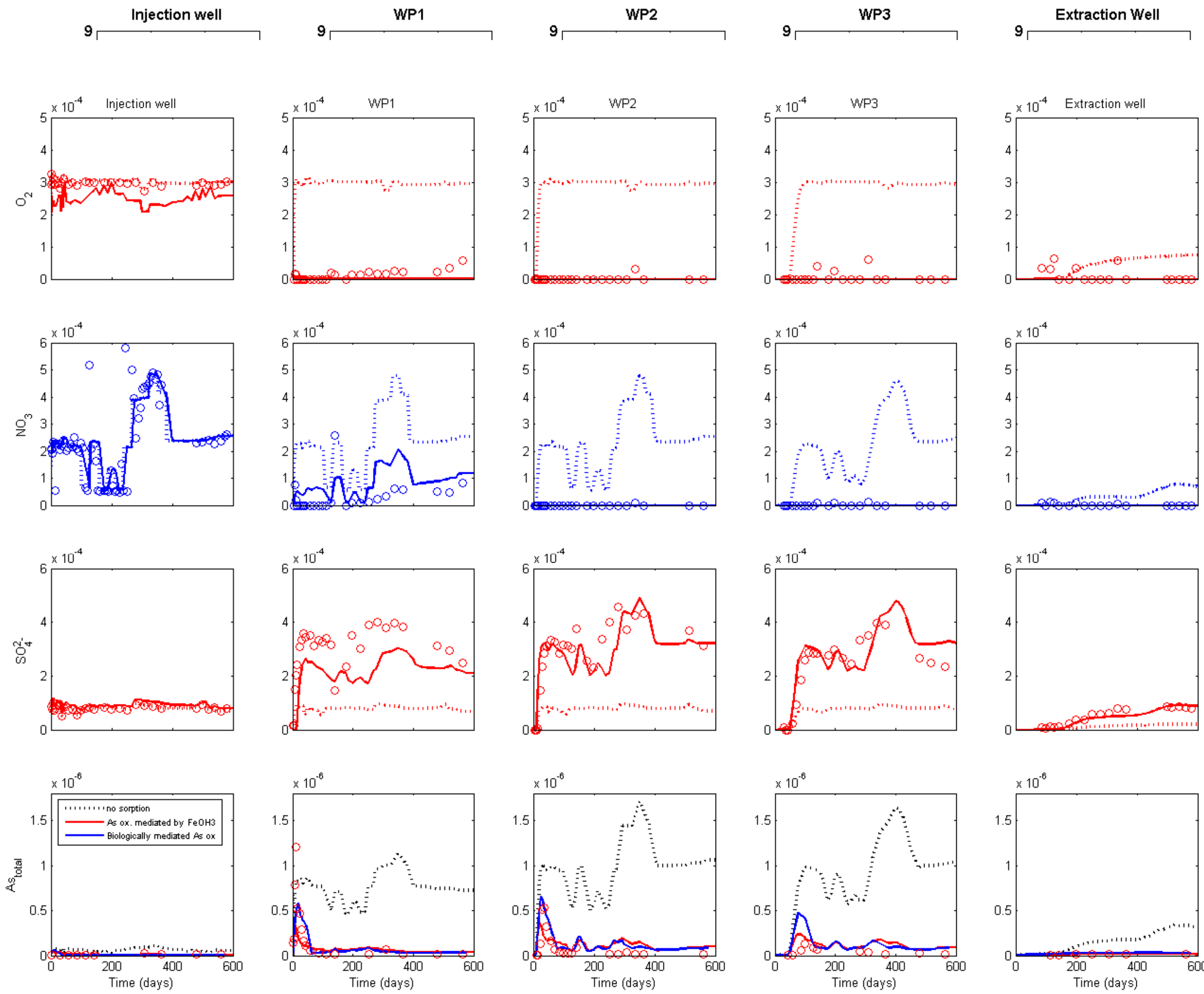

Figure 2 Measured (circles) and simulated (solid lines) concentrations (mol L^{-1}) of aqueous components at the injection well, at WP1, WP2, WP3 and in the extraction well for the deep part of the aquifer (Layer 5). Dotted lines indicate simulated results of the non-reactive model for O_2, NO_3 and SO_4^{2-}. Total arsenic concentrations are shown for (a) black line: no attenuation by sorption, (b) red line: abiotic oxidation of As(III) by $Fe(OH)_3$, (c) blue line: biologically mediated As(III) oxidation.

Abiotic As (III) oxidation has been shown to be catalysed by the presence of MnO_2, iron oxides and/or Fe(II) (De Vitre et al. 1991, Manning et al. 2002, Hug and Leupin, 2003). The most likely electron acceptors for As(III) oxidation at the Langerak site was found to be either freshly precipitated Mn or Fe-oxides. A second-order kinetic reaction for arsenic oxidation was therefore included in the reaction network and its application tested. The rate expression incorporated previous results of batch experiments that suggested a close interactions of arsenic with pyrolusite (Radu et al. 2008) :

$$\frac{\partial As(III)}{\partial t} = -k \times C_{As(III)} \times C_{oxidant}$$

where $C_{As(III)}$ and $C_{oxidant}$ are the arsenite and oxidant concentrations (i.e., MnO_2 and ferrihydrite) and k is the second-order rate constant. In the simulations that employed kinetic As(III) oxidation arsenic remained initially mobile as long as As(III) remained the predominant redox state. However, after oxidation to As(V) attenuation occurred due to its relatively higher affinity for the sorption sites provided by ferrihydrite under the geochemical conditions at the site.

Besides abiotic arsenic oxidation, several microbial respiratory and non-respiratory enzymatic systems were reported to potentially influence the oxidation state of arsenic (Oremland and Stolz, 2003). Therefore a second kinetic model was developed to test this hypothesis. This kinetic model considered the microbial growth and decay of a single group of specific arsenite oxidizers. Growth rates were assumed to depend on the availability of As(III) (electron donor) and nitrate (electron acceptor), with the energy derived being used to fix carbon into organic cellular material and to maintain growth according to:

$$6NO_3^- + 5HCO_3^- + 19H_2AsO_3^- \rightarrow C_5H_7O_2N + 5NO_2^- + 2H_2O + 19HAsO_4^{2-} + 13H^+ \qquad (3)$$

The mass balance of the arsenic oxidizing microbial group was assumed to follow:

$$\frac{\partial X}{\partial t} = \left.\frac{\partial X}{\partial t}\right|_{growth} + \left.\frac{\partial X}{\partial t}\right|_{decay} \qquad (4)$$

A standard Monod-type growth model:

$$\left.\frac{\partial X}{\partial t}\right|_{growth} = v_{\max} X \frac{C_{As(III)}}{K_{As(III)} + C_{As(III)}} * \frac{C_{NO3-}}{K_{NO3-} + C_{NO3-}} \qquad (5)$$

and a first-order biomass decay term were used

$$\left.\frac{\partial X}{\partial t}\right|_{decay} = -v_{decay} X \qquad (6)$$

whereby X , $C_{As(III)}$ and C_{NO3-} are the microbial arsenite and nitrate concentrations respectively, and v_{decay} and $v_{\max}$ are the decay and uptake rate constants, respectively.

After model calibration this kinetical model also allowed for a successful description of the observed As behaviour (Figure 2).

CONCLUSIONS

The reactive transport simulations provided a comprehensive description of the spatial and temporal hydrochemical changes that were documented during the injection of oxidized waters into a pyritic aquifer. Reactive transport modelling has thereby shown to be a valuable tool for integrating specific processes and to test their individual impact on As behaviour during the injection experiment at the Langerak site, where elevated As concentrations were observed during the initial phase of the experiment. This type of coupled modelling is useful were physical transport as well as geochemical processes can similarly affect concentration changes. The developed conceptual model serves as hypotheses to be further tested and adapted for other anoxic, pyritic field sites that are recharged by oxic waters.

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Comparing water quality during urban stormwater MAR via ASTR and ASR

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Abstract
Managed aquifer recharge (MAR) can be used to recycle urban stormwater and wastewater to supplement the available water resources. While subsurface storage can provide natural treatment, it can also have deleterious effects on the recovered water quality through water-rock interactions. A recent investigation into the potential for stormwater recycling via a carbonate aquifer is used to determine the important hydrogeochemical processes that impact on the recovered water quality under two operating scenarios; (1) separate wells for injection and recovery wells, representing Aquifer Storage Transfer and Recovery; and (2) a single well for injection and recovery, representing Aquifer Storage and Recovery (ASR).

Calcite dissolution leads to the dominant inorganic water quality change regardless of the mode of MAR operation (ASTR or ASR). Cation exchange is evident through the ASTR mode but is limited to the initial pore flushes of the storage zone. Aquifer storage provides treatment through removal of ~35 % of the dissolved organic carbon (DOC). During ASTR, enhanced DOC removal is attributed to adsorption. Oxidation of pyrite is evident during the initial stages of injection until the pool of reactive pyrite within the storage zone is consumed, affecting the quality of water recovered via the ASTR mode only.

Keywords
calcite dissolution, cation exchange, organic matter, pyrite oxidation

INTRODUCTION

The reuse of urban stormwater and wastewater is becoming increasingly important as the availability of water resources declines under climatic change and increasing demand from urbanisation. Managed aquifer recharge (MAR) is a means of water recycling via aquifers and the period of aquifer storage can be designed to provide adequate residence time for passive treatment processes (Dillon and Toze, 2005) such as inactivation of pathogens (Page *et al.*, 2010; Toze *et al.*, 2010), degradation of organic chemicals (Ying *et al.*, 2008) and removal of nitrogen (Fox *et al.*, 2002).

Subsurface storage can also have deleterious effects on the recovered water quality (Vanderzalm *et al.*, 2009) and the integrity of the aquifer matrix through aquifer clogging (Olsthoorn, 1982; Pavelic *et al.*, 2007), mixing with more saline groundwater (Pavelic *et al.*, 2006), mineral dissolution (Mirecki *et al.*, 1998) or redox processes induced by the injection of oxidisable organic matter (Vanderzalm *et al.*, 2006) which is particularly relevant when recycled water of impaired quality is used. These processes can lead to increases in the concentration of major ions and salinity, but also increases in iron, arsenic and hydrogen sulfide (Stuyfzand, 1998; Stuyfzand *et al.*, 2002; Arthur *et al.*, 2003; Prommer and Stuyfzand, 2005; Pyne, 2005) which can exceed relevant water quality guidelines (NHMRC-NRMMC, 2004; NRMMC–EPHC–AHMC, 2006; NRMMC-EPHC-NHMRC, 2009). It has also been previously reported that equilibrium with carbonate minerals through dissolution occurs soon after injection (Mirecki *et al.*, 1998) and counteracts the hydraulic impacts of aquifer clogging when a nutrient rich source water, such as reclaimed wastewater, is used in MAR (Pavelic *et al.*, 2007).

A recent investigation into the potential for stormwater recycling to provide a drinking water supply via ASTR in a confined limestone aquifer (Page *et al.*, in press), allows a rigorous investigation of the dominant processes that impact on the recovered water quality (Page *et al.*, 2009; Vanderzalm *et al.*, in press). Prior to this, a number of field trials illustrated that ASR within the same aquifer system can be used to recycle stormwater and wastewater for non potable uses (Herczeg *et al.*, 2004; Vanderzalm *et al.*, 2006). The experience with water recycling via MAR for non-potable end-use has highlighted the propensity for a number of geochemical reactions that may impact on the suitability of the recovered water for potable purposes. Comparison between ASTR and ASR operations undertaken within a carbonate aquifer can be used to evaluate the influence of the source water quality, in particular the concentration and character of dissolved organic matter within the source water, and the MAR technique utilised (ASR vs ASTR) on the extent of these processes and their impact on the recovered water quality. This level of understanding is essential in developing the predictive capability regarding the human health and environment risks associated with water recycling via aquifers (NRMMC-EPHC-NHMRC, 2009).

This paper examines the response of a carbonate aquifer to the injection of urban stormwater during MAR in order to understand the importance of geochemical reactions on recovered water quality. The specific objectives of this paper are to:

- Evaluate the dominant geochemical reactions that impact on recovered water quality.
- Identify the difference in processes induced during ASTR and ASR in the same aquifer.

METHODS

Site and operational details

The Salisbury ASTR project aims to store wetland treated urban stormwater in a brackish carbonate aquifer for recovery at a potable quality. Urban stormwater is harvested from a mixed residential and industrial catchment area in the City of Salisbury, South Australia and treated via passage through two sedimentation basins in series ending with a constructed wetland. The ASTR well field consists of four outer injection (IW) wells surrounding two inner recovery (RW) wells (Figure 1), with a minimum well spacing of 50 m between the inner and outer wells providing an average aquifer residence time of 240 days.

The IW and RW wells were completed as open hole between 165 and 185 m below ground surface (bgs), and target the lower Tertiary marine sediments of the Port Willunga Formation, a well-cemented sandy limestone aquifer known locally as the T2 aquifer. The T2 aquifer is approximately 60 m thick at this location (160-220 m bgs) and is overlain by a 7 m thick clay aquitard of Munno Para Clay. The ASTR wells partially penetrate the T2 aquifer to preclude a zone of higher hydraulic conductivity in the lower part of the target aquifer (T2c) (Kremer *et al.*, 2010). Karstic features were not identified during well construction on this site. The aquifer mineralogy comprises 65 $\pm$ 23 % calcite (~6 % Mg-calcite), 30 $\pm$ 22 % quartz and traces of ankerite, goethite, hematite, pyrite, albite, and microcline (Page *et al.*, 2009). The organic carbon content is <0.5 %. Trace elements within the sediments include arsenic (6-144 ppm), chromium (34-128 ppm), nickel (<2-15 ppm), vanadium (10-189 ppm) and zinc (<2-13 ppm).

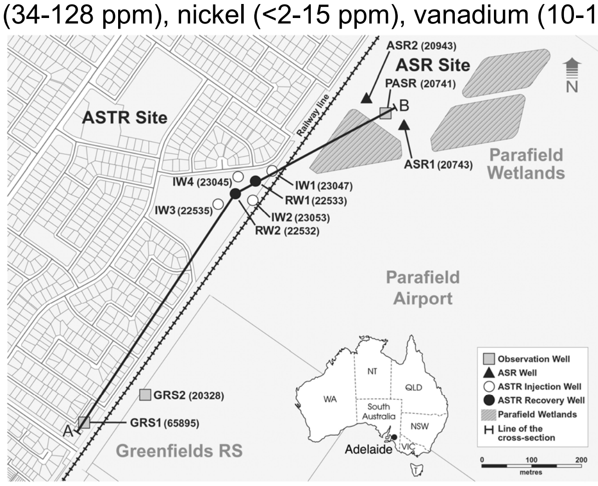

Figure 1 City of Salisbury water harvesting facilities in the Parafield area, identifying the location of wells at the ASTR site and a nearby ASR site (after Kremer *et al.*, 2010).

Sampling and analysis
The quality of the ASTR source water was determined from grab samples collected at the outlet of the wetland, while groundwater was collected by pumping after 3 well volumes were purged as indicated by stable *in situ* measurements of temperature, pH and electrical conductivity (EC). Field parameters, pH, temperature, dissolved oxygen (O_2), EC and redox potential (Eh), were measured using a TPS-FL90 water quality analyser, with probes in a flow-through cell to prevent exposure to the atmosphere. Samples for the analysis of dissolved species; iron, arsenic, nitrogen, phosphorus and organic carbon, were filtered through 0.45 µm filter immediately after sampling, and all samples for trace metals and metalloids were acidified to pH <2 with HNO_3. Water samples were maintained below 4 °C and analysed within 4 hours according to standard methods (APHA, 2005). Alkalinity (as HCO_3^-) was determined by potentiometric titration to a fixed end-point pH; chloride, total Kjeldahl nitrogen (TKN), ammonia, nitrate, nitrite, total and filterable reactive phosphorus (TP, FRP) were analysed by colorimetric flow analysis; major ions, iron, manganese, arsenic, nickel, lead and zinc were measured by ICP-ES; organic carbon (OC) was determined by flame ionisation; and biodegradable DOC (BDOC) was calculated from the change in DOC during incubation with a bacterial inoculum.

Mineral saturation indices were calculated using PHREEQC (Parkhurst and Appelo, 1999). Chloride was used as a conservative tracer to calculate the fraction of source water present in groundwater samples. The species mass balance was then calculated as the difference between the measured concentration and that predicted if conservative behaviour was followed.

RESULTS AND DISCUSSION
The ambient groundwater within the storage zone is brackish, anoxic and low in nutrients, while in contrast the wetland treated urban stormwater is fresh, oxygenated and of variable nutrient content. The salinity of the ambient groundwater renders it unsuitable for drinking with several parameters including TDS, sodium, chloride, sulfate and iron in excess of the Australian Drinking Water Guidelines (ADWG; NHMRC-NRMMC, 2004) (Table 1).

Samples of the wetland treated stormwater prior to storage in the aquifer meet the ADWG aside from colour, iron and faecal indicator bacteria, all of which can be generated within the wetland itself (Page *et al.*, 2009). The stormwater is sub-saturated with respect to calcite, indicated by the saturation index for calcite $SI_{calcite} < 0$ (average -1.2; Table 1) showing the potential for the source water to induce calcite dissolution upon injection into the storage zone. The C, N and P concentrations in the ASTR source water are greater than present in the ambient groundwater, but are generally low due to the nature of the residential urban catchment area. Nutrient removal is achieved during transport through the wetland, with removal efficiencies reported at approximately 43 % for total N and P (Page *et al.*, 2009).

Nutrients and DOC in the source water will induce redox reactions that serve to reduce the DOC content via bacterial oxidation of organic matter. However, redox reactions can also lead to increased concentrations of iron or hydrogen sulfide if the redox sequence progresses to reduction of iron(III) or sulfate, and will stimulate additional calcite dissolution to buffer the acidity that is produced. Much uncertainty still exists about the effect of organic matter character and concentration on biogeochemical reactions. The changes in organic matter character and concentration, observed through the behaviour of DOC, result from both physical and biogeochemical factors whose contribution changes with changing environmental conditions.

Reduced minerals, such as pyrite, are likely to by oxidised when oxygenated stormwater is introduced into the anoxic T2 aquifer, thus competing with organic matter for reaction with injected oxygen. Nitrate can also react with pyrite, but is present at low concentrations in the stormwater. Pyrite oxidation has been reported as a prevelant water quality concern in MAR schemes due to the mobilisation of trace constituents (Stuyfzand, 2002; Arthur *et al.*, 2003).

Following aquifer storage, groundwater in the outer wells at the end of the flushing phase (ASTR) illustrates similar quality to the water recovered from the central wells (ASR) and is fresh, anoxic with lower redox potential (Eh) and nutrient content than the source water. The period of aquifer flushing was successful in freshening of the groundwater in the MAR storage zone, with the major ion chemistry progressing toward the source water end-member.

Table 1 Composition of the water recovered from the outer and inner wells of the ASTR field trial in comparison to the ambient groundwater (GW), the urban stormwater (SW) source and Australian drinking water guidelines (ADWG).

mg/L unless stated	ADWG[A]	Ambient GW[(3)]	SW source water[(43)]	Outer wells (ASTR)[(4)]	Inner wells (ASR)[(4)]
SS		3±1	3.7±3.7	<1	<1
TDS	500	2020±10	137±32	408±60	293±59
pH (pH units)	6.5-8.5	6.9±0.1	7.1±0.5 [(19)]	7.9±0.3	7.4±0.6
Temp (°C)		25.5±0.4	12.2±5.0 [(19)]	19.9±2.2	17.8±1.2
O_2		<0.1	4.8±1.8 [(19)]	<0.1	1.2±1.1 [(3)]
Eh (mV SHE)		200 [(1)]	328±62 [(19)]	-5±7	80±90
Colour (HU)	15	14±24	50±38 [(24)]	nd	25±3
Na^+	180	501±5	19±4	80±21	45±16
K^+		13.3±0.2	3.5±1.3	4.6±0.4	4.7±0.7
Ca^{2+}		135±5	23±8	41±3	54±5
Mg^{2+}		83±0.3	4.4±1.8	23±2	9.4±2.1
Cl^-	250	920±7	27±7	109±24	61±21
SO_4^{2-}	250	275±5	10.4±2.5	48±14	20±6
HCO_3^-		317±11	89±34	211±14	206±28
Fe-total	0.3	1.54±0.06	0.58±0.44	0.35±0.01	0.38±0.01
Fe-soluble		1.54±0.02	0.19±0.20	0.32±0.04	0.35±0.07
Mn-total	0.1	0.007±0.001	0.072±0.10	0.0021±0.0003	0.060±0.003
Mn-soluble		0.007±0.001	0.046±0.10 [(28)]	0.0021±0.0004	0.059±0.002
As-total	0.007	0.011±0.001	0.001±0.002	0.005±0.001	0.002±0.001
Ni-total	0.02	<0.0005	0.0012±0.0016	<0.0005	<0.0005 [(1)]
Pb-total	0.01	<0.0005	0.001±0.001	<0.0005	<0.0005 [(1)]
Zn-total	3	0.038±0.007	0.024±0.021	0.014±0.007	<0.003 [(1)]
TOC		1.4±0.2	7.3±4.0	2.5±0.1	4.8±0.7
DOC		1.4±0.2	6.3±3.2	2.5±0.3	4.7±0.8
BDOC		nd	2.4±1.9 [(6)]	nd	nd
NH_4^+-N	0.5 as NH_3	0.035±0.03	0.023±0.040	0.026±0.003	0.14±0.02
NO_3^-+NO_2^--N	50 as NO_3^-	<0.005	0.008±0.023	<0.005	0.006±0.004
TKN		0.04±0.03	0.41±0.21	0.12±0.02	0.33±0.08 [(3)]
TP		0.012±0.009	0.054±0.034	0.011±0.003	0.030±0.003
FRP		0.005±0.003	0.015±0.012	0.004±0.002	0.015±0.005
SAR		8.4±0.1	0.9±0.1	2.8±1.0	1.5±0.5
$SI_{Calcite}$		-0.06±0.02	-1.2±0.5 [(18)]	0.3±0.3	-0.1±0.5
Faecal coliforms (cfu/100 mL)	0	0	40±60	0	0
E-coli (cfu/100 mL)	0	0	40±60	0	0

[A] NHMRC-NRMMC, 2004; nd=not determined; [(n)]=number of samples

Dominant reactions that impact on recovered water quality

Mass balance calculations reveal some changes to the major ion chemistry aside from conservative mixing between the end-member waters (Table 2). Gains in bicarbonate, calcium, magnesium and sodium are the most significant changes to the major ion chemistry of the recovered water. The excess calcium and bicarbonate is indicative of calcite dissolution, but the gain in calcium alone is not sufficient to balance the bicarbonate release which is better balanced by the sum of the excess in Ca^{2+}, Mg^{2+} and Na^+. This suggests that ion exchange is also occurring during injection, removing Ca^{2+} from solution for Na^+ and Mg^{2+} on exchange sites. Gains in these ions result in a slight increase in the sodium adsorption ration (SAR) of the recovered water from 0.9 in the source water to 1.5 in the recovered water but this does not pose any risk of soil sodicity resulting from use in garden irrigation (NRMMC–EPHC–AHMC, 2006).

The magnitude of calcite dissolution following the injection of wetland treated urban stormwater into the T2 aquifer, regardless of the mode of MAR operation, is approximately 1.0 mmol/L, based on the Ca^{2+}+Mg^{2+}+Na^+ excess in the recovered water. This is slightly greater than a previous study of stormwater recharge during ASR also within the same aquifer system where 0.6 mmol/L calcite dissolution was calculated from the recovered water quality (Herczeg *et al.,* 2004). Despite evidence of a higher rate of calcite dissolution than previous ASR applications in the T2 aquifer, excessive dissolution is not expected to limit the lifetime of injection wells based on the projections for typical annual injection volumes (Page *et al.*, 2009).

A gain in sulfate is evident in the ASTR mode only, declining in significance when the groundwater in the outer wells is over 80 % source water. This increase in sulfate can be attributed to oxidation of pyrite, which has limited impact on water quality during the ASTR mode and no impact in ASR mode due to the low content of reactive pyrite within the storage zone. The concomitant increase in iron is not seen presumably due to precipitation of iron(hydr)oxides providing sorption sites that can act to control any trace species that are mobilised.

Table 2 Calculated 'excess' in major ions and dissolved organic carbon in groundwater within the ASTR storage zone (positive value= gain; negative value=loss).

	ASR-inner wells[(4)]	ASTR-outer wells end of injection[(4)]	ASTR time series-IW1 during injection				
			5/7/07	13/8/07	11/9/07	9/1/08	1/9/08
fraction of source water (f)	0.96±0.02	0.92±0.02	0.36	0.50	0.66	0.83	0.88
			meq/L excess				
HCO_3^-	1.8±0.4	1.7±0.2	1.1	1.8	2.3	2.1	1.6
SO_4^{2-}	*-0.02±0.06*	0.3±0.2	0.6	1.0	1.5	0.6	0.2
Ca^{2+}	1.3±0.1	0.42±0.06	0.6	0.6	0.6	0.4	0.4
Mg^{2+}	0.11±0.06	0.89±0.04	0.8	0.8	0.9	0.8	0.9
Na^+	0.3±0.3	0.8±0.5	1.7	2.1	3.1	1.8	0.8
K^+	0.01±0.02	*-0.001±0.005*	0.03	0.03	0.03	0.01	*0.003*
Ca^{2+}+ Mg^{2+}	1.5±0.1	1.3±0.1	1.4	1.4	1.5	1.2	1.4
Ca^{2+}+ Mg^{2+}+Na^+	1.8±0.4	2.1 ±0.4	3.1	3.5	4.6	3.0	2.1
(Ca^{2+}+ Mg^{2+}+Na^+)/ HCO_3^-	1.0	1.2	2.8	1.9	2.0	1.5	1.3
DOC (mmol/L)	-0.10±0.07	-0.26±0.02	-0.14	-0.17	-0.25	-0.30	-0.24

Italics indicate the change is <10 % from conservative behaviour; [(n)]=number of samples

There is also evidence of nutrient removal during aquifer storage, with DOC removal of 0.26 mmol/L (3.1 mg/L, ~50 %) in the outer wells (ASTR) at the end of the flushing phase and 0.10 mmol/L (1.2 mg/L, ~20 %) from the inner wells (ASR). Consumption of injected oxygen accounts for approximately 0.15 mmol/L (1.8 mg/L) of DOC oxidation, which is slightly greater than the removal seen via ASR. This may be attributed to some mineralisation of organic matter that has accumulated around the point of injection and is also illustrated in an increase in ammonium concentration (0.14 mg/L recovered *c.f.* 0.023 mg/L injected), masking some of the DOC removed via aerobic degradation. If the DOC removal during ASR is corrected for additions through nutrient recycling, it indicates an average of 0.18 mmol/L (2.2 mg/L) removal. This is consistent with consumption of all of the O_2 in the source water (on average 0.15 mmol/L) and the estimates of source water BDOC (2.4 mg/L). It is feasible that some of the DOC reduction achieved through ASTR is attributed to sorption onto iron oxides rather than removal through oxidation as estimates of source water reactivity are lower than that removed through ASTR.

During the initial stages of injection, oxidation of pyrite provides competition for consumption of the injected oxygen. While the pool of reactive pyrite in the sediments is limited, BDOC in the source water is considered to be unlimited unless the source water characteristics are altered through catchment management or pre-treatment steps. It is also possible that additional DOC removal can be achieved through reaction with Fe(III) within goethite or hematite in the aquifer sediments subsequently producing increased concentrations of iron. Overall there is a net removal of the total iron due to filtration of particulate material during injection, and a marginal increase in soluble iron (~0.1 mg/L) which may be due to reductive dissolution of iron oxides which would be expected to continue with subsequent injection. The soluble iron concentration in the recovered water is consistent in both the inner and outer wells but exceeds the aesthetic ADWG of 0.3 mg/L (NHMRC-NRMMC, 2004). To date there is no evidence for arsenic or trace metal mobilisation from the sediments.

Impact of mode of MAR operation – ASTR or ASR
This combination of calcite dissolution and ion exchange is evident during both types of MAR operation, ASR and ASTR, but the dominant cation released varies. A consistent degree of calcite dissolution through injection and recovery is indicated by the average amount of bicarbonate release during these periods (Table 2). After ASR, the excess in calcium (1.3 meq/L), exceeds that of sodium (0.3 meq/L) and magnesium (0.1 meq/L); while after ASTR the release of magnesium (0.9 meq/L) and sodium (0.8 meq/L) exceeds that of calcium (0.4 meq/L). The importance of ion exchange processes in IW1 diminishes within 2 pore flushes as the ratio of excess cations to anions approaching 1. This is also apparent for the water recovered from the inner wells (ASR), where multiple pore flushes have occurred (Vanderzalm *et al.*, in press).

The key difference in the nature of the MAR operation impacting on ion exchange is the number of pore flushes that have occurred as the effect of ion exchange on water quality declines with subsequent pore flushes (Stuyfzand, 1998; Vandenbohede *et al.*, 2008). In ASR the first water recovered is from the zone of the aquifer that has had the most flushing, while in ASTR the first water recovered has had the least flushing. With subsequent injection and recovery cycles in this carbonate system with a relatively low cation exchange capacity, the effect of cation exchange will ultimately be the same despite the nature of the MAR technique utilised.

The nature of the MAR technique, ASR or ASTR, also influences the impact of redox processes on water quality. The main redox process that impacts on recovered water quality is aerobic oxidation of injected organic matter, consuming the injected oxygen and reducing the DOC content. The injection well receives the greatest flux of nutrients and thus represents a zone of higher microbial activity which can be reflected by the development of a more reducing area than in the bulk of the storage zone where recycling of organic matter that has accumulated by filtration and adsorption can occur. Unlike ion exchange, the influence of redox processes on injected oxygen and DOC removal is not expected to reduce with subsequent pore flushes, but to continue as long as a nutrient source is available. Pyrite oxidation is observed during the initial stages of injection, affecting the water recovered via the ASTR mode only. The impact of pyrite oxidation is limited to the availability of reactive pyrite in the storage zone.

Enhanced DOC reduction is evident through ASTR where removal by sorption is equally as significant as oxidation, with a total DOC reduction of approximately 3.1 mg/L. DOC removal by sorption continues through the entire flushing period, in contrast to cation exchange which reduces in significance over time. Nonetheless the amount of DOC removal from the inner wells, after correction for localised DOC recycling suggests the impact of sorption on DOC removal does reduce with time and pore flushes and a net removal of 2.2 mg/L (35 %) is the ongoing extent of DOC removal expected through aquifer storage.

CONCLUSIONS
Calcite dissolution is the dominant influence on changes to the quality of recovered stormwater following storage in a carbonate aquifer. Initial freshening of the aquifer storage zone leads to cation exchange, increasing the sodium content for a short period during the early pore flushes. The magnitude of calcite dissolution exceeds that previously reported within the same aquifer, primarily driven by the nature of the source water and its reactivity with carbonate minerals.

Aquifer storage also provides passive treatment through nutrient and DOC removal. Initially the DOC removal is greatest when the MAR operation involves separate injection and recovery wells (ASTR). This is in part due to increased removal of nutrients via sorption in ASTR mode and in part due to DOC recycling around the injection well in ASR masking some of the removal through this mode. In this case the significant portion of the total DOC in wetland treated stormwater appears to be reactive. Injection of oxygenated source water induces oxidation of source water DOC and pyrite within the aquifer sediments and effectively removes all of the injected oxygen. In contrast to the oxidation of DOC, which is supplied continuously during injection, oxidation of pyrite is limited by the pool of reactive pyrite within the sediments.

With subsequent injection and recovery cycles in this carbonate system, the effect of cation exchange on major ion concentrations and adsorption on DOC will reduce as the capacity of surface sites is diminished. However the impacts on nutrient cycling due to the microbially active zone around an ASR well will continue as long as the nutrient source is supplied.

Water quality differences between the two MAR techniques were transient with water quality differences becoming negligible after a few pore volumes of injected water. The initial differences in water quality were generally a reflection of the different volumes of aquifer sediment the injected water passed through during ASTR and ASR.

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Physical, geochemical and microbial processes induced during the aquifer recharge using treated wastewaters: laboratory and pilot experiments and numerical simulations

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Abstract

In order to transform wastewater into new water resources with specific and appropriate uses (irrigation, stop saltwater intrusion, etc.) several concepts and scenarios for artificial recharge of deep aquifers have been proposed. Even if this new technology can help supply water to several parts of the world with water shortages, it is necessary to understand and control the physical, physico-chemical and bacterial processes conditioning the fate of any recalcitrant organic and inorganic pollutants contained in treated wastewater and / or surface water of poor quality.
In order to study these biogeochemical processes, a pilot study with unsaturated hydrodynamic has been elaborated. The availability of pilot results (monitored hydraulics and geochemical parameters) over a period of eighteen months will be presented coupled with an interpretative modelling approach incorporating the mechanisms of transport and reactions of dissolution - precipitation of minerals, degassing and geobiochemical processes. Current results show a marked decrease of nitrate concentrations without significant evolution of sulphates. The expected results of the pursuing experience, with changes in temperature and perhaps of the feed water chemistry, should provide important results for the development of an integrated approach.

Keywords Unsaturated zone (UZ), reactive transport modelling, geobiochemical processes, hydrodynamic modelling, SAT, Artificial Recharge of Aquifers.

INTRODUCTION

Wastewater treatment is necessary for the protection of the environment and public health. These treatments aim to reduce, among other pathogens, heavy metals, and potentially dangerous organic compounds. Reuse of wastewater is becoming a technological answer for remediation in order to address the water scarcity and increased population in the world, especially in arid and semi-arid regions.

Indeed, this research aims to develop/adapt numerical tools to estimate the purification capacity of natural soils filtering the wastewater. The results will provide a better understanding of the interaction and feedback mechanisms between recharge water and the unsaturated zone (UZ). The study of reagent transfer mechanisms through the UZ is a prerequisite for the development of a proposed wastewater reuse. A pilot study consisting of a column of soil was built to analyze the scale of displacement of reactive fronts, their amplification or mitigation. The column is 4 meters high with a diameter of 3 m. It is fully insulated from the ground to comply with regulatory constraints, and filled with a characteristic soil of the concerned site. The top of the column is open to local climatic conditions to simulate a real operational site in terms of temperature, rainfall, etc. In order to monitor the flow and changes in water content, the column is equipped with a set of 5 tensiometers, 5 water content probes, and 5 thermometers with an hourly measurement frequency. The rainfall and temperature are monitored on a daily basis. A model of flow, mass, and energy transfer through the vadose zone has been developed to capture the hydrodynamics of the column.

This project aims to develop tools and methodologies to study the purification capacity of natural soils and to better understand the process of interaction between recharge water and the vadose zone water (UZ). Indeed, the UZ is an area of intense physical, geochemical, and biogeochemical reactivity between aqueous solutions flowing through a specific type of soil. This interaction may lead to a strong retention of contaminants by physico-chemical, mineralogical, and biogeochemical characteristics of the vadose zone/recharge water system. A detailed study of the mechanisms of reagent transfer from recharge water through the UZ is the first step for developing a draft wastewater reuse plan (Azaroual et al. 2008; Azaroual et al., 2009). The common aim of various existing monitoring systems is to monitor physical, chemical and biological reactive transfer of effluent through unsaturated soil water. The adopted approach based on the semi-industrial scale is intended to better understand the complex reactions of mass exchange and flow of “hotspots” to taken into account in the numerical simulation of a process commonly called SAT "Soil Aquifer Treatment" (Idelovitch and Michail, 1984, Wilson et al. 1995; Drewes et al., 2003).

The pilot study results over a period of eighteen months highlighted the main physical and geobiochemical processes occurring in a soil column used to study SAT. The expected results of the experiment, such as changes in temperature and perhaps of the feed water chemistry, will provide important results to develop an integrated approach.

METHODS

Treated wastewater is supplied according to a daily volume measured several times daily to ensure the steady state of the multiphase system (solid - liquid - gas – “micro-organisms") within the reactor. Since June 3, 2009, daily intake was set at 1000 L/day divided into four tanks of 250 L each. The monitoring was automated by a system of control-commands and the results were recorded on a data logger monitoring physical, physico-chemical parameters tracing the reactive transfer of treated wastewater through the unsaturated soil column. The understanding of elementary processes of reactive transfer through the UZ and the validation of numerical simulations that will be derived, based on recent theoretical approaches, provide the basis for demonstrating the health and environmental safety of recharge by treated urban wastewater.

Experimental Soil Column

The objective of this project was to optimize the load of recycled water through a soil column to reduce the costs of tertiary treatment for artificial recharge based on the power of self-purification of a soil. This scale study of a soil column (Figure 1) allowed analysis of the development of reactive fronts at a scale between small laboratory analyses and the industrial recharge site monitoring of the behaviour of reagents fronts. This pilot will lead to the identification of hotspots i.e., the mineral assemblages buffering the pH and Eh, potentially active electron donors or acceptors (e.g. organic matter, NH_4^+, O_2, $SO_4^=$, NO_3^-…) according to the nutrient content, and the redox sensitive chemicals contained in the recycled water.

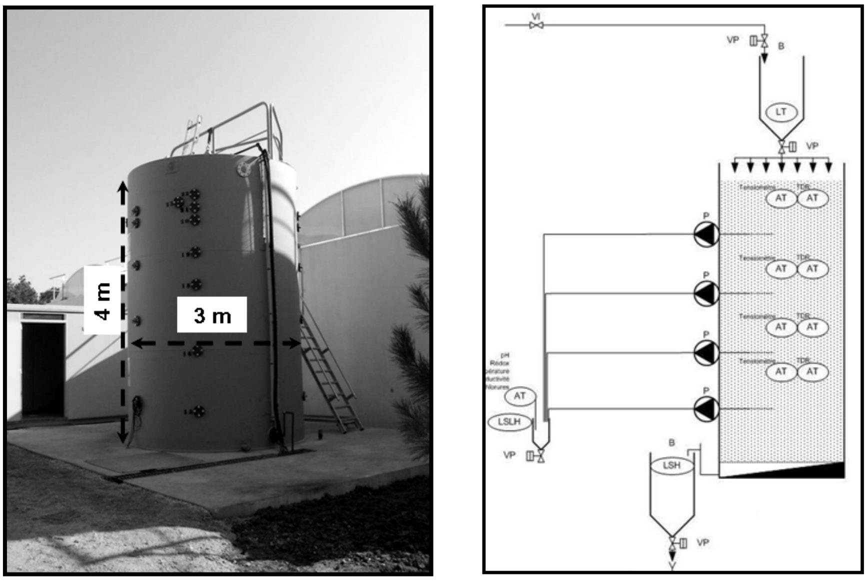

Figure 1 The experimental pilot dimensions showing the water, soil, and gas sampling points at different depths.

The very detailed and complete monitoring program was established for 18 months. The processes focused on are: physical modifications of porous material, chemical and biological reactions induced in a reconstituted soil, and microbiological and chemical composition of the flowing water. The experiment contained in situ high-resolution monitoring system for evaluating biological and geochemical composition of pore waters. The feed water was supplied at a rate of about 1000 L/d; a dosing station injected a tracer into the feed water. The effluent percolated through the driver and was recorded before reaching the effluent system of the processing plant. The entire monitoring system was placed in the driver for the duration of the experiment. Five TDR (Time-Domain Reflectometry) sensors for measuring water content were positioned at different depths and connected to a data logger via a multiplexer to measure water content values. At the same depths, five tensiometers for measuring soil water pressure and temperature were positioned and connected to the data logger. Lysimeters were installed to take water samples from different soil depths to analyze physico-chemical properties of various parameters. The different sensors are remote from each other to avoid interference. The in situ monitoring was automated and continuous. Control parameters and sensor output were recorded on a data logger. Data was retrieved from the computer connected to the pilot study soil column.

Numerical modelling

The calibration and consolidation of the modelling of flow in the unsaturated zone is a prerequisite step before introducing the geochemical processes and simulations of the reactive unsaturated zone. Two kinds of numerical tools were used to allow the study of elementary processes and coupled and integrated phenomena. The hydrodynamic simulation code MARTHE (Thiéry, 1990) was used to study the water transfer through the unsaturated zone of the soil column. This code allows quantifying the water balance (evapo-transpiration, water retention in the soil, the outflow, etc.) and mass transport. The detailed mechanistic analysis of thermo-kinetic geochemical reactions, geobiological reactions, adsorption - desorption of metals at the mineral surface, degassing/bubbling of volatile chemicals, etc., was performed using PHREEQC and MIN3P (Parkhurst and Appelo, 1999; Mayer, 2002). To ensure the global consistency of the reactive transport modelling, the coupled code MARTHE-REACT is used (Xu et al., 2004; Thiéry et al., 2009).

RESULTS AND DISCUSSIONS

The hydrodynamic analysis of the unsaturated zone (UZ)

Detailed analysis of pressure and water content time series, in connection with the outflow variations, was used to select periods during which the measurements seemed to be reliable and had an interpretable pattern. Data from these periods, after smoothing to remove high frequency noise fluctuations, yielded a first estimate of the characteristic curves at five depths. Figure 2 displays the retention laws at 110 cm and 340 cm. Water pressure and water content relationship measured at two depths and corresponding homographic laws are reported in Figure 2.

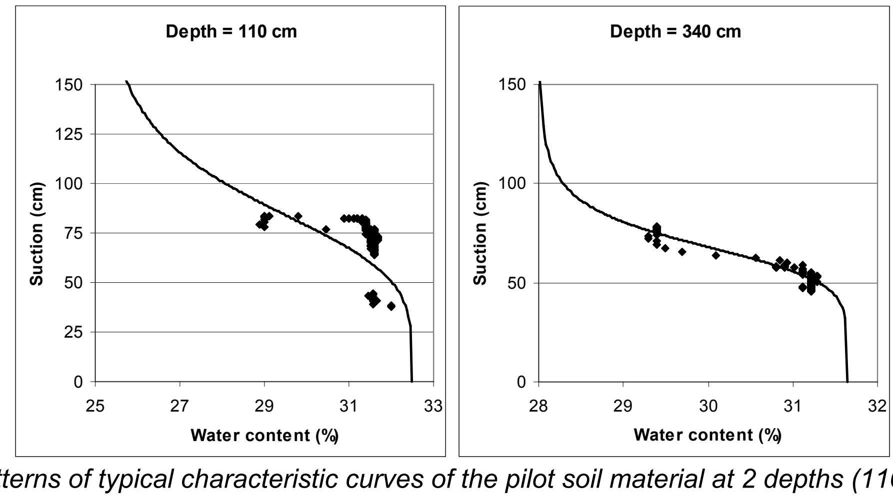

Figure 2 Patterns of typical characteristic curves of the pilot soil material at 2 depths (110 cm and 340 cm).

Using the characteristic curves determined from data obtained in large variations of saturation, and using reasonable permeability laws, a numerical model of the vertical dimension was developed for the period from January 12 to February 10, 2010, during which the flow was subject to many changes (Figure 3): The model first analyzed a stable period, then a stay of inflows for a period of five days following a simulated breakdown, recovery rates for 5 days, 10 days of internal drainage, and then a recovery of inflows for eight days. In the model, a water saturation condition was prescribed at the bottom of the pilot study soil column, and the monitored inflow was prescribed at the top.

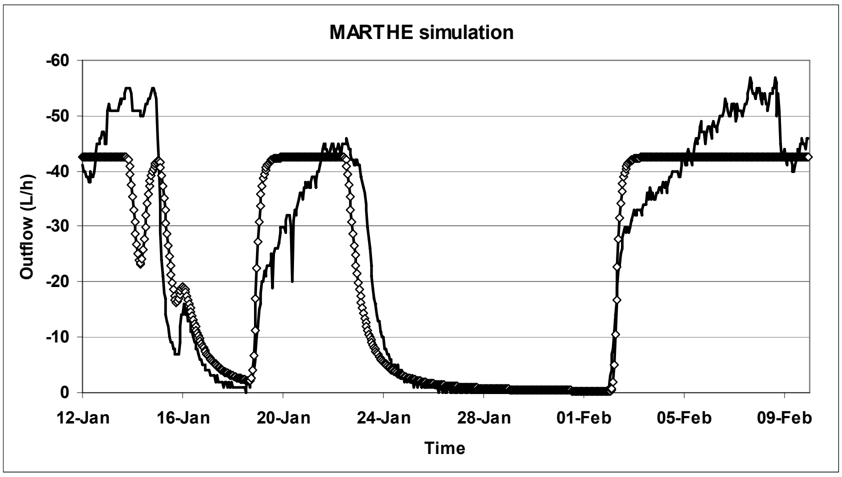

Figure 3 Simulation of flows in experimental column (pilot) with the code MARTHE for the period of January-February 2010. Observed (solid) and simulated (diamonds) outflows.

Figure 3 shows that the model can reproduce satisfactorily the outflows measured at the bottom of the soil column. Changes in water content, 1% near the bottom and 5% near the surface, (not shown) are also satisfactorily simulated... The model pressure variations (not shown) are slightly higher than those observed, which is not surprising, given the slight inconsistency between the changes of retention and the volume changes observed. The high clay contents in the soil leads to high water retention (around 25%).

Physicochemical and geochemical monitoring

The soil column is equipped with 42 sampling/measurement points to field sample an important suite of physico-chemical parameters (pH, T, Eh (Ag/AgCl), conductivity, dissolved O_2, NO_3^-, NO_2^-, Fe^{2+}). The physio-chemical analysis was complemented with laboratory analysis of cations and traces (Ca, Na, K, …, Se, As, Cr, Cu, Mn, Ni, Pb, Zn, B), anions (Cl^-, NO_3^-, NO_2^-, SO_4^{2-}, $S_2O_3^{2-}$, F^-, PO_4^{3-}, Br^-) and DIC, NH_4^+, DOC, $^{11}B/^{10}B$ ratio, and some reactive gases. The organic pollutants, pesticides, hormones, and some emergent pollutants were also measured. The evolution of the redox parameters (dissolved O_2 and Eh (mV)) along the pilot reactive column is shown in Figure 4 and Figure 5.

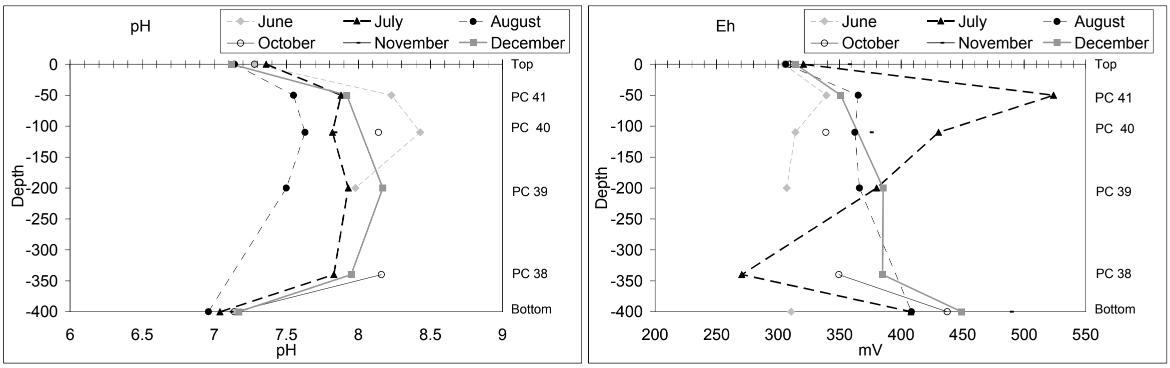

Figure 4 Evolution of the physico-chemical parameters (pH and Eh) during the first semester of functioning of the pilot (June to December 2009). PC = Lysimeters.

The pattern of these parameters highlighted the global dynamic of the reactive gas (O_2) and its impact on the redox state of pore water (Eh > 0). Knowledge of Eh is necessary to establish the distribution/stabilization of reactive redox zones, allowing understanding the redox-sensitive species repartition. However, it has been shown in many studies of natural systems (i.e., Lindberg and Runnels 1984; and references therein) that the measured Eh is still a global indication and indeed, the thermodynamic equilibrium is rarely achieved for many species sensitive to redox conditions (Van Cappellen and Wang, 1996; Appelo et Postma, 2005). The reason for these redox disequilibria is essentially due to the slow kinetics of the redox reactions. Moreover, these reactions are often strongly influenced by bacterial activity (Lipponen et al. 2002; André et al, 2007; Kremer et al., 2010).

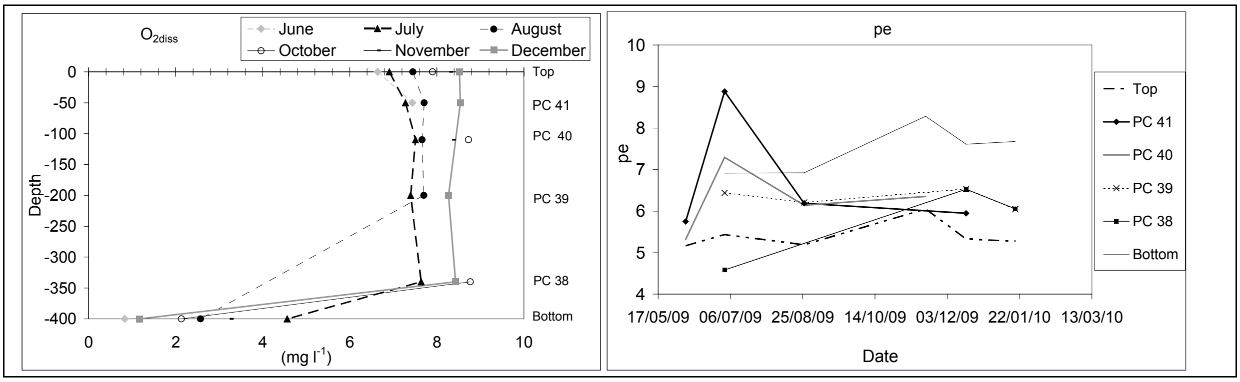

Figure 5 Pattern of pore water dissolved O_2 and potetiel redox (pe) measured at various points of the pilot from the inlet parts (feed water, top) to the outlet (outflow water, bottom). One do notice that dates on the figures are conform with the french convention: dd/mm/yy. PC = Lysimeter.

Generally, the microbial population of surface environments derives its energy from the decomposition of organic matter by redox reactions. The main mechanisms of organic matter degradation are aerobic respiration, denitrification, manganese reduction, iron reduction, sulphate reduction and fermentation. Metabolic processes consist of electron exchange processes decomposed in complex multi-step characteristics. The acceptors of the first 5 degradation mechanisms mentioned above, are O_2, NO_3^-, oxides of Mn (III, IV) hydroxides of Fe (III) and SO_4^{2-}. Fermentation in turn, is not connected to an external electron acceptor, but a partial reduction or oxidation of organic carbon substrate (Hunter *et al.,* 1998). Figure 5 demonstrates the reactive capability of the soil column and the relevant conditions of the system to maintain a reactive unsaturated zone through the entire height (4 m) of the column.

Thermodynamic driving forces and equilibrium state of the pilot soil column

Figure 5 highlighted the importance of redox potential (pe = (Eh) F/2.303RT), the complexity of redox reactions, and the disparity of kinetic constants in each step of the chain of reactions. It is therefore difficult to ascribe the behaviour of species (redox state) to the only response that integrates directly. Indeed, other reactions such as sulphate reduction occur. In addition, micro-organisms catalyzing biogeochemical reactions are often very specific to each reaction. The nitrogen cycle analysis is part of a whole network of reactions that would be characterized with the approach developed in this project (i.e., the implementation of complementary experiments: pilot and laboratory experiments). The pattern of the main aqueous nitrogen species (Figure 6) confirms the fast reactions related to the recharge of treated wastewater containing organic matter and some inorganic pollutants (NO_3^-, NH_4^+, As, etc.). Two series of experimental protocols were developed to study the kinetics and different steps of denitrification and nitrification reactions:

NO_3^- ===> NO_2^- ===> NO ===> N_2O ==X=> N_2 (this last step was inhibited using acetylene)

and

NH_4^+ ===> NO_2^- ===> NO_3^-

for which some intrinsic kinetic constant values were obtained. The remnant of geochemical reaction driving forces of the reactive column is highlighted by progressing denitification/nitrification reactions stipulated from the pilot geochemical monitoring results (Figure 6) and confirmed by more constrained laboratory experiment and modelling results (Kremer et al., 2010). The thermodynamic energetic of different steps was analyzed and expressed in terms of redox state for different steps of the nitrogen cycle compared to the measured redox potential (Figure 6). This thermodynamic diagnosis demonstrates the redox disequilibria between different and concomitant nitrogen aqueous species.

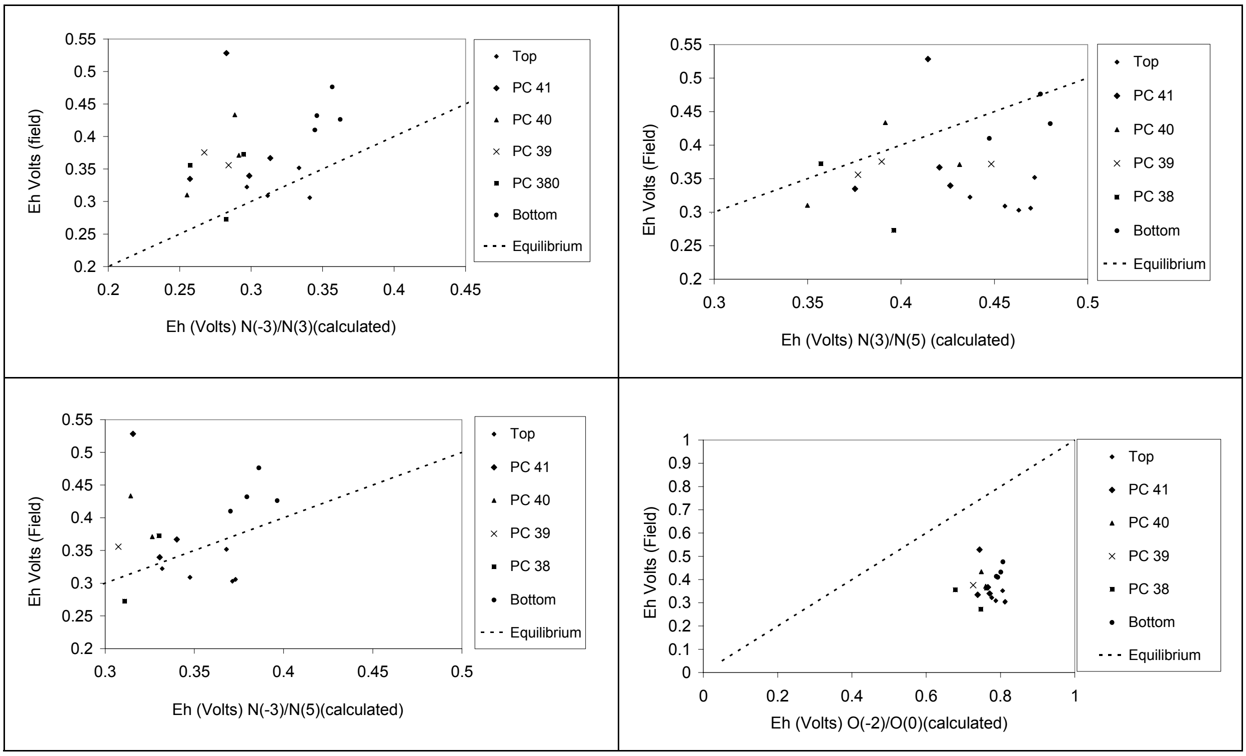

Figure 6 Computed redox potential (Eh) from the redox couples of nitrogen species based on their concentrations in the feed water compared to measured values. One do notice that dates on the figures are conform with the french convention: dd/mm/yy. PC = Lysimeter.

This part devoted to establish a thermodynamic diagnosis through analysis of some thermodynamic functions as the Saturation Index (SI) corresponding to the logarithmic ratio of ionic activity product (IAP) determined from water analysis and the equilibrium constant (K) representing the equilibrium state of an aqueous solution with respect a mineral. The IS function describes the deviation from equilibrium of a solution with respect of a given mineral and thus understands the degree of geochemical reactivity of the system and its driving forces to evolve under specific conditions. For SI = 0, the aqueous solution has reached thermodynamic equilibrium; SI <0 means that the solution is under saturated (favouring of dissolution), and SI> 0 corresponds to the supersaturation of the solution (favourable conditions for precipitation). The SI *vs.* time of calcite plotted at different depths shows that the pore solutions tend to be equilibrated with respect to calcite (Figure 7). The decreased amount of bicarbonate (alkalinity) in the waters of the soil column and the increase of pH confirms this trend. However, a deviation from this general behaviour is observed in December at the depth of 200 cm.

Saturation was also reached for magnesite over time. A slight supersaturation for this mineral was also observed at the depth of 200 cm in December. Given the margin of error which exists on the value of log K used to calculate the saturation indices of the different carbonate minerals, we can assume that the solutions are stable with respect to these minerals for values ranging from -1 to SI to 1. The carbonated water from the sewage treatment plant would allow precipitation in the soil column of carbonate minerals such as calcite. The aqueous solutions obtained at different levels of the column are under-saturated with respect of anhydrite, gypsum, strontium and celestite. Only barium sulphate could be supersaturated with respect percolating water through the column. This water is close to thermodynamic equilibrium with respect to quartz and chalcedony. The global tendency is rather dissolving mineral silicates during the hottest periods (lower SI of silicates) and to precipitate these minerals during the cold season (positive SI values). The high clay content in used soil material contributes to the phenomena of mass transfer between phases and can partially explain the saturation/supersaturated solutions with respect silica minerals. Finally, whatever the depth and the date considered, percolating aqueous solutions are systematically supersaturated with respect kaolinite, illite, smectite and muscovite (SI > 0).

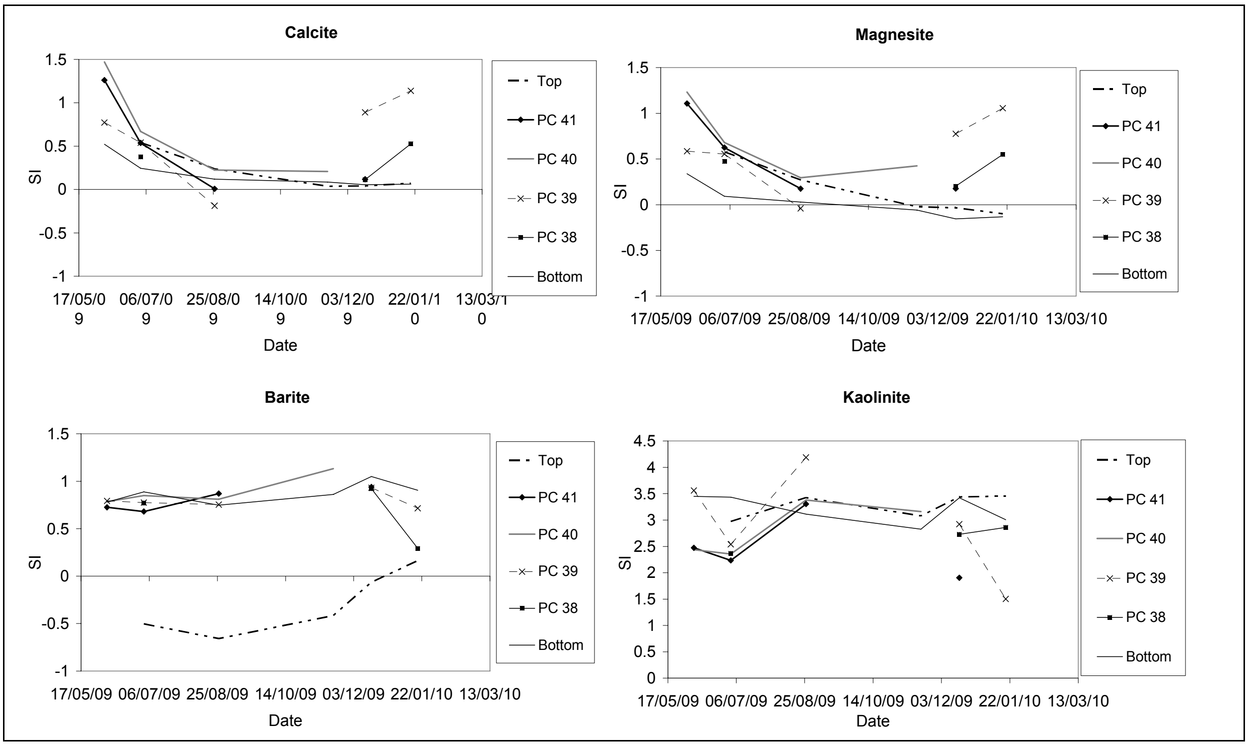

Figure 7 Evolution of the SI function of calcite, magnesite, barite and kaolinite at various levels of the pilot over a period of nine months. One do notice that dates on the figures are conform with the french convention: dd/mm/yy. PC = Lysimeter.

The purpose of presentation of the behaviour of different chemical elements through the reactive column is to achieve consolidation patterns and to examine the possibility of grouping some element families. These data will be integrated after using thermokinetics modelling of transfer process by including all relevant mechanisms and identified geochemical and biogeochemical reactions. We will use this data as observational data for comparison with models or as direct data for the evaluation of some kinetic parameters integrated in the modelling of complex and interconnected reaction networks. At this stage of project progress, only a global thermodynamic diagnosis is made allowing evaluating the system disequilibria highlighting the "driving forces" of the pilot scheme simulating artificial recharge according to the SAT concept and engineered reactive barriers. Additional analysis on the mineralogy of the pilot will be conducted at the end of experience to better constrain reactive transport models simulating the mass/heat transfer through the pilot column.

CONCLUSIONS AND PERSPECTIVES

The objective of this project was to produce models coupling flow in the unsaturated zone and reactive transfer based on experimental results. The concept of the pilot study was successful because the unsaturated zone water was almost stabilized over a period of one year and oxygen from the atmosphere pervades the entire column. The hydrodynamic parameters and laws of behaviour, water content versus suction of the reactive column, have been established. The geochemical reactivity and the phenomena of interaction and mass exchange between phases have been identified based of an extended monitoring program of physical and geochemical biogeochemical parameters over several months. The denitrification reactions catalyzed by bacteria respiration were highlighted through the pilot study and laboratory experiments. Thermodynamic analysis of column performance showed strong redox disequilibria and a tendency to precipitate carbonate minerals in some parts of the pilot study soil column. The silica concentration after the dissolution of aluminosilicate minerals and feed water could be involved in precipitation reactions, chalcedony, clay minerals, etc. but also in surface complexation reactions. The highly carbonated water, with intense biodegradation of organic matter, may generate some carbonate deposits (i.e., calcite, etc.). Additional analysis on the mineralogy of the pilot study soil column will be conducted at the end of the experiment to better constrain the numerical modelling of mass transfer through the water unsaturated soil column.

The “driving forces” of geochemical and biogeochemical reactions have been clearly identified. Work continues on the experiment to validate the coupled model processes involving geochemical and biogeochemical reactions and transfer of reactive gases in the unsaturated zone using numerical tools (MARTHE-REACT and MIN3P). It is also planned to analyze the mineralogical stratification to identify areas of high reactive (precipitation of minerals, clogging zones, biofilm development, etc.) at the end of the pilot experimentations.

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Arsenic behavior in SW Florida ASR systems and its expert modeling

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Abstract
Aquifer Storage Recovery (ASR) wellfields in karstified limestone have been operating at many locations in Florida since 1983. When in 2005 the water quality standard for As was lowered from 50 to 10 ug/L and elevated As concentrations showed up at some sites, ASR became a much disputed water storage technique. Pyrite oxidation during injection and storage was identified as the main driver behind the As problem, with subsequent As desorption from neoformed ferric (hydr)oxides (HFO) and reductive dissolution of HFO especially during recovery.
Data analysis revealed many complications in trying to simulate and predict As behavior with reactive transport models, due to scarcity of specific or high quality data. Therefore, considering the vast amount of hydrochemical data that have been collected on a routine basis on Floridan ASR sites, it was decided to construct an expert model for predicting As behavior in SW Florida ASR systems. This model is presented and tested against field data (from 6 ASR well fields with a total of 53 wells, each with 3-17 ASR cycles). It consists of an analytical model which calculates the As concentration by a sine function with decreasing amplitude during subsequent ASR cycles. The initial amplitude is determined by the As peak calculated via a mass balance equation which multiplies the average As content of pyrite with the total amount of pyrite oxidants in the recharge water.
The model predicts trends in As concentrations on any site, old or new, and the effects of changes in ASR operation, like reducing the oxidant input or recovery efficiency.

Key words:
ASR, arsenic, pyrite, limestone, expert modeling

INTRODUCTION

Aquifer Storage & Recovery (ASR) is a water management technique, in which water is stored in an aquifer during periods of water excess, and recovered by the same (injection) well from the groundwater reservoir during periods of water shortage (Pyne, 2005).

In Florida, ASR wellfields in karstified limestone have been operating at many locations since 1983. Initial indications of elevated As concentrations at some ASR wellfields (up to 220 ug/L) in SW Florida became evident in 2001. In 2005 the water quality standard for As was lowered in USA from 50 to 10 ug/L. Since then, ASR became a much disputed water storage technique, instigating several investigations (Pyne et al., 2004, 2008; Mirecki, 2004; Price & Pichler, 2006; CH2M Hill, 2007; ASR Systems, 2007; Arthur et al., 2007; Jones & Pichler, 2007), to better define the nature and extent of the problem, to advance understanding of the science of As mobilization and attenuation in the Floridan limestone aquifer and to seek potential solutions. Most investigations relied upon extensive operating and monitoring data from many ASR wells and a steadily increasing number of storage zone monitor wells.

Pyrite oxidation during injection and storage was identified as the main driver behind the As problem, with subsequent As desorption from neoformed hydrous ferric oxides (HFO) and reductive dissolution of HFO especially during recovery.

Nevertheless, many uncertainties remained due to the complexity of As behavior, heterogeneity of karstic limestones, and doubts about the quality or representativity of the hydrochemical data collected (Stuyfzand, 2008). Despite these and other important areas of uncertainty, clear patterns have become evident regarding arsenic mobilization and attenuation at Florida ASR wellfields (Pyne et al., 2008):

- Peak arsenic concentrations in the recovered water from ASR wells tend to decline with successive operating cycles, when the volumes for recharge and recovery are kept rather constant;
- Arsenic concentrations tend to decline during extended storage periods;

- Elevated arsenic concentrations do not extend very far from ASR wells, typically reaching acceptable concentrations within a radius of about 120 m;
- Formation and maintenance of an adequate buffer zone around an ASR well, separating the stored water from the surrounding ambient groundwater, tends to control arsenic concentrations within acceptable levels; and
- During ASR recovery, if over-recovery is allowed to occur and all or a portion of the buffer zone is recovered, the change in groundwater quality destabilizes arsenic sorbed onto ferric hydroxide floc and on other surfaces near the well, releasing the desorbed arsenic.

The improved insights stimulated to construct an expert model for predicting As behavior in SW Florida ASR systems (Stuyfzand et al., 2008). This model is presented here and tested against field data from 6 ASR well fields with a total of 53 wells (each with 3-17 ASR cycles). It constitutes a fast, easy to handle, predictive tool with a very low data hunger.
As future research will probably yield more detailed and better field data, the use of more sophisticated reactive transport models like PHT3D (Prommer et al, 2003) will become a better alternative.

MATERIAL AND METHODS

Field sites

The 5 studied ASR field sites are situated in SW Florida (Fig.1) and are composed of either a single ASR pilot well (Bradenton, North Port) or a set of 4-12 active ASR wells (Tampa, Peace River with 2 separate ASR well fields, and Punta Gorda). All ASR wells are open hole in the (semi)confined Floridan limestone aquifer system, at depths varying between 60 and 430 m below land surface (BLS). Characteristics of the field sites and wells are summarized in Table 1. Most ASR wells inject in the period July – December (some years continuing till March), when high quality surface water is available, and they recover the stored water in the period March – July. Injection and recovery rates are typically 40-260 m^3/h, with a recovery efficiency of 50-150%. On most sites there is 0.5-1 monitor well for each ASR well, at a distance of 25-162 m. These wells are open hole over about the same depth interval as their respective ASR wells (Table 1).

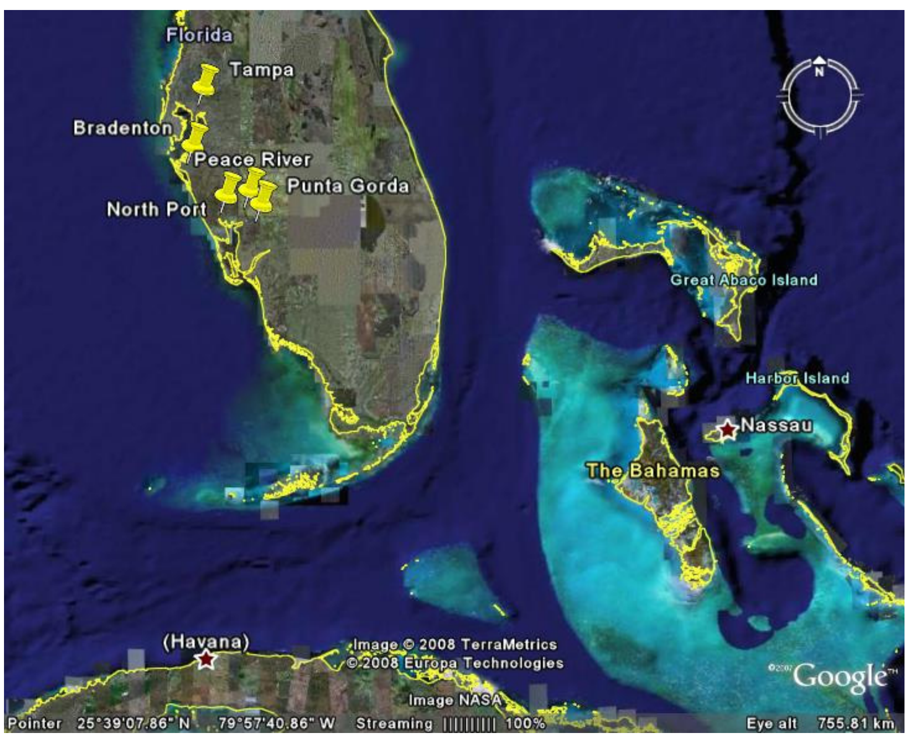


FIG.1. Location of the 5 studied ASR sites, with 2 separate ASR well fields at Peace River.

TABLE 1. Data on the target aquifer, the studied ASR and monitor wells, and the availability of water quality data.

ASR Site		Target aquifer			ASR well			Monitor well				Chem data	
		Name	Top	Base	No.	Top	Base	No.	Dist	Top	Base	for	since
			m BLS		n	m BLS		n	m	m BLS		Cycles	
1	Bradenton	Suwannee	122	183	1	127	154	1	68	123	168	1-7	2004
2	North Port	Suwannee	178	198	1	178	198	1	138	178 #	198 #	1-3	2002
3	Peace River #1	Suwannee	180	277	8	190	291	4	25-140	177	204	7-16	2004
4	Peace River #2	Suwannee	180	277	12	180	276	8	53-162	174	207	1-7	2002
5	Tampa RAP	Suwannee	91	131	8	91	122	3	46-107	91 #	122 #	1-9	1999
6	Punta Gorda	Suwannee	233	284	4	213	233	2	50-142	213 #	233 #	1-6	2002

\# = estimated

TABLE 2. Modal quality of the ambient groundwater in the UFA prior to ASR application, and of the infiltration water.

Par	Unit	Bradenton		North Port		Peace River WF#1		Peace River WF#2		Tampa RAP		Punta Gorda	
		ambient	input	ambient	input	ambient	input	ambient	input	ambient	input	ambient	input
EC	uS/cm	1400	529	2750	446	1428	469		469				673
TDS	mg/L	1200	330	3396	315	905	299	922	299			1959	429
pH		7.25	7.75	7.6	7.29	7.4	8.22	7.6	8.22	7.5	7.9	7.2	7.08
temp	oC	26.5	22.4	29.7	24.9	27.7	23.9		23.9				
ORP	mV	-330	385	-310		-300	448		448	-290			
O2	mg/L	0.02	9	0.1	6.9	0.9	5.7		5.7	0.45	20		
Oxidant			NH2Cl				NH2Cl		NH2Cl		O3		
H2S	mg/L	2.3		2.8	0.1	2.3							
As	ug/L	6.2	0.8	<8	<1	1	<1.3	<1	<1.3	4	<1	<2.2	3
Cl	mg/L	36	32	1490	43	224	32	167	32	39	31	865	98
SO4	mg/L	650	157	430	134	253	130	264	130	31.5	118	336	124
HCO3	mg/L	130	61	120	66	142	21		21	118	52	100	24
F	mg/L	1.8	1.0	1.8	1.0		1.0		1.0		1.0		
Na	mg/L	28	51	671	53	104	35		35				
Ca	mg/L	210	45	161	41	102	20		20		71		
Mg	mg/L	79	10	105	10	48	12		12		4		
Fe	mg/L	0.03	0.02	<0.1	0.03	<0.03	0.059	<0.012	0.059	0.02	0.02	0.137	
Mn	mg/L	0.0017		<0.01			0.006		0.006	0.01	0.01		
TOC	mg/L	1.4	2.9								2.7		

Hydrogeological and hydrochemical setting

On all 5 sites, ASR utilizes the Upper Floridan Aquifer (UFA), which is mainly composed of a (semi)confined, highly karstified limestone of middle Tertiary age, with some dolostone in the lower parts. Data on the geochemical composition of the ASR target aquifer indicate that arseniferous pyrite is abundant, especially along bedding planes, with an average content of 2,300 ppm As for pyrite, with extremes from 100 to 11,200 ppm (Price & Pichler, 2006).
Arthur et al. (2007) mention that the pyrite contains on average 1,300 ppm As, together with several other trace elements (notably Mo, Co, Ni, Sb and U).
The quality of the ambient groundwater in the UFA and of the infiltration water are shown in Table 2. The ambient groundwaters are (deeply) anoxic, in many cases with high SO_4 due to dissolution of gypsum, and with low Fe and As but high H_2S concentrations testifying of active pyrite formation. The infiltration waters are mostly composed of drinking water prepared from surface water, with additions of a disinfectant (extra oxidant) and sometimes F and/or $ZnHPO_4$. They are (sub)oxic and contain less TDS than the ambient groundwaters.

Hydrochemical monitoring during ASR operations

Hydraulic and hydrochemical data have been collected at all sites following regulatory requirements by the Florida Department of Environmental Protection (FDEP). Measurements of pH, O_2, ORP (Oxidation Reduction Potential) and temperature were done in the field, all other components in certified laboratories. Cations (including Fe) and As were analyzed in samples that were acidified to pH 1.5 by adding suprapure HNO_3 without prior filtration. Thus, some suspended fines may have contributed to the concentrations of especially Fe-total and As-total.
In most cases the analytical programme consisted of weekly samples of the injection water and half-weekly samples of the recovered water from each ASR-well, with analysis on Total Dissolved Solids (TDS), Electrical conductivity (EC), O_2, ORP, pH, temperature, Cl, SO_4, HCO_3, Fe and As. On some sites Fe-dissolved, Na, Ca, Mg, U, F, total sulfur, TOC, TTHMs, HAAs, $^{226/228}Ra$ and gross alpha were measured as well. In the open source FDEP data files it

is mentioned for each sample, under which ASR setting it was taken (during injection, storage or recovery), during which cycle number and at which total injection or total recovery volume.

OBSERVATIONS

Observations on EC and As are shown for Bradenton's ASR and monitor well in Fig.2. They show simultaneous As and EC peaking some time after the start of pumping, in both the ASR and monitor well. This indicates that the arrival of higher salinity water (with raised SO_4, Cl and H_2S concentrations) from the mixing zone and beyond, triggers the As mobilization, because the ambient groundwater contains much less As (6 ug/L).

Only during the longer injection periods 5-7 the infiltration water with lower EC clearly arrived in the monitor well, after about 15 days. Arsenic peaking is relatively small in the ASR well during recovery phase of the very short cycles 1-4, but much higher during recovery phase of cycle 6 when it also showed up in the monitor well, although at a lower level. This lower peak in the monitor well (20 ug/L) during recovery phase of cycle 6 indicates that As is mobilizing most in between the monitor and ASR well. The peak in the monitor well shows, however, that the As mobilization front extends beyond 68 m. Another important observation is that the As concentration declined during the long storage phase of cycle 6. This shows that mobilized As needs time to sorb to neoformed HFO or to settle as a particle suspended in the infiltration water.

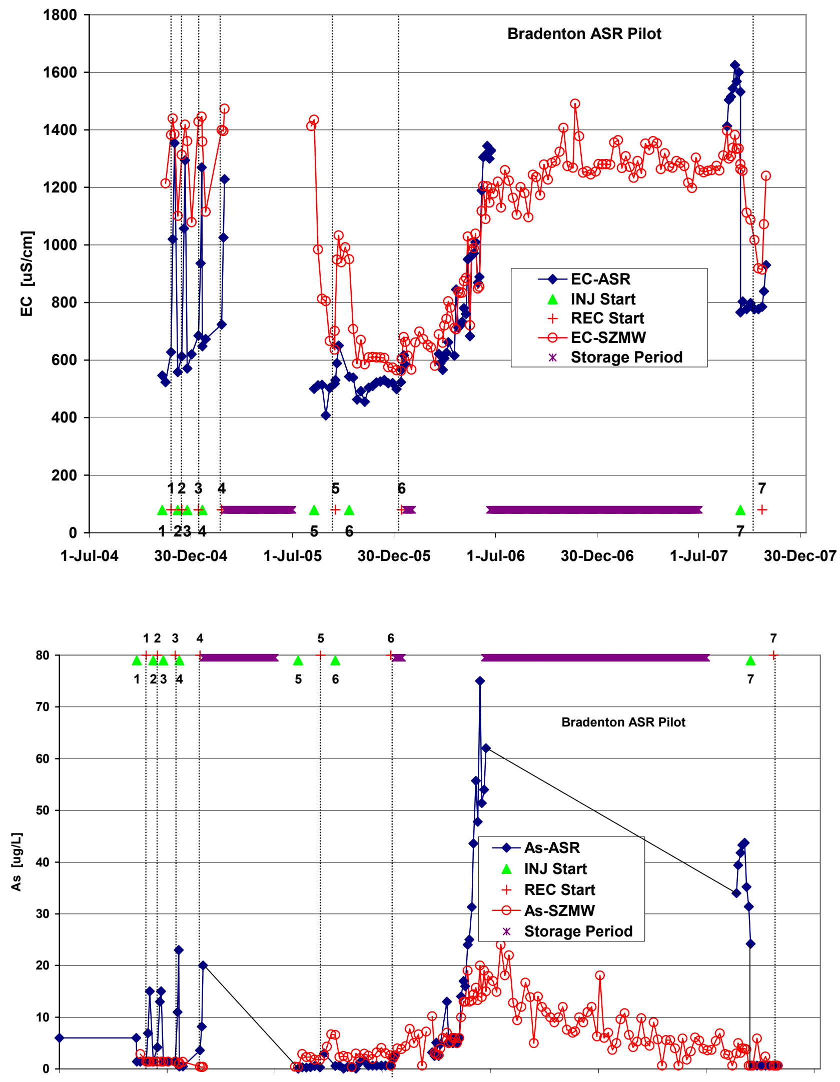
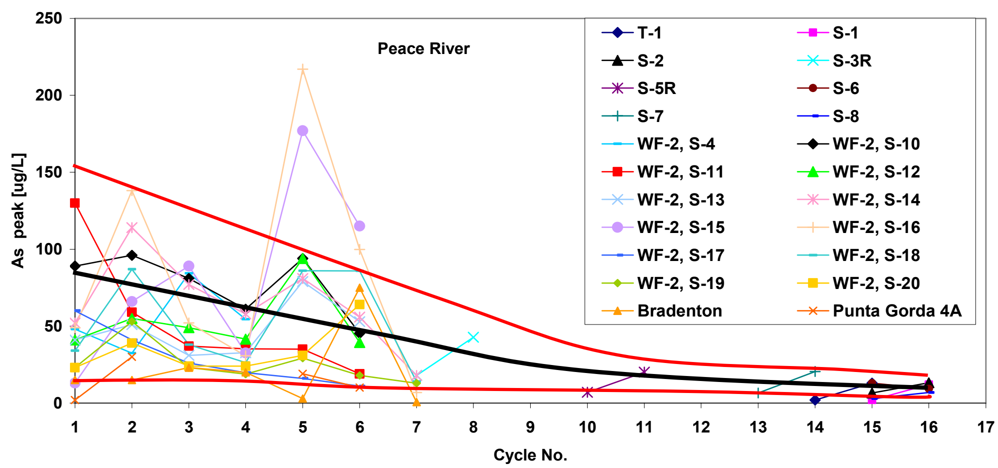


FIG. 2. Electrical Concuctivity (EC) and As concentration in the ASR well and monitor well SZMW, during cycles 1-7 on ASR site Bradenton. The vertical lines indicate the start of the recovery phase of the numbered ASR cycle.

The observations on 6 ASR well fields, with a total of 53 wells (ASR and monitor wells), each with 3-17 ASR-cycles, show a general tendency of declining maximum As concentrations during recovery, with increasing ASR cycle number (Fig.3). This corresponds with world-wide observations and accepted theory that arsenic becomes less mobile in increasingly more oxygenated environments when depleted in pyrite, due to decreasing chances on reductive dissolution of hydrous ferric oxides (HFO) that captured the mobilized As, and due to oxidation of uncharged, more mobile arsenite (H_3AsO_3) into charged, less mobile arsenate ($HAsO_4^{2-}$ + $H_2AsO_4^-$).
As can be seen in Fig.3, there are rather wide 'optical' confidence limits flanking the general trend (see ad discussion). Bradenton and Punta Gorda fit reasonably well within the pattern of Peace River, and the mean trends of Peace River and Tampa do not deviate much.
The ASR well in North Port did not follow the general trend at all. During the 3 investigated cycles there was a steady increase in As peak value, and the peak started in the first backflush during recovery. We suspect that arseniferous pyrite particles detach from the aquifer matrix (limestone or dolostone) upon its dissolution in rather aggressive infiltration water, and that these particles are thus included during sampling (which is without filtration) and dissolve in nitric acid added for sample conservation and total extraction. It is well known that the amount of suspended matter is highest during the first flush of a new pumping cycle (Stuyfzand et al., 2002).



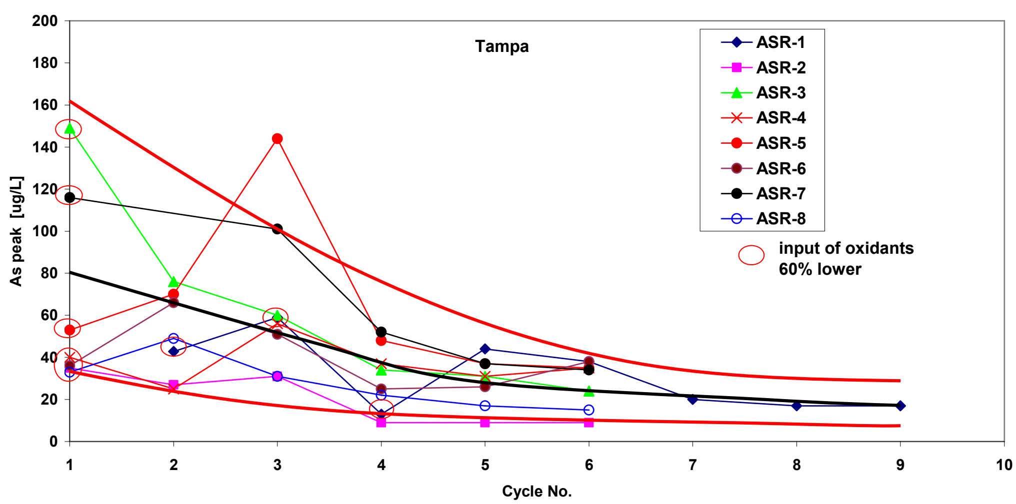


FIG. 3. Trends in As peak concentrations during ASR cycles 1-16 at Peace River (both well fields), Bradenton and Punta Gorda (above), and during ASR cycles 1-9 at Tampa (below). Heavy black line = mean trend; heavy red lines = resp. upper and lower 'optical' confidence limits.
There was a ca. 60% lower input of oxidants during cycles 1-2 for ASR-wells 2-8, and during cycles 1-4 for ASR-well 1, on well field Tampa.

TABLE 3. Spread sheet mass balance model for calculating the composition of the water injected after full reaction with pyrite, carbonate minerals (calcite and dolomite) and organic material (either dissolved as TOC or in solid aquifer phase as CH_2O; here O_2 assumed to be the only oxidant). Example is Bradenton's ASR well.

												0.0037					
	INFILTRATION WATER QUALITY + MAXIMUM PYRITE REACTION			48	70.906	51.453	32	35.45	96.06	61.02	62	74.922	40.08	24.31	18.0	12.0	44.0
	Reaction equation		**Fraction**	**O3**	**Cl2**	**NH2Cl**	**O2**	**Cl**	**SO4**	**HCO3**	**NO3**	**As**	**Ca**	**Mg**	**NH4**	**TOC**	**CO2**
			reacting	**mg/L**	**mg/L**	**mg/L**	**mg/L**	**mg/L**	**mg/L**	**mg/L**	**mg/L**	**ug/L**	**mg/L**	**mg/L**	**mg/L**	**mg/L**	**mg/L**
						4	8.5	32	157	61	1	1	45.4	9.5	0.02	2.9	
	mmol/L			0.000	0.000	0.078	0.266	0.903	1.634	1.000	0.016	0.00001	1.133	0.391	0.001	0.242	0.000
Org.	O2 + [TOC or CH2O] --> CO2 + H2O	TOC-fraction	0.2				-0.048									-0.048	0.048
Pyrite	3.75O2+FeS2+4HCO3 --> Fe(OH)3+2SO4+4CO2+0,5H2O						-0.108		0.058	-0.116		0.00011					0.116
Pyrite	3NO3+FeS2+HCO3+H2O --> Fe(OH)3+2SO4+CO2+1,5N2								0.047	-0.024	-0.071	0.00009					0.024
Pyrite	2O3+FeS2+4HCO3 --> Fe(OH)3+2SO4+4CO2+0,5H2O			0.000					0.000	0		0.00000					0.000
Pyrite	7.5Cl2+FeS2+19HCO3 --> Fe(OH)3+2SO4+15Cl+19CO2+8H2O				0.000			0.000	0.000	0.000		0.00000					0.000
Pyrite	7.5NH2Cl+FeS2+4HCO3+7H2O --> Fe(OH)3+2SO4+7.5NH4+7.5Cl+4CO2					-0.078		0.078	0.021	-0.041		0.00004			0.078		0.041
	2O2 + NH4 + 2HCO3 --> NO3 + 2CO2 + 3H2O	NH4-fraction	0.7				-0.109			-0.109	0.054				-0.054		0.109
Carb	CO2 + H2O + CaCO3 <--> Ca + 2HCO3	CO2-fraction	0.5							0.338			0.169				-0.169
Carb	2CO2 + 2H2O + CaMg(CO3)2 <--> Ca + Mg + 4HCO3	CO2-fraction	0.5							0.338			0.084	0.084			-0.169
	Output, after reaction of infiltration water with aquifer matrix		mmol/L	0	0	0	0	0.980	1.760	1.386	0.000	0.00025	1.386	0.475	0.024	0.193	0.000
			mg/L	0.0	0.0	0.0	0.0	34.8	169.1	84.6	0.0	18.4	55.6	11.6	0.4	2.3	0.0

MODELING

The analytical, semi-empirical formula

The following semi-empirical formula describes both the general trend of declining As peaks during successive cycles in the ASR well during recovery (as shown in Fig.3; sin() = 1), and the As concentration over time within each recovery cycle (Fig.4):

$$As = f\, As_{MAX}\, e^{-N/Y} \sin(0.9\, \pi/180\, [\alpha\, \%REC_{t,C\#X} + b]) \qquad (1)$$

With: As_{MAX} = Maximum As concentration as calculated with the mass balance approach [µg/L]; N = Cycle No.; $\%REC_{t,C\#X}$ = percentage of water recovered at time t = t during cycle N, of water infiltrated during cycle N [%]. Value can become >100%; f = fit factor for As peak level (3.5 on average); Y = fit factor for As peak decline with increasing cycle number (5 on average); $\alpha = a/\sqrt{(2N)}$; a = fit factor to regulate the sine frequency (3 on average); b = fit factor to regulate sine phase shift (often 0).

In Eq.1 the factor 0.9 π / 180 is needed to make sin( ) = 1 when %Rec = 100% (and α = 1, and b = 0).

If sin() ≤ 0, then the As concentration equals the natural background (ca 1 ug/L) or concentration in infiltration water (ca 1 ug/L).

By taking the best fit factors for the average situation, Eq.1 reduces to:

$$As = 3.5\, As_{MAX}\, e^{-0.2N} \sin(0.06664\, \sqrt{(N)}\, \%REC_{t,C\#X}) \qquad (2)$$

And when only the peak value during each cycle is needed, Eq.1 simplifies further to:

$$As = f\, As_{MAX}\, e^{-N/Y} \qquad (3)$$

Calculating As_{MAX} by a mass balancing

The mass balance calculation method of As_{MAX} is indicated in Table 3. It is assumed that all oxidants (O_2, NO_3 and the added disinfectant: O_3, NH_2Cl or Cl_2) react with pyrite, after subtracting a part for internal oxidation of injected NH_4 and TOC. The oxidation of organic material in the aquifer matrix has been neglected (which is reasonable in a limestone aquifer). The part of NH_4 and TOC oxidized is based on observations in the water recovered in various systems.

It is also assumed that the fraction of As in pyrite is known and released stoichiometrically without losses due to As sorption, coprecipitation with neoforming $Fe(OH)_3$ or other reactions. Thus, for pyrite the following average composition has been fixed: $FeS_{1.9963}As_{0.0037}$. This corresponds with an average arsenic content of pyrite in the Suwannee limestone aquifer of 2,300 ppm as observed by Price and Pichler (2006). However, this value may need adjustment, by changing either the coefficient 0.0037 or the value of f.

In a simplified form, As_{MAX} [in ug/L] can be directly calculated (without using a spread sheet mass balance calculation as shown in Table 3), namely as follows:

$$As_{MAX} = 74922\,[As_{IN} + As_{FeS2}\,\{(O_2 - c\,TOC - 2d\,NH_4 - 2e\,NH_2Cl)/3.75 + O_3/2 + Cl_2/7.5 + NH_2Cl/7.5 + (NO_3 + d\,NH_4 + e\,NH_2Cl)/3\}] \quad (4)$$

Where: all concentrations (except for As_{MAX} and As_{FeS2}) refer to the infiltration water [mmol/L]; As_{MAX} [ug/L]; As_{IN} = As in infiltration water [mmol/L]; As_{FeS2} = As content of pyrite on a mol basis, here fixed at 0.0037; c,d,e = fractions of input concentration [0 - 1], with standard setting as shown in Table 3.
In Eq.4 it is assumed that TOC and NH_4 (also deriving from NH_2Cl) are oxidized exclusively by O_2. This is not realistic as the disinfectants will be partly involved in their oxidation as well, but it yields a practical approximation of the loss of oxidative strength of the input. It is also assumed that NH_4 deriving from input NH_4 and NH_2Cl (if present) is first partly nitrified, and thus generated NO_3 is subsequently denitrified by pyrite.
Data of recovered water suggest that the following values can be maintained in the FAS: c = 0.2, and d = e = 0.7. Now there still are 4 fit factors to deal with: f ,Y, a and b. The factors f and Y determine the As peak concentration trend during successive ASR cycles. They are more important than a and b which are needed to get the sine wave in place within the recovery phase of an ASR cycle. Factor b can even be skipped in most cases, but a value of 50 is performing best for Bradenton.

Arsenic during storage
During storage As concentrations generally decline (Fig.2). The following relation is used:

$$As_t = As_0\, e^{-Z\,t} \quad (5)$$

With: t = storage time [d]; Z = rate of concentration decline [1/d]; As_0 = As concentration at the end of injection or recovery, prior to storage [ug/L];
At the Bradenton site Z is about 0.006, at Peace River 0.012. When recovery is resumed after an intermittent storage phase (separating 2 recovery phases within the same cycle), then Eq.2 is applied again, continuing with the earlier %Rec, however, subtracting ΔAs from each calculated value within that remaining cycle, where

$$\Delta As = As_0\,(1 - e^{-Z\,t}) \quad (6)$$

Results of application
The arsenic behavior of various ASR wells has been successfully modeled with Eq.1. An example is shown in Fig.4. Such a good fit could not be realized for all ASR wells, which is not a big surprise (see discussion).

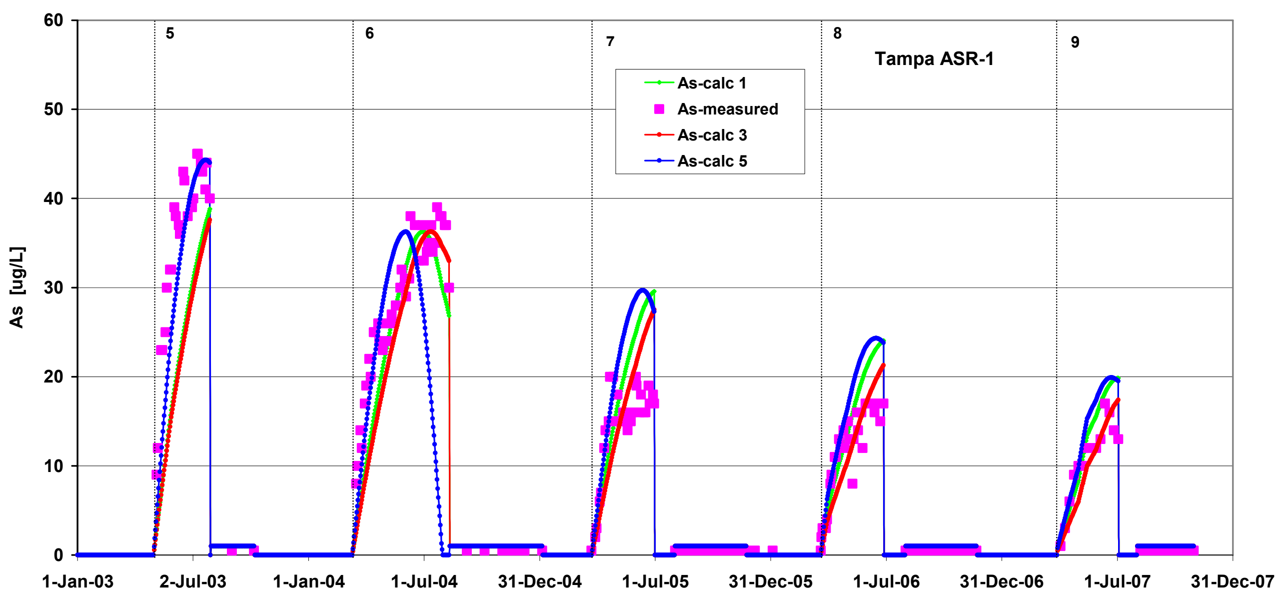


FIG. 4. Arsenic modeling results for Tampa ASR well No.1. using Eq.1 with f = 3.5, As_{MAX} = 34.3 ug/L, b = 0. α = 1 for calc 1; α = 3√(2N) for calc 3; α = 5√(2N) for calc 5. Best simulation for calc 3.

DISCUSSION

The expert model here presented yields satisfactory results (0.5 As_{MEAS} < As_{CALC} < 1.5 As_{MEAS}) in about 50% of all 144 cases (Stuyfzand et al., 2008). Much better is, of course, the prediction of the general trend of declining peak arsenic concentrations in the recovered water from ASR wells with successive cycles (Fig.3).

A better performance is logically handicapped by the following:

- The pyrite content of the limestone and the As content of pyrite vary in space. Subsequent ASR cycles will normally address an increasing aquifer volume, which may therefore show different contents;
- The As content of pyrite varies also with time because the most reactive pyrite, the composition of which may differ from the average pyrite, will oxidize first;
- Water samples have not been filtered, so that suspended arseniferous pyrite particles or arseniferous iron(hydr)oxide flocs will dissolve upon addition of HNO_3 for analysis on total concentrations. In all cases the infiltration water is dissolving limestone, which contains very finely dispersed pyrite particles, and may thereby mobilize particles containing As;
- The amount of participating suspended arseniferous pyrite particles is probably subject to fluctuations during pumping, thereby causing significant noise in the As data;
- Effects of upconing or buoyancy of the injected ASR-water body cannot in all cases be distinguished from the effects of lateral flow of ambient groundwater;
- Total arsenic analytical methods may be biased by matrix effects (Cl, SO_4) and incomplete conversion of arsenate into arsenite (Van der Jagt & Stuyfzand, 1996). Results can be positively and negatively biased.
- Water quality of the input is ill defined because of lack of data on its total oxidation capacity (O_2, NO_3, O_3, Cl_2, NH_2Cl), internal consumption of oxidation capacity (NH_4, TOC) and anion desorptive strength (DOC, PO_4, F, SO_4, HCO_3, SiO_2);
- Not all oxidants in the water infiltrated react with pyrite; a yet unknown part will be consumed by organic matter in the UFA;
- Use of HCl or CO_2 for improving well yield or infiltration capacity. This may have 2 important consequences: (a) the carbonate rock is dissolving while pyrite grains detach from their matrix and become either suspended and transported away, or they sediment; and (b) iron monosulfide may dissolve as follows: $2HCl + FeS \leftrightarrow H_2S + Fe^{2+} + 2Cl^-$. The latter does not happen with CO_2 additions, which appears to be more common practice.
- Heterogeneity of the limestone, both in hydraulic conductivity and geochemistry.
- Mixing of different water qualities due to too long well screens; and
- Effects of neighbouring wells (especially Peace River WF#2) and bubble drift are not included.

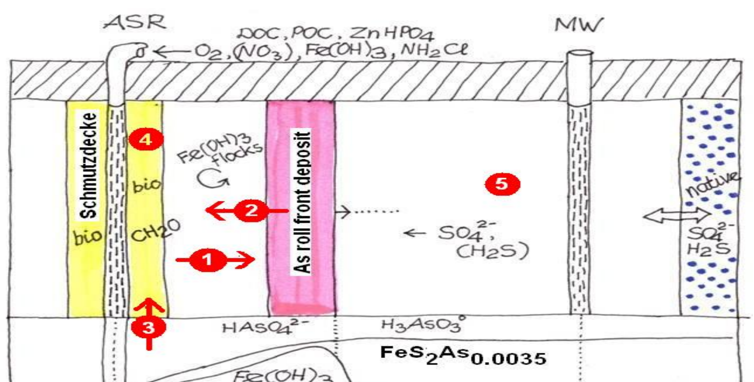


FIG. 5. Conceptual model of arsenic peaking during backpumping (example Bradenton), with zonation of various processes that mobilize and immobilize As during ASR-operations in Florida.

1: pyrite oxidation during injection, with mobilization of As (and SO_4);

2: As remobilization during recovery, by either reductive dissolution of $Fe(OH)_3$ or desorption;

3: upconing of brackish groundwater with high Cl, SO_4 and H_2S concentrations, of which SO_4 could desorb arsenate and H_2S could redissolve $Fe(OH)_3$ or convert As(V) into As(III);

4: during storage lack of nutrients may lead to anoxic conditions, remobilizing As;

5: In the anoxic buffer zone mixing with ambient groundwater may lead to supply of H_2S and SO_4 which favor As remobilization during pumping.

The expert model can be used to predict the effects of changes in ASR operation in order to bring the As concentrations back into an acceptable range. Changes like reducing the oxidant input (which reduces the parameter As_{MAX} in Eqs 1-4) or reducing the recovery efficiency (which lowers the sine value) have a direct impact on the calculated As output via Eqs 1-4.

CONCLUSIONS

With the analytical solutions presented, the arsenic peak concentration in the water recovered during each ASR cycle can be predicted. However, it should be realized that the resulting information has a rather high inaccuracy when local hydrogeochemical test data are lacking.

Further research is needed to provide better data that are unbiased by among others analytical errors (O_2, As), problems associated with observation wells having long open hole sections, and suspended particles (like arseniferous pyrite grains and ferrihydrite flocks). Also, a broader scan of water quality is needed, including all oxidants (also those added for disinfection) and important As desorbents (like PO_4, HCO_3, F and SiO_2).

Better data are a prerequisite for evolving from the expert model given here, to a well calibrated, true reactive transport model also for arsenic, like PHREEQC-2 or PhT3D. The success of the latter model has been demonstrated by Wallis et al. (in press), in simulating As behavior during Aquifer Storage Transfer Recovery (ASTR, using injection and remote recovery wells) in the Netherlands. In preparing reactive transport models the many processes acting in different zones indicated in Fig.5 need to be considered.

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Multitracing of artificially recharged Rhine River water in the coastal dune aquifer system of the Western Netherlands

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Abstract
The coastal dunes of the Western Netherlands are recharged by pretreated Rhine River water on a large scale since 1955, for drinking water supply. Ecological interests and both national and EU legislation compel the water utilities to monitor the expansion of the resulting artificial groundwater bodies amidst the surrounding, natural dune groundwater. However, multitracing techniques are needed to unambiguously identify the infiltrated Rhine water amidst coastal dune groundwater, because of overlapping tracer contents. In addition, the identification is becoming more difficult due to ongoing trends that reduce the contrast between both waters.
The performance of 28 environmental tracers is shown, both single and in various combinations (multitracing), for diverse hydrogeochemical compartments of the aquifer system. To that purpose 165 samples of pure Rhine infiltrate are compared with 150 samples of pure dune groundwater, both taken from (sub)oxic to deeply anoxic, from coarse sandy to clayey, and from Holocene to Pleistocene deposits.
The best tracer combination for discerning Rhine infiltrate from dune groundwater in all hydrogeochemical compartments of the aquifer system, was the Cl/Br-ratio plus the stable ^{18}O isotope of the water molecule. In specific hydrogeochemical environments non conservative tracers could still be useful, like Li, B and Mo in the upper aquifer.
This study does not only reveal how multitracing works in the dutch coastal dunes, but it also offers methods with general applicability, namely how to: (a) combine various tracers; (b) rank tracer performance, and (c) quantify and reduce uncertainties in hydrochemically discriminating 2 water bodies and in determining end member contributions to their mixture on the basis of known analytical errors and expected uncertainties in end member concentrations.

Key words:
Multitracing, tracer, artificial recharge, coastal dunes, Rhine water

INTRODUCTION
The coastal dunes of the Western Netherlands are locally recharged by artificial recharge (AR) using basins with pretreated Rhine River water, in order to supply drinking water to large cities like Amsterdam and The Hague. This happens on a large scale since 1955, when the drawdown of groundwater tables and aquifer salinization became unacceptable (Stuyfzand, 1993).
Tracing the infiltrated surface water amidst the original autochthonous groundwater body is of general interest for the following reasons (Stuyfzand, 2006): (1) national and EU legislation compel water utilities to monitor the expansion of the resulting artificial groundwater bodies; (2) water quality monitoring can be optimized by knowing the origin of the water sampled (for instance, expensive analyses on pharmaceuticals are useless in dune groundwater, but useful in infiltrated Rhine water; (3) ecological problems in wet areas surrounding AR sites, like eutrophication, could relate to exfiltration of infiltrated eutrophic river water, which needs confirmation; and (4) hydrological models can be independently validated by mapping the spatial extension of the infiltrated river water.
The distinction between, in this case, the infiltrated Rhine water and coastal dune groundwater requires, however, multitracing techniques to be applied, because both watertypes show a large variation in water quality yielding overlapping tracer contents. In addition, the identification is becoming more difficult due to ongoing trends that reduce the contrast between both waters. However, economic constraints call for a simple, low cost tracing protocol. These developments stimulated Waternet, the municipal water utility of Amsterdam with the largest AR site in the Netherlands utilizing Rhine water, to initiate a study to find the best tracer or tracer combination during the coming decades for identifying, on a routine basis at costs as low as possible, infiltrated Rhine River water amidst autochthonous dune groundwater in their dune water catchment.

Tracers like the ones tested here, have been used on many AR sites and in many hydrological situations world wide, for various reasons (Käss, 1998; Clark & Fritz, 1997). Little attention has been paid, however, to the performance of environmental tracers, both single and in various combinations (multitracing), in diverse hydrogeochemical compartments of the aquifer system. Also little attention has been paid to uncertainties in quantifying the mixing ratio of 2 end-members in a mixed water sample of known composition (Carrera et al., 2004). These gaps are addressed in this contribution.

FIELD SITE AND METHODS

Setting

The dune water catchment area of the city of Amsterdam, which is recharged artificially by basins since 1957, is situated in the coastal dunes about 25 km to the west of the city (Fig.1). The intake of Rhine River water is about 10 km to the south of the city of Utrecht, where it is pretreated by sedimentation, coagulation and rapid sand filtration, and from where it is transported by pipeline to the dunes. Further details about the system are given in Table 1.

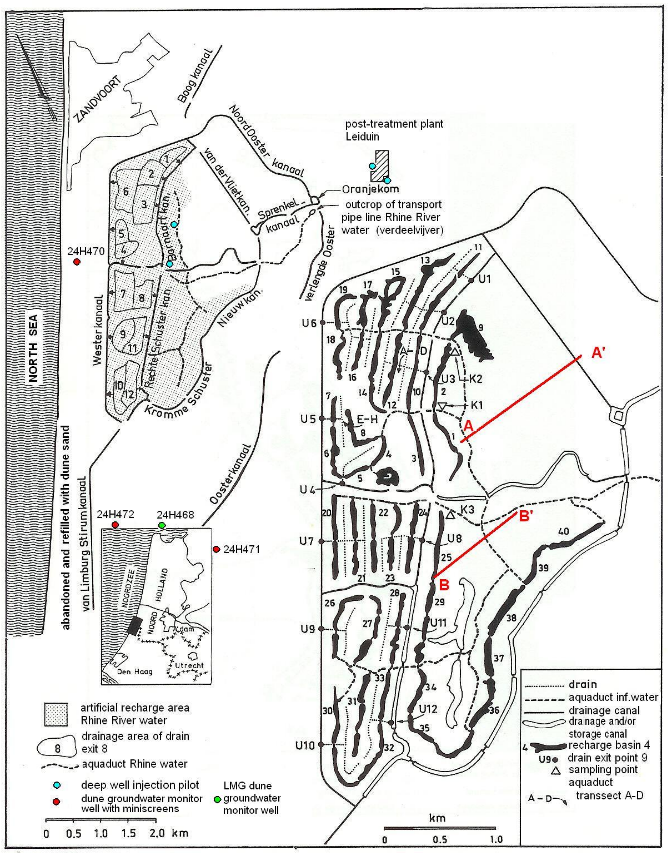

FIG. 1. The Amsterdam dune catchment area, with its artificial recharge system and the position of section AA'.

Feature	Unit	Number or date
Start dune water exploitation	date	1853
Start artificial recharge	date	20-Apr-57
Import infiltration water in 1995	Mm^3/y	56.8
Total withdrawal in 1995	Mm^3/y	66
Total surface recharge basins	m^2	$8.5\ 10^5$
Total surface recharge area	m^2	$102\ 10^5$
Total surface catchment area	m^2	$360\ 10^5$
Mean depth recharge basins	m	0.8
Infiltration intensity	m/d	0.19
Admixing of rain water	%	1
Admixing of shallow dune water	%	10
Admixing of deep dune water	%	5
Modal distance aquifer passage	m	81
Modal travel time in basins	d	5
Modal travel time in aquifer	d	81
Improvement pretreatment	date	Augustus 1974
Sludge removal from recharge basins	times/year	1x/3y till 1974, then 1x/20y

TABLE 1. Characteristic numbers of the artificial recharge system for the municipal water supply of Amsterdam

Groundwater sampling and analysis

In the period 2006 –2007 315 groundwater samples were taken from monitor wells, consisting of either individual or nested PVC piezometers with a 0.2-2 m long well screen, or a set of 10-30 PE miniscreens (0.01 m) attached to a 25-35 m deep piezometer. The monitor wells used were installed >25 years ago, thus reducing bias by residual drilling water and unstable bentonite seals. Prior to sampling, the water content of well screen plus riser was evacuated at least 3 times and until a stable electrical conductivity (EC) was attained.

Sample conservation for multi-element analysis by ICP-MS and ICP-OES consisted of on-site filtration over 0.45 µm membrane filter, acidification by 65% HNO_3 suprapure (0.7 mL/100 mL), and storage in the dark at 4°C. The

elements addressed consist of all main cations (Na, K, Ca, Mg, Fe, Mn), Cl, SO_4, SiO_2, PO_4 and 62 trace elements (heavy metals, metalloids, Li, B, Br, rare earth elements etc.). Samples for Cl, SO_4, HCO_3, NO_3, NH_4, F, DOC and UV-extinction, and for the stable isotopes ^2H and ^{18}O of the water molecule were taken without filtration and without additives for conservation, in 3 separate vials with an air-tight cap construction, and stored in the dark at 4°C. The anions were analyzed by ion chromatography and spectrophotometry (analysis within 0.5 – 7 days), and the stable isotopes (within 3 months) by pyrolysis and mass spectrometry.
Temperature, pH, EC and O_2 were measured in the field, the other quality parameters in the lab. The redox state of each sample was determined on the basis of redox sensitive main components of water, i.e. O_2, NO_3, SO_4, Fe and Mn, following a scheme by Stuyfzand (2006).

A priori division between infiltrated Rhine water and dune groundwater
The groundwater samples taken, were a priori selected and subdivided into 2 groups: 100% dune groundwater (n = 150) and 100% infiltrated Rhine water (n = 165). Any mixing of both groups was excluded by selection of those monitor wells that are unambiguously sited within either the natural dune groundwater body or waterbody recharged by Rhine River water. Also, the hydraulic and hydrochemical monitoring record contradicted any approaching of the Rhine / dune groundwater interface in the considered areas with natural dune groundwater, and of dune groundwater / Rhine water interface in the considered areas with infiltrated Rhine water.

Division into different hydrogeochemical zones
The aquifer system has been subdivided into characteristic hydrogeochemical zones, because some environmental tracers do not behave conservatively in specific environments. The subdivision is based on extensive knowledge of the (hydro)geology and earlier studies on tracer behavior (Stuyfzand, 1993). The following 6 zones are discerned: (sub)oxic eolian dune sand (0 – 7 m above sea level; ASL), largely anoxic beach sand (0 – 5 m below sea level; BSL), anoxic, shallow marine, silty fine sand (5-10 m BSL), (deeply) anoxic, marine, very fine sand and clay (10-20 m BSL), Upper Pleistocene sand (20-40 m BSL), and Middle Pleistocene sand (40-80 m BSL). The upper zones (above 10 m BSL) constitute the phreatic (first) aquifer, the zone of 10-20 m BSL the principal aquitard, and the sands below it the semiconfined (second) aquifer.

Classification of potential tracers
Environmental tracers can be classified on the basis of their origin, behavior and use (Table 2). The conservative tracers (^2H, ^{18}O, Cl, Br) normally perform best in all circumstances. Complications arise when they do not behave conservatively, as in case of evaporation (^2H, ^{18}O and Cl/Br ratio) or interaction with biomass (Br). Non conservative, reactive tracers with retardation factor R = 1 (EC, HCO_3, SO_4, ^3H) may perform well if the reaction accentuates the difference (EC, HCO_3, SO_4) or decay is negligible due to a short residence time (^3H). Reactive tracers with R > 1 (temperature, F, I, Mo, B, Li, Sr, Na, K, Mg) can only be used with great care. Interaction with the porous medium needs to be excluded; for instance sufficient aquifer flushing guarantees lack of sorption reactions. The fourth group of tracers consists of expensive, hard to sample or organic pollutant tracers. This group is not addressed here.

TABLE 2. Classification of useful tracers on the basis of origin, behavior and use.

	Environmental tracers	Origin			Behavior			Use:	
		Natural	Semi-Natural	Synthe-tic	Conser-vative	Reactive R = 1	Reactive R > 1	Approx costs €	Ease + Cheap
1	2H, 18O	X			X			20-120	X
1	Cl, Br		X		X			50#	X
2	Electrical Conductivity		X			X		5-10	XX
2	HCO3, SO4		X			X		20, 50#	X
3	Temperature		X				X	3	XX
3	F, I, Mo, B		X				X		X
3	Li, Sr		X				X	50#	X
3	Na, K, Mg		X				X		X
4	3H / 3He (T1/2 = 12.43 years)		X			X		500	
4	Gd-DTPA			X	X			?	
4	Various: MTBE, 133-TMO, Na-dikegulac			X	X				
4	Pharmaca: carbamazepine			X	X			##	
4	Pesticides: bentazone			X	X				

 # = multi-element analysis via ICP-MS + ICP-OES (50-72 elements) **## = single ca 200, multi ca 1200 €**
133-TMO = 1,3,3-trimethyloxindole (a painting substance)

TRENDS IN RHINE RIVER WATER AND DUNE GROUNDWATER

Rhine River water (the input)

The most relevant quality trends are shown in Table 3. There is a clear decreasing trend for highly soluble salts (Cl, SO_4, Na, K), SEC, NO_3 (since the 1980s), many trace elements (like B, Ba, Br, F), tritium and the Cl/Br ratio. This is due to strong reductions in among others the discharge of waste water from the salt mines in Elzas, overall sanitation of the drainage basin of the Rhine, and reduced atmospheric tritium inputs.
Increasing trends exist for temperature (discharge of cooling water, climate change), ^2H and ^{18}O (probably due to climate change and increased open water evaporation), and HCO_3.

Dune groundwater

The upper 10-15 meters of calcareous coastal dune groundwater experienced since 1970s decreasing trends in Cl (less sea spray deposition, higher rainfall), SO_4 (strongly reduced SO_2 emissions), NO_3 (raised groundwater tables, more uptake by vegetation) and Ca (less acidic atmospheric deposition). At greater depth dune water quality remained fairly stable due to increasing age and dispersion (Stuyfzand, 2008).

OVERALL TRACER CONTRAST AND PERFORMANCE

With some simple statistical parameters the contrast between 28 quality parameters of pure dune groundwater and pure Rhine infiltrate was evaluated, in order to rank their value as an 'overall' tracer (Table 4). An overall tracer is a tracer that performs well in the whole population, independent of the hydrogeochemical zone.
The Student t-test (two-tailed, homoscedastic) was applied to test the probability that both means (dune versus Rhine) are the same. The extremely low probablities for all 28 parameters (Table 3) shows that their means are significantly different, and that the Cl/Br ratio scored as the best and the Cl/SO_4-ratio as the worst tracer.
In order to obtain another measure of the significance of the relative distance for tracer or parameter X between both means, Δ_X and W_X have been defined:

$$\Delta_X = \{\,|C_R - C_D| \;/\; \sqrt{(S_R\, S_D)}\}_{\text{TRACER X}} \qquad (1)$$

$$W_X = \Delta_X \,/\, \mathrm{Max}(\Delta_{\text{ALL TRACERS}}) \qquad (2)$$

with: C_R , C_D = mean concentration or parameter value of respectively Rhine infiltrate and Dune groundwater;
S_R , S_D = standard deviation for respectively Rhine infiltrate and Dune groundwater;
$\mathrm{Max}(\Delta_{\text{ALL TRACERS}})$ = highest score of Δ_X of all tracers considered (Cl/Br in Table 3).

TABLE 3. Quality trends in Rhine River water, since the start of AR in the Amsterdam dune catchment area (after Stuyfzand, 2008). Yellow = increasing; green = decreasing; blue = stable

Parameter	Unit	1957-1966	1967-1976	1977-1986	1987-1996	1997-2006
EGV	uS/cm	771	868	816	781	651
Temp	`C	11.9	12.1	12.4	12.5	13.0
3H	TU	249	199	104	50	33
Cl	mg/L	150	178	158	143	101
HCO3	mg/L	145	150	161	161	168
SO4	mg/L	78	85	69	67	58
NO3	mg/L	11	19	20	15	12
Na	mg/L	78	97	92	78	52
K	mg/L	6.7	7.5	6.3	5.7	4.8
Ca	mg/L	79	83	78	79	73
Mg	mg/L	11.6	12.2	11.4	11.2	10.8
B	ug/L				114	74
Ba	ug/L				113	88
Br	ug/L			226	200	176
F	mg/l		329	210	168	138
Cl/Br	mg/L			695	672	579
SO4/Cl	mg/L	0.53	0.48	0.44	0.47	0.58
HCO3/An	meq/L	0.28	0.26	0.30	0.32	0.39
$\delta^{18}O$ #	‰		-9.6		-9.1	-8.8

 # = estimate based on data in Rhine near Lobith (Lobith + 0.3 o/oo)

TABLE 4. Overall tracer contrast and performance, as tested on 165 samples of pure Rhine infiltrate and 150 samples of pure dune groundwater, independent of hydrogeochemical zone: from top to bottom decreasing contrast indicated by score W_X.
C_P, C_D = Mean concentration of Precipitation, Dune groundwater; C_I, C_R = Mean concentration of Infiltration water (in basins), Rhine infiltrate (in aquifer); S_D, S_R =standard deviation of Dune groundwater, Rhine infiltrate.
$P_{T\text{-}test}$ = probability in Student t-test that $C_D = C_R$.

Tracer application				Dune groundwater			Rhine infiltrate			Δ_X	$P_{T\text{-}test}$
Tracer	**unit**	**type**	W_X	C_P	C_D	S_D	C_I	C_R	S_R		
Cl / Br	mg/L	1	1.00	293	267	68	639	595	79	4.5	2.2E-123
δ18O	‰	1	0.99	-7.1	-7.13	0.49	-9.1	-9.17	0.43	4.4	8.3E-68
HCO3/ΣA	meq/L	2	0.88	0.05	0.75	0.10	0.38	0.42	0.07	3.9	2.9E-86
2H	‰	1	0.84	-46.8	-47.7	5.1	-63.7	-65.1	4.25	3.7	2.8E-46
Cl	mg/L	1	0.74	12	37	21	100	114	26	3.3	1.0E-91
SO4	mg/L	2	0.72	5	15	13	60	60	15	3.2	5.5E-87
Na	mg/L	4	0.64	6.6	21	11	52	58	15	2.9	2.5E-56
K	mg/L	4	0.43	0.24	1.5	0.8	4.6	3.6	1.5	1.9	4.9E-29
SEC	uS/cm	2	0.42	60	480	130	621	721	127	1.9	2.9E-43
Mo	ug/L	3	0.41	0.04	0.26	0.27	1.6	1.07	0.71	1.9	2.2E-23
Li	ug/L	4	0.41	0.6	2.6	1.9	11	7.8	4.2	1.8	1.2E-25
Cl / SO4	mg/L	2	0.41	2.4	58	312	1.66	2.31	2.99	1.8	3.1E-02
100 HCO3/SEC	meq/L, uS/cm	2	0.37	0.05	0.92	0.55	0.42	0.45	0.15	1.6	1.3E-18
UV-Ext	E/m	5	0.35		15.7	31		4.7	1.6	1.6	4.4E-03
TOC	mg/L	5	0.34	1.5	6	9.1	2.5	2.2	0.7	1.5	2.1E-04
Ca/Sr	mg/L	4	0.32	86	292	49	193	220	52	1.4	2.7E-23
Ba	ug/L	4	0.31	3	14	7	78	31	22	1.4	2.7E-13
Rb	ug/L	4	0.28	0.5	0.98	0.79	4.5	2.27	1.38	1.2	1.8E-16
Sr	ug/L	4	0.27	7	297	96	399	418	107	1.2	2.2E-17
B	ug/L	3	0.26	2	32	33	65	67	27	1.2	2.4E-21
Mg	mg/L	4	0.24	0.4	5.3	2	10.4	7.9	2.9	1.1	1.0E-14
Mn	mg/L	4	0.23	0.02	0.17	0.19	0.005	0.46	0.41	1.0	1.8E-11
HCO3	mg/L	2	0.23	2	253	68	159	197	43	1.0	2.1E-15
2H / 18O	‰	1	0.23	6.59	6.7	0.52	7.00	7.16	0.40	1.0	4.5E-08
Gd	ug/L	4	0.19		0.04	0.12	0.02	0.01	0.01	0.9	1.2E-02
U	ug/L	4	0.19	0.02	0.32	0.37	0.8	0.11	0.16	0.9	3.9E-08
Br	ug/L	1	0.19	42	143	81	157	194	45	0.8	1.5E-10
F	ug/L	3	0.15	30	152	74	114	112	51	0.7	1.0E-07

$\Delta_X = \{|C_R - C_D| \,/\, \sqrt{(S_R S_D)}\}_{\text{TRACER X}}$ $W_X = \Delta_X / \mathrm{Max}(\Delta_{\text{ALL TRACERS}})$ 0.12 = S > C

Tracer type: 1 conservative; 2 reactive, R = 1; 3 reactive anion or uncharged, R > 1; 4 reactive cation, R > 1; 5 organic, R > 1

The results of ranking the 28 overall tracers according to the value of W_X is shown in Table 4. For most tracers this ranking method corresponds reasonably well with ranking on the basis of the lowest P-value in the Student t-test (Table 4). When we would set the Δ_X threshold for a good tracer at > 3, then Cl/Br, ^{18}O, HCO_3/Σanions, ^{2}H, Cl and SO_4 would qualify.

TRACER CONTRAST AND PERFORMANCE IN DIFFERENT HYDROGEOCHEMICAL ZONES

The above given ranking would change when comparing the 28 tracers in each individual hydrogeochemical zone. This has not been worked out, but the data shown in Fig.2 clearly demonstrate that tracers like B and Mo, with a bad overall performance (Table 4), do an excellent job in the upper aquifer, and do not function at all as a tracer in the semiconfined aquifer (below 20 m BSL). The reason is that both B and Mo, with significantly higher concentrations in Rhine infiltrate than in shallow dune groundwater, are sorbed to the aquitard on their way to the second aquifer, while B is enriched in deep dune groundwater by desorption from that aquifer due to exchange processes linked to former salt water intrusion.

MULTITRACING FOR REDUCING UNCERTAINTIES

The definition of multitracing here is the combined use of ≥ 2 tracers for identifying the origin or age of water or the processes acting on it, usually aiming at more precision than when using 1 tracer. Well known tracer tandems are $^{2}H/^{18}O$ (Mook, 2000), Cl/Br (Alcala & Custodio, 2005) and $^{3}H/^{3}He$ (Mook, 2000). The more tracers are pointing in the same direction, the higher the probability that the right conclusion is being drawn. An example is shown in Fig.3, where a better 'overall' separation of dune groundwater and Rhine infiltrate is obtained by using 3 tracers (Cl/Br + ^{18}O) than by using 2 tracers (^{2}H and ^{18}O). In fact, multitracing with Cl/Br + ^{18}O is considered the very best method to discern both groundwaters in all environments on a cost effective way.

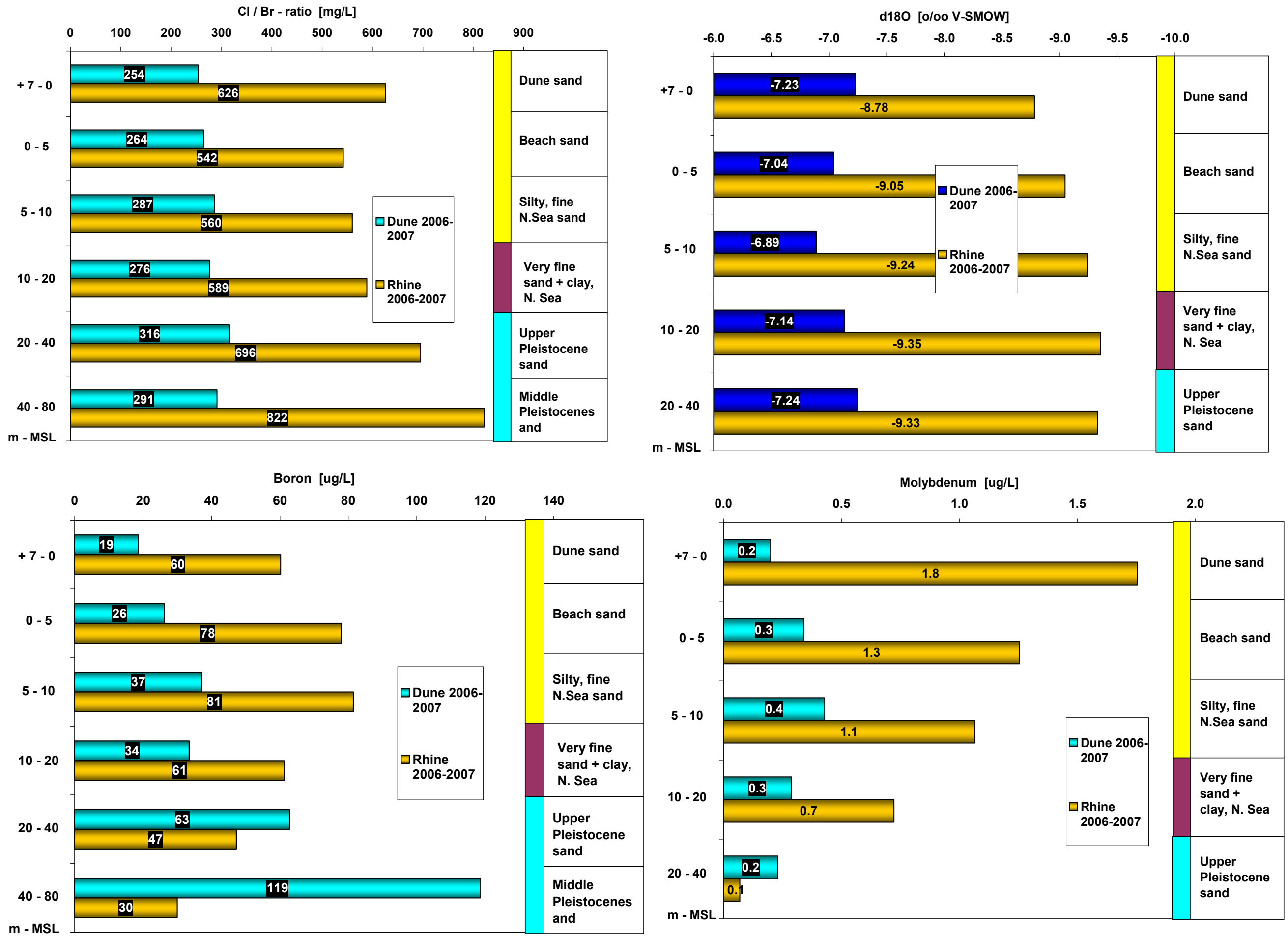

FIG. 2. Mean tracer concentrations (Cl/Br, ^{18}O, B and Mo) of pure Dune groundwater and Rhine infiltrate in 2006-2007, in the 6 hydrogeochemical zones. Cl/Br and ^{18}O are 'overall' conservative; B and Mo are not conservative (reactive, R > 0).

REDUCING UNCERTAINTIES IN QUANTIFYING MIXING RATIOS

Hydrologists or hydrogeochemists frequently need to determine the mixing ratio of 2 different solutions, composed of end-members A and B, in a mixed sample of known composition, by taking:

$$f_A = (C_{MIX} - C_B) / (C_A - C_B) \tag{3}$$

$$f_B = 1 - f_A \tag{4}$$

where: C_{MIX} = concentration C in mix as sampled [mg/L]; C_A = ditto in end member A [mg/L]; C_B = ditto in B [mg/L] f_A = fraction of A in mix [-].

The accuracy of the calculated fraction of A and B strongly depends on a precise estimate of their end-member concentration, which can be a major problem (Carrera et al., 2004). These uncertainties can be reduced as follows: (a) select tracers with a large contrast (Table 4) and well known input record; (b) estimate the age of the sample to narrow down the tracer input; (c) reduce analytical errors; and (d) apply Eqs.3-4 to other tracers and take the average of the different calculations.

When the performance of various tracers has been ranked (like in Table 4), then the average mixing factors (as based on several tracers) could be weighted according to W_X by taking:

$$f_A = \Sigma f_A W_X / \Sigma W_X \quad (5)$$

The accuracy of the calculation of f_A and f_B also depends on their value, when the accuracies for end-member A and B are different, and the analytical accuracy of the mixed sample differs from both. The example in Fig.4, which is representative of a typical 'unmixing' problem for a mixed sample of Dune groundwater and Rhine infiltrate, illustrates this.

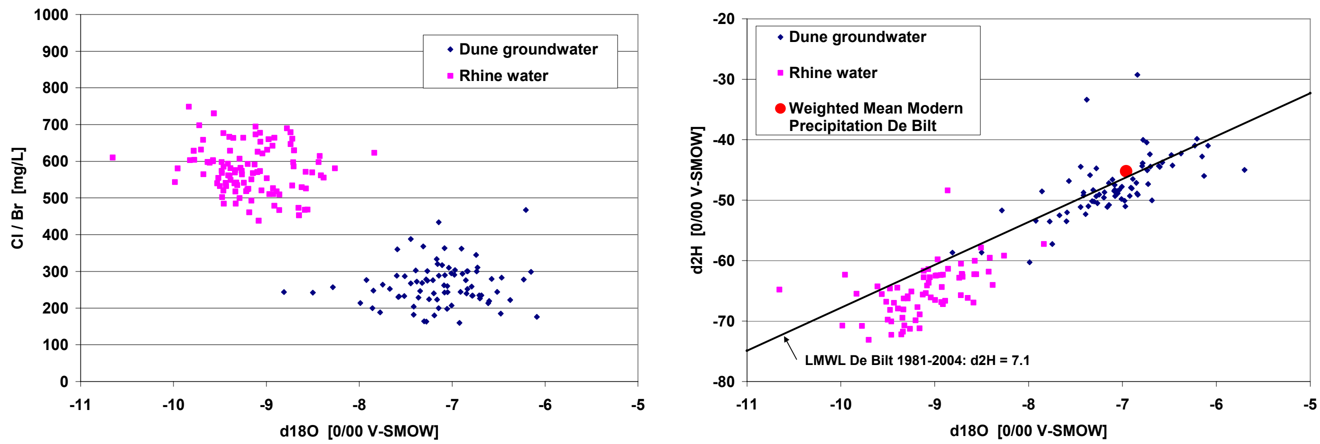

FIG. 3. Overall tracer contrast between pure Dune groundwater and Rhine infiltrate in 2006-2007, for the combination Cl/Br and ^{18}O (best performance) and ^{2}H and ^{18}O (after Stuyfzand, 2008). Also plotted in ^{2}H / ^{18}O diagram: weighted mean rain water for De Bilt (central Netherlands, 1981-2004), with its Local Meteoric Water Line (LMWL) and the Global Meteoric Water Line (GMWL).

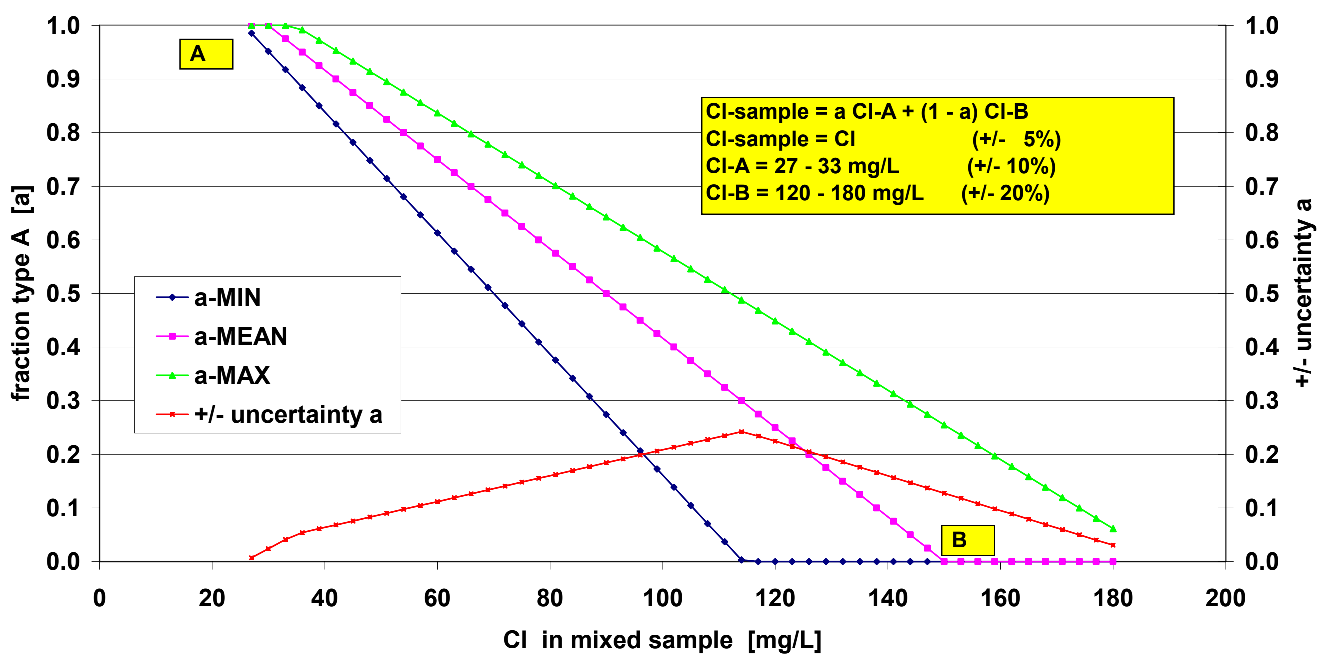

FIG. 4. Spectrum of uncertainty in the calculated fraction of end-member A in a mixture of A+B, with the indicated confidence limits for the concentrations of end-members A and B, and the analytical inaccuracy for the measured Cl concentration in the sampled mixture.

Conclusions

In the coastal dunes of the Western Netherlands, artificially recharged Rhine River water can be discerned from autochthonous Dune groundwater by various envionmental tracers. The ranking of 28 different tracers according to a new parameter (W_X) resulted in the following top-5 of best 'overall' tracers (which work in all studied hydrogeochemical compartments of the aquifer system), in order of decreasing performance: Cl/Br, ^{18}O, HCO_3/Σanions, ^{2}H and Cl.

The best 'overall' tracer combination for recognizing Rhine water in the dunes (but also in the Rhine fluvial plain as River Bank Filtrate) consists of the Cl/Br–ratio plus the stable isotope ^{18}O of the water molecule. Using more tracers than these 3 may further reduce uncertainty levels, but adds to the costs (unless multi-element analysis is applied; see below).

Tracing can be very simple and less expensive, for example by using EC or Cl alone, but requires that a narrow window for the quality of both water types be ascertained. Tracers with limitations due to their reactive or sorptive behavior are among others: Na, K, Ca, Mg, B, Ba, Li, Mo and Sr. However, relatively low costs of their multi-element analysis by ICP-MS + ICP-OES make it worthwile to add them to the tracer list, even more when this analysis also includes Cl, SO_4 and Br (at the same price).

In specific situations B, Li and Mo proved to be excellent tracers, in others they were worthless. This calls for a good definition of the hydrogeochemical environment when interpreting tracer results.

Trends in Rhine River and Dune groundwater chemistry may change the strength of a tracer or tracer combination. It is therefore recommended to verify the tracer's validity by regular monitoring of the end members. Yet, it is expected that the tracer combination of the Cl/Br–ratio plus ^{18}O will continue to perform well for the coming decades.

This study does not only reveal how multitracing works in the dutch dunes, but also offers methods with general applicability, namely how to: (a) combine various tracers; (b) rank tracer performance, and (c) quantify and reduce uncertainties in hydrochemically discriminating 2 water bodies and in determining end member contributions to their mixture on the basis of known analytical errors and expected uncertainties in end member concentrations.

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Using COP Method to Assessment of Karst Vulnerability in Izeh, South West Iran

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Abstract
The 'COP method' has been developed for the assessment of intrinsic vulnerability of Naal-e-Asbi karst aquifer, south east Izeh. This method uses the properties of overlying layers above the water table (O factor), the concentration of flow (C factor) and precipitation (P factor) over the aquifer, as the parameters to assess the intrinsic vulnerability of groundwater. This method considers karst characteristics, such as the presence of swallow holes (C factor) and their catchments areas as well as karstic landforms, as factors which decrease the natural protection provided by overlying layers (O factor). The P factor allows for consideration of the spatial and temporal variability of precipitation, which is considered the transport agent of contamination. The results have been compared with nitrate concentrations in the Naal-e-Asbi karst aquifer. Comparisons with these data show the advantages of the COP method in the assessment of vulnerability of karstic groundwater.

Keywords: Vulnerability, Karst aquifer, COP method, recharge, Iran

Introduction
Groundwater is a natural drinking water resource often subjected to severe human impact. Strategies are required to preserve optimum groundwater quality, and so management of this vital natural resource has become a worldwide priority. Groundwater in karstic aquifers can be extremely sensitive to contamination from the land surface. Major karstic carbonate aquifers are among our most valuable groundwater sources. Protection of groundwater resources is not only environmentally sound but also is cost-effective because prevention of contamination is less expensive than remediation after it has occurred. The objective of vulnerability mapping is to identify the most vulnerable areas and priorities those. A vulnerability map may thus help the decision makers to find a scientifically based balance between groundwater protection and socioeconomic aspects (Goldscheider, 2005).
Vrba and Zaporozec (1994) state that vulnerability is a qualitative, relative, non-measurable and dimensionless property. They suggest distinguishing between intrinsic and specific vulnerability. The former only depends on the natural properties of an area, while the latter also takes in to account the properties of the contaminant.
Vulnerability mapping, while receiving some criticism internationally and largely on account of uncertainty surrounding the definition of the term "vulnerability", is nonetheless a common method for representing spatially and semi-quantitatively the relative susceptibility of an aquifer to contamination from surface sources. Assessment of vulnerability is based on the environmental characteristics of a landscape that facilitate or impede contamination, and represents the "likelihood of a contaminant to reach a specified position in the groundwater system after introduction at the surface" (National Research Council 1993 in Bekesi and McConchie 2002). Various systems of vulnerability evaluation and ranking have been developed and applied in the past including AVI (van Stempvoort et al. 1992) and DRASTIC (Aller et al. 1987). Such vulnerability methodologies take into account that the natural environment protects itself when a contaminant is introduced.
However, while the methodologies for undertaking vulnerability assessments are reasonably well-developed for unfractured aquifers, standard approaches for representing fractured bedrock aquifers are currently limited. The EPIK (Doerfliger and Zwahlen 1998; Doerfliger et al. 1999) and PI (Goldscheider et al. 2000) methods which were specifically developed for the assessment of vulnerability in karstic areas.
At the same time, karst aquifers are particularly vulnerable to contamination: Due to thin soils, flow concentration in the epikarst (the uppermost, often intensively fractured and karstified layer of a carbonate aquifer) and point recharge via swallow holes, contaminants can easily reach the groundwater, where they may be transported rapidly in karst conduits over large distances. The residence times of contaminants are often short, and processes of contaminant attenuation, therefore, often do not work effectively in karst systems. Karst aquifers consequently need special protection (Goldscheider, 2005).
The Directorate General for Science, Research and Development of the European Commission supported COST Action 620 (COST is the acronym for "COoperation in Science and Technology) which considered "Vulnerability and Risk Mapping for the Protection of Carbonate (Karst) Aquifers" and ran between 1997 and 2003.
An European Approach to vulnerability mapping in karst aquifers has been developed in the framework of COST Action 620, and considered four factors (Daly et al. 2002; Goldscheider and Popescu 2004): overlying layers (*O*

factor), concentration of flow (*C* factor), precipitation regime (*P* factor) and karst network development (K factor). Resource vulnerability maps may be prepared by a combination of the *O*, *C* and *P* factors whereas for source vulnerability maps, the addition of the K factor is required. Originally, Daly et al. (2002) developed a conceptual framework but did not propose detailed guidelines, tables or formulae for vulnerability assessment. They highlighted the need for developing methods, which facilitated flexible application in different European regions using available data and accommodating varying climatic conditions. Intrinsic vulnerability maps are used as environmental management tools so must be capable of validation with natural or artificial tracers in order to ensure that the conceptual understanding of prevailing hydrogeological conditions are valid (Bruy`ere et al. 2001; Jeannin et al. 2001; Goldscheider et al. 2001; Perrin et al. 2004).
This paper proposes a recently developed method (COP) that considers the special hydrogeological properties of karst. The method can be applied in different climatic conditions and different types of carbonate aquifers (diffuse and conduit flow systems). In addition, the COP method uses variables, parameters and factors in line with those proposed in the European Approach (Daly et al. 2002; Zwahlen 2004) and it can be applied using different levels of available data. A quantification and category system for each parameter has been established, using the combination and weighting of variables. The proposed method has been used to map the intrinsic vulnerability of two carbonate aquifers in Southern Spain with differing climate, hydrogeology and geology. Both aquifers have been previously investigated using other vulnerability mapping methods (Longo et al. 2001; V´ıas et al. 2005; Andreo et al. 2005) and some validation of the results has been carried out. Consequently, it is possible to compare results and to demonstrate the effectiveness of the COP method in karst areas, particularly within Mediterranean aquifers.
Groundwater from karst aquifer is among the most important resources of drinking water for the growing urban and country population of the south west Iran. In Izeh, carbonate terrains occupy 45 % of the land surface and a significant portion of the drinking water supply is abstracted from karst aquifers. In Izeh, karst water contributes 90% to the total drinking water supply. The study area, Izeh polje, is a 140 Km^2 alluvium, located in the southwest of Iran, approximately 200 Km northeast of Ahvaz city, in Khuzestan province (Fig. 1). The study area lies within the Karun River Basin. Most of the area of the Izeh polje (65%) is used for agriculture. In the surrounding environs, the primary land uses is agriculture.

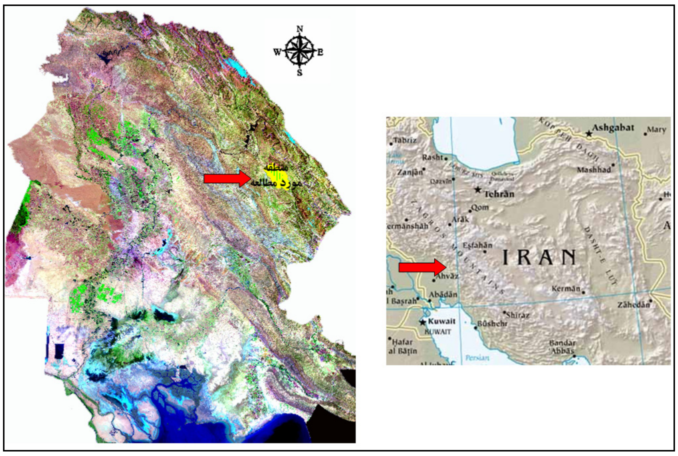


Figure 1- Map of the study area

Average annual rainfall is 660 mm, with the most of the precipitation resulting from late autumn and early spring thunderstorms. The main annual temperature is 20.7 ^{o}C, with a July high of 33 ^{o}C and a February low of 5.9 ^{o}C. The average annual real evapotranspiration from free surface of water is 1632 mm. The climate present across the Izeh requires farmers to supplement precipitation with irrigation to sustain their crops. Over all, 40% of all groundwater in the region is used for irrigation. Ten karst wells are exploited for domestic use, with discharge about 350 m^3/day for each well. Surface water flowing in the Izeh polje, which is a closed basin, enter to two relatively vast lakes, are located north and southeast of Izeh town, called Miangaran and Ab-e-Bondan, respectively (Fig. 2).

Geological settings
The Izeh plain occurs in massive limestones in a tectonically complex zone. Following the classification given by Ford and Williams (1992), Izeh plain is a structural polje. The axis of elongation of the polje parallels the Nal-e-Asbi Syncline axis from northwest to southeast. The Izeh polje with 1–8 km wide and up to 20 km long is one of the largest poljes of the Zagros Ranges. The southeast nose of doubly plunging syncline, called Nal-e-Asbi syncline, that enclosed Ab-e-Bondan Lake have U shape. From the viewpoint of regional geology, Izeh polje is located in the Folded Zagros Region.
Once a season, and sometimes more frequently, the Izeh polje flood because of greatly increased flow from the mountains and low infiltration capacity surface of the polje, which cause runoff occurs, extensively. In flood time the entire base of the polje may become a lake. The recent morphological features of the Izeh polje are a

consequence of tectonics and corrosion. Karstic erosion of the land surface is controlled by chemical and mechanical processes occurring in the upper portion of the limestone where the most intense dissolution occurs.
Geological formations from lower Cretaceous to recent age have outcrops in Izeh area (Fig. 2). Thick limestone of Darian-Fahlian Formations are the oldest rocks in the study area that underlay by Kazhdumi shale formation, thin bed limestone Ilam- Sarvak Formations, marl and shale of Pabdeh and Gurpi Formations, Asmari limestone formation, Gachsaran gypsum and marl beds, Tukak conglomerate, and alluvial sediments, respectively.

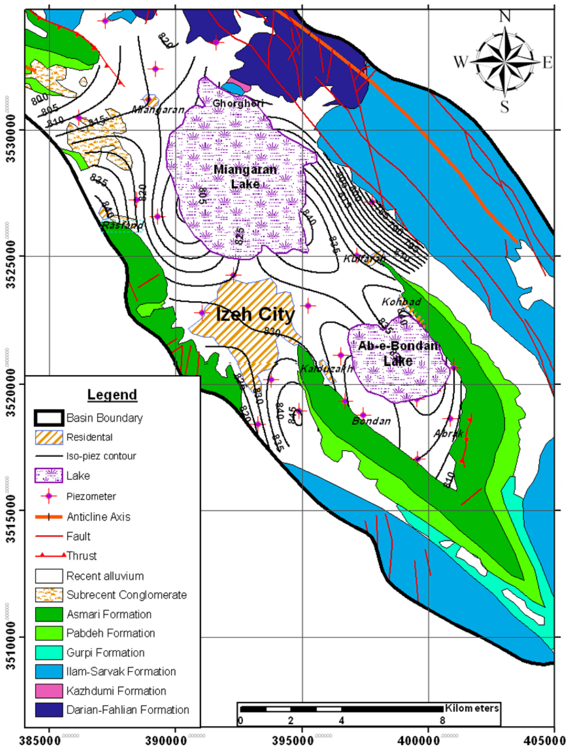


Figure 2. Geological map of Izeh polje

Hydrogeology
The main surface recharges to the Izeh alluvial aquifer are infiltration of rainfall and irrigation water. Izeh karst aquifer recharges alluvium in the south and west of Ab-e-Bondan Lake and west of Izeh city. Instead of this, Izeh alluvial aquifer discharges to the karst aquifer in the north of the plain and west and east of Miangaran Lake via underground dissolution canals that coincident to the faults on the limestone formations. Probably, hydraulic connection between the lakes and Izeh alluvial aquifer is poor because the soil in the lake areas is very fine. Two natural lakes in the Izeh polje, called Miangaran and Ab-e-Bondan, are formed by the emerging groundwater. Two factors that effect lake formation are karst development in host limestone and thickness of unconsolidated overburden (the confining unit). The host limestone of Miangaran and Ab-e-Bondan lakes are Ilam-Sarvak and Asmari formations, respectively, which are highly karstic. Thickness of overburden is the other controlling factor. A slight surface depression will form over a collapse in an area with a thick unconsolidated overburden (up to 130 m).
Over-exploitation of karst wells in the Jamushi and Bondan villages caused a groundwater flow direction toward the Nal-e-Asbi karst aquifer in these areas but in the late of rainfall season the ground water flow reversed from the karst aquifer to the Izeh alluvial aquifer. In the Nal-e-Asbi area, where groundwater is used intensively, the water table can fluctuate to cause reverse flow between the karst and alluvium.

Methods
The COP method has been applied in two aquifers in the south east Izeh, the aquifer of Nal-e-Asbi. This is a karstic aquifer with a conduit flow system. Common methods of intrinsic vulnerability assessment have been applied in Izeh alluvial aquifer (Nassery et al., 2009). GIS analyses have been defined on the basis of groundwater conditions in the study area and appropriate weighting has been assigned to each criterion maps according to COP method. In this study, the index overlay approach has been used to generate the commensurate and aggregate criterion maps. The methodology adopted in the present study involves several steps that have been explained schematically in the Figure 3. Finally, the groundwater vulnerability map has been prepared for the study area.
The procedures for generating criterion maps overlap with the process of creating geographical data base. This process is based on GIS functions, which includes geographical data input (acquisition, reformatting, geo-referencing, compiling, and documenting relevant data), storage, manipulation and analysis (to obtain information), and out put. In a sense, the criterion maps can be considered as output of GIS- based data processing and analysis. In this research, these processes were performed in ARCGIS 8.3 software.

The COP acronym comes from the three initials of the factors used: flow Concentration, Overlying layers and Precipitation (Fig. 3). The conceptual basis of this method, according to the European Approach (Daly et al. 2002; Goldscheider and Popescu 2004), is to assess the natural protection of groundwater (*O* factor) determined by the properties of overlying soils and the unsaturated zone, and also to estimate how this protection can be modified by the infiltration process – diffuse or concentrated – (*C* factor) and the climatic conditions (*P* factor – precipitation).

Discussion

The *O* factor considers the protection provided to the aquifer by the physical properties and thickness of the layers above the saturated zone. Daly et al. (2002) proposed subdivision into four layers: topsoil, subsoil, nonkarstic rocks and unsaturated karstic rocks. In the proposed COP method only two layers with important hydrogeological roles are used in order to evaluate the *O* factor: Soils [*O*S] and the lithological layers of the unsaturated zone [*O*L]. The *soil subfactor* [*O*S] deals with the biologically active part of the subsurface, where attenuation processes occur and as a consequence, when present, should be taken into account in vulnerability mapping. The *lithology subfactor* [*O*L] reflects the attenuation capacity of each layer within the unsaturated zone. The assessment criteria for its quantification are the type of rock (which determines its hydrogeological characteristics, mainly effective porosity and hydraulic conductivity) and the degree of fracturing (ly), the thickness of each layer (*m*) and any confining conditions (cn) (Fig. 3). The attenuation capacity increases with the sum of the protective layers. Thus, an *O* score is obtained by adding the subfactors *soil* [*O*S] and *lithology* [*O*L], yielding a corresponding *protection value*. The lowest values of the *O* factor (higher vulnerability) correspond to areas where carbonate materials outcrop and where the soil is poorly developed or absent. Moderate and High protection values (lower vulnerability), derived from higher *O* scores, are representative of areas where the degree of protection is high, either due to the presence of soil or of low-permeability lithologies.

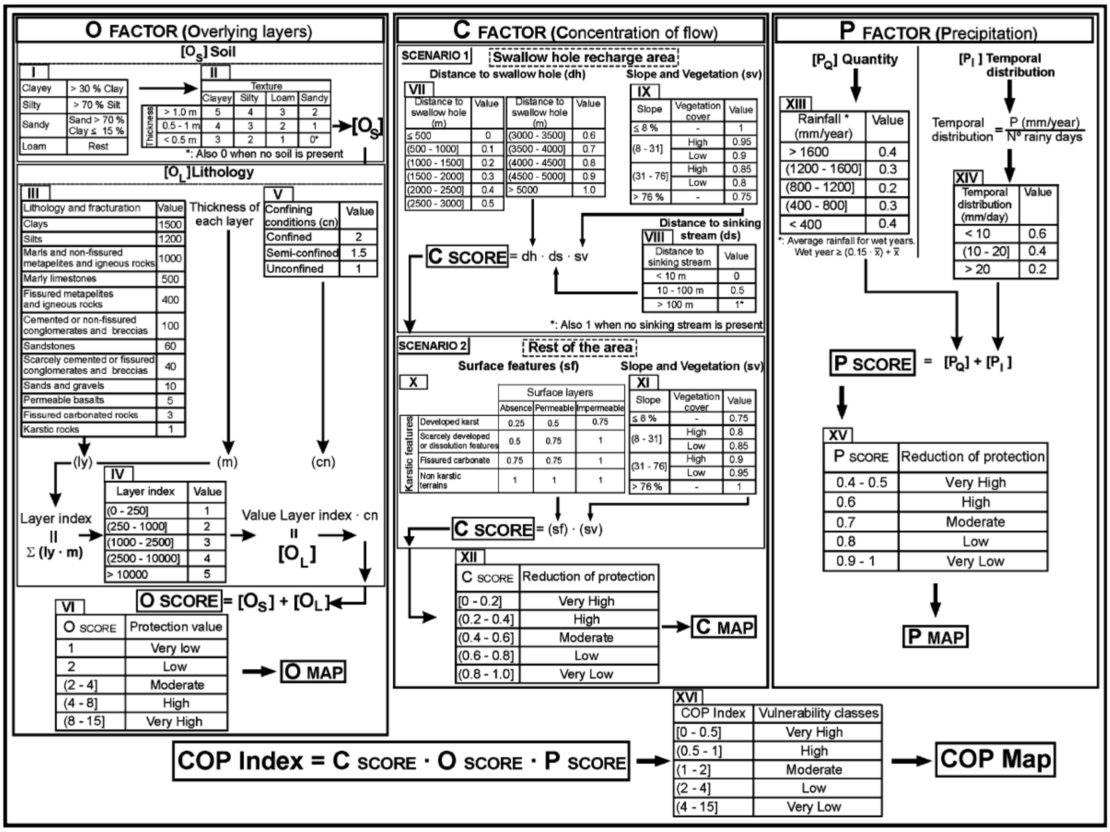
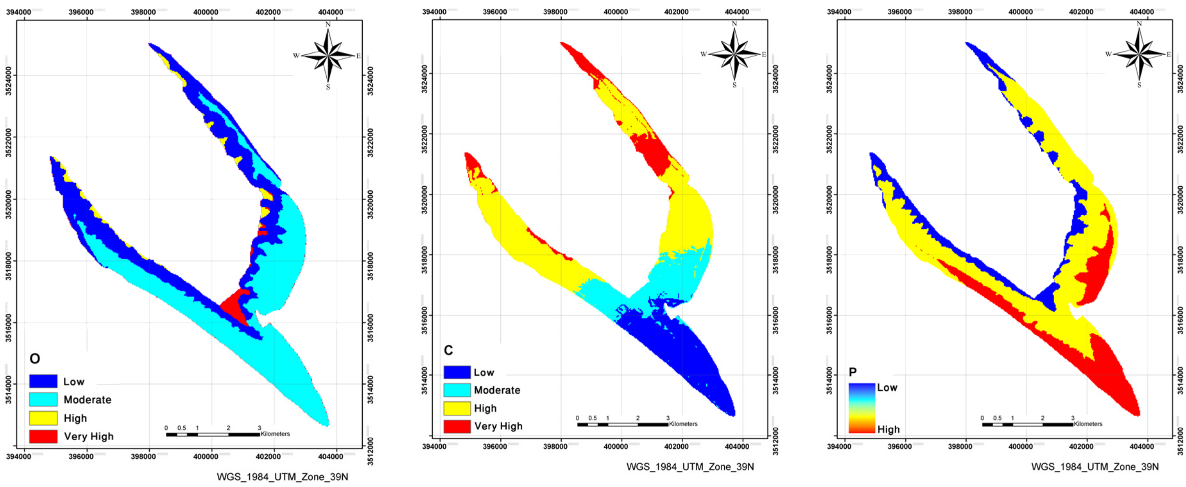


Figure 3- Diagram of the COP method, showing the differentiation of the *C*, *O* and *P* factors (Vias et al., 2006)

The *O* factor of Nal-e-Asbi karst aquifer (Fig 4a) describes the vulnerability of groundwater from contamination where infiltration through the unsaturated zone is diffuse. The *C* factor (Fig 4b) is a modifier of the *O* factor (overlying layers) and represents the potential for water to bypass the protection provided by the overlying layers (Daly et al. 2002). The *C* factor represents the degree to which precipitation at or near aquifer outcrop is concentrated into a swallow hole, bypassing the unsaturated zone. This is based on the PI method (Goldscheider et al. 2000) and the EPIK method (Doerfliger and Zwahlen 1998).

The *C* score under situation characterized autogenic recharge with no concentrated infiltration via a swallow hole or at the foot of a slope is evaluated by the combination of only three variables: surface features (*sf*), slope (*s*) and vegetation (*v*). The *Surface features* parameter (Fig. 3) considers those geomorphological features specific to carbonate rocks and the presence or absence of any overlying layers (permeable or impermeable) which determine the importance of runoff and/or infiltration processes. Zones with concentrated infiltration via swallow holes where the natural protection given by the *O* factor is bypassed, have low values for the *C* factor (Fig. 3).

Conversely, where diffuse infiltration occurs in the absence of karst features or where there is runoff out of the aquifer, then the aquifer retains some natural protection.
A certain percentage of precipitation and snowmelt infiltrates into the ground to recharge aquifers. The amount of recharge is dependent on the rate, duration, and frequency of precipitation. Other factors such as topography, soil type, characteristics of bedrock, vegetation, evaporation rate, and transpiration rate determine how much precipitation or snowmelt will infiltrate and how much will be accounted for as surface runoff. Recharge can be estimated, based on these factors, and expressed in inches per year. A high recharge rate can cause leaching or mobilization of contaminants from the surface to the water table. It follows that higher ratings should be given to areas with higher recharge rates.
The *P* factor includes the quantity of precipitation and factors which influence the rate of infiltration, i.e. frequency, temporal distribution, duration and intensity of extreme rainfall events. These factors help determine the ability of precipitation to transport contaminants from the surface to the groundwater; the greater its capacity to transport contaminants towards the aquifer, the higher the implied vulnerability. The *P* factor is evaluated by two subfactors: *Quantity* of precipitation [*P*Q] and *temporal distribution* of precipitation [*P*I]. The [*P*Q] subfactor (Fig. 3) describes the effect of rainfall quantity and the annual recharge on groundwater vulnerability. It corresponds to the mean annual rainfall of a historical series of wet years.
The [*P*I] subfactor concerns the temporal distribution of precipitation in a certain period of time and thus is indicative of the intensity of precipitation. This subfactor enables a comparison to be made between zones within Europe, where rainfall and intensity conditions are highly variable. The greater the daily rainfall, the greater the amounts of runoff towards swallow holes that favour concentrated infiltration. Where infiltration is diffuse and slow, the [*P*I] sub-factor is low; usually in such circumstances the volumes of recharge are relatively small. Higher values of the *P* factor (Fig. 4c) indicate a lower impact on the level of protection afforded by the *O* factor. However lower values indicate that precipitation, as a function of quantity and intensity, diminishes the protection afforded by the *O* factor and increases groundwater vulnerability.



a) O layer b) C layer c) P layer

Figure 4- Maps of the layers in COP method

The factors of the COP method have been combined to evaluate the intrinsic vulnerability of a groundwater resource, as proposed in the following formula:

$$\text{COP Index} = C \cdot O \cdot P$$

The final numerical representations of the *C*, *O* and *P* factors (the *C*, *O* and *P* scores) are multiplied, because each one is considered to impact on the assessment of vulnerability of karst aquifers. Within the COP method, the values for the intrinsic vulnerability index range between 0 and 15. The COP map of the Nal-e-Asbi karst aquifer (Fig. 5) shows high vulnerability throughout most of the system, due to the high degree of karstification of the carbonate outcrop. This high degree of karstification favours rapid infiltration from the surface to the saturated zone.
The COP map of the Nal-e-Asbi karst aquifer (Fig. 5) shows high vulnerability throughout most of the system, due to the high degree of karstification of the carbonate outcrop. This high degree of karstification favours rapid infiltration from the surface to the saturated zone. Within the carbonate outcrop, the *O* factor determines Very High vulnerability in areas where the thickness of the unsaturated zone is less than 30 m, as in the NW edges of the aquifer (Fig. 5). The *C* factor determines elevated vulnerability in areas where infiltration processes are dominant rather than runoff, i.e. areas where karst landforms are not covered by impermeable layers or areas where the karst is not highly developed and the slope/vegetation favours infiltration. These areas can be small, consequently the map shows many small zones of very high vulnerability.
This study produced a pollution potential map for the Nal-e-Asbi karst aquifer and classified 1.6% as having low pollution potential, 2.1% as having moderate pollution potential, , 40.7% as having high pollution potential, and 55.6% as having very high pollution potential. The limbs of Nal-e-Asbi Syncline have more pollution potential rather than NE parts of the plain.

Nitrate contamination of karst groundwater (more than 45 mg/l NO3) has also been reported over the last two decades, in the Nal-e-Asbi karst aquifer. The eastern part of karst aquifer has witnessed rising nitrate concentration in groundwater reported since the1980s. Nitrate concentrations in the ground water of Izeh karst water were typically under 30 mg/l but occasionally exceeded 40 mg/l.

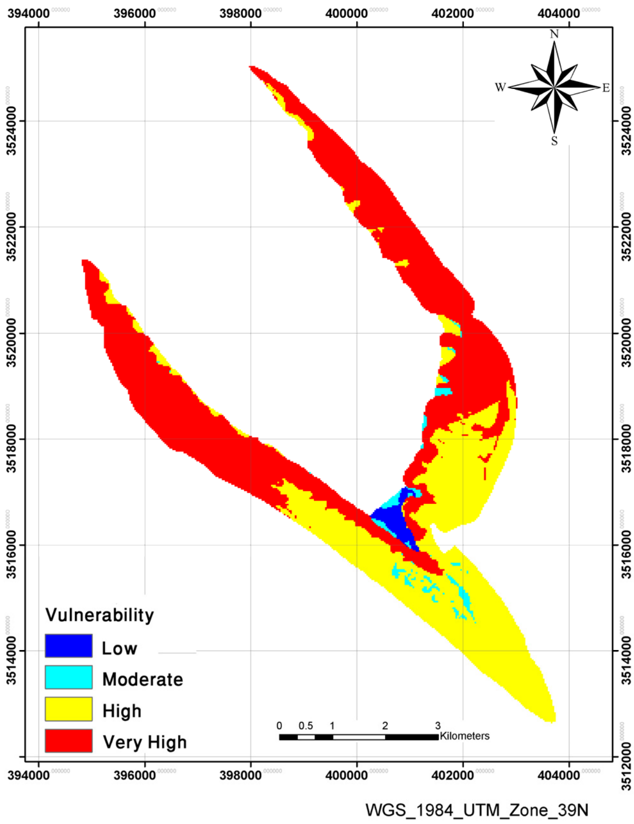


Figure 5- COP map of Nal-e-Asbi karst aquifer

Conclusions

The COP method has been designed to evaluate the intrinsic vulnerability of the groundwater resource in carbonate aquifers with different degrees of karstification and can be successfully used to consider both diffuse and conduit flow systems, under different climatic conditions. It takes into account three factors: *C* (flow concentration), *O* (overlying layers) and *P* (precipitation). In addition, the COP method establishes detailed guidelines, tables and formulae for vulnerability assessment and selects the variables, parameters and factors to be used according to the European Approach proposed by COST Action 620. The method can be applied using geo-environmental data available in most countries, with some fieldwork but without extensive input from geographical information systems (GIS). As a result the COP method is likely to be practical and useful for decision makers implementing groundwater protection schemes. The COP method has been applied to Nal-e-Asbi carbonate aquifer (SE of Izeh) with differing hydrodynamic characteristics and climatic conditions. The COP method represents a significant step forward in assessing the vulnerability of groundwater within karst aquifers, particularly in Middle East type conditions.

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Modeling the Managed Aquifer Recharge for Groundwater Salinity Management in the Sokh River Basin

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Abstract

The vulnerability of surface water sources in the Syrdarya River Basin, due to their transboundary nature and climatic change, raises the importance of the shift from canal irrigation to conjunctive use of surface water and groundwater. However, groundwater development for irrigation may increase salinity of water due to leaching of dissolved solids from the salt-affected vadoze zone and blending of freshwater and saline water. In this paper managed aquifer recharge and discharge are analyzed as a strategy to maintain the groundwater quality of the Sokh aquifer of the Fergana Valley located upstream of the Syrdarya River Basin. Field studies suggested that groundwater recharge from the river floodplain may contribute to maintaining good-quality water in the groundwater system. The modeling study examines groundwater salinity change over a 5-year period under different managed groundwater recharge and discharge scenarios. The modeling results show that adopting water saving technologies and increased groundwater recharge through the river floodplain allows maintaining low groundwater salinity. The studies found that developing groundwater for irrigation increases salinity in the aquifer due to downward saline water fluxes. The results indicate that managed aquifer recharge and discharge contribute to maintaining salinity levels in the vadoze zone and groundwater.

Keywords: Managed aquifer recharge, groundwater use for irrigation, numerical modeling and scenario analysis, Fergana Valley, Central Asia

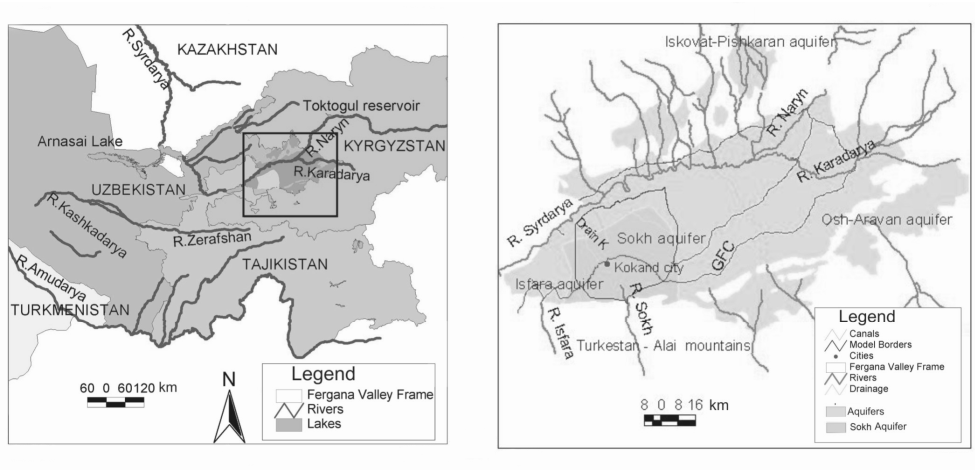
Introduction

Vulnerability of surface water sources in the Syrdarya River Basin due to their transboundary nature and climatic change raises the importance of shifting from canal irrigation to conjunctive management of surface water and groundwater. This strategy is important for the Fergana Valley located upstream of the river basin. Although the valley has plenty of underdeveloped groundwater resources, surface water diversions for irrigation exceed one-third of the river flow. Whereas high water shortage occurs in the midstream and in the lower reaches of the basin, the surplus of groundwater resources of the Fergana Valley causes waterlogging and salinity issues. These issues emphasize the need to shift from surface irrigation to conjunctive use of groundwater and surface water.

However, conjunctive use associated with extensive groundwater abstraction may cause degradation of groundwater quality due to leaching of salts from the vadoze zone and blending of groundwater of different layers. This may increase the salinity of both groundwater of shallow aquifers, which are the sources of irrigation water, and deeper aquifers, which are the sources of drinking water supply. Under such conditions it is important to develop a strategy to ensure water quality in the shallow aquifers suitable for irrigation and in the deep aquifers for drinking water supply. The goal of this study was to develop a strategy of managed aquifer recharge and discharge aimed at maintaining water quality under intensive groundwater abstraction for irrigation uses. The objectives of the study were to assess 1) the effect of the groundwater recharge on water quality (salinity), and 2) the efficiency of different managed aquifer recharge strategies aimed at maintaining groundwater quality. In this paper, two strategies were analyzed:

1) water conservation and managed aquifer recharge and 2) managed aquifer recharge and discharge by constructing shallow wells for irrigation.

The effect of the proposed strategies on groundwater salinity is analyzed taking the Sokh aquifer, one of the typical aquifers of the Fergana Valley, as an example (Figure 1).



a) b)

Figure 1. The upstream of the Syrdarya River Basin (a) and location of the Sokh aquifer in the Fergana Valley (b)

Recently, Gracheva et al. (2009) tested managed aquifer winter recharge (MAWR) at the Sokh aquifer through the hydrodynamic modeling and found that the groundwater abstraction at 620 million cubic meters per year (Mm^3/yr.) creates free capacities at 300 Mm^3/yr. available for banking the winter flow of the Sokh and Naryn rivers. However, these promising results were obtained without considering possible water-quality changes in the groundwater system. That is why these field and modeling studies were initiated aimed at understanding possible changes in water quality, which is important prior to wide dissemination of the recommendations on groundwater banking (Karimov et al., 2010). Studies on field water budgeting were carried out in 2009 to estimate groundwater recharge from the river floodplain. Based on the findings of the field study, long-term groundwater recharge from the floodplain was estimated using the river flow data followed by examining the correlation between the groundwater recharge and water salinity under the current conditions. Finally, modeling of groundwater quality was applied to assess the efficiency of managed aquifer recharge strategies to maintain the quality of the groundwater.

This paper starts with the description of the study area, applied field studies and modeling approach, and continues with the collection of initial data for compiling the groundwater salinity model of the Sokh aquifer. This is followed by the results of the model calibration. Groundwater recharge from the river floodplain is estimated for current conditions. Then, changes in water quality are predicted under maximum groundwater abstraction levels. Finally, three strategies of managed aquifer recharge were analyzed to maintain groundwater quality.

Methodology and characterization of the study area

Characterization of the study area

The Sokh River Basin extending over 180,000 ha has two geomorphological zones with steep coarse shingle deposits upstream and flatter loamy clay deposits lower down. There are two main sources of water in the study area: Sokh River and the Big Fergana Canal (BFC) delivering water of the Naryn River to the water-short areas of the Fergana Valley. The Sokh River flows north from the Turkestan-Alai Mountains into the alluvial valley along the Syrdarya River. In the upper part of the study area the river has a floodplain 600 m wide and 10,000 m long formed by shingle and gravel deposits. The head works of Sarykurgan on the river can be found just as the river enters the study area. The river, which has a long-term average flow at 12 m^3/s, is fed by snowmelt from glaciers, with a maximum flow in summer and a minimum flow in winter. The BFC delivers water of the Naryn River during the dry summer months, when the flow of the Sokh River does not cover irrigation demand of the basin. The Sokh aquifer, selected for the modeling studies spreads in deposits of the fan of the Sokh River. The water-bearing strata in the study area consist of upper quaternary (Q_{III}), intermediate quaternary (Q_{II}) and lower quaternary (Q_{I}) deposits. These deposits contain gravel and shingle with an interlayer of loamy sand and loamy deposits. The gravel and shingle deposits predominate in the southern part of the study area with increasing proportions of loamy sand and loamy soils in the northern parts. The depth to groundwater varies from 72 to 116 meters (m) at the head of the system, and can be as little as 0.5-2.5 m below ground level in the discharge zone.

Six zones were specified within the Sokh aquifer for water and salt budgeting. In the natural recharge zone, which is in the head part of the river basin, the transmissivity of the water-bearing strata exceeds 2,000 m^2/d (Figure 2).

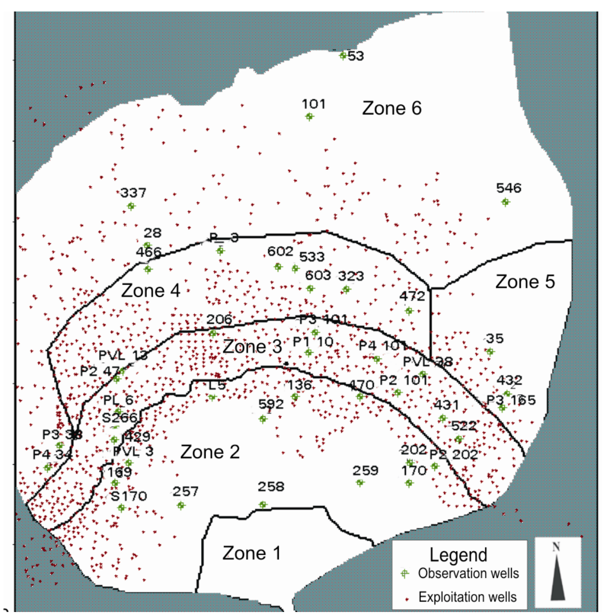


Figure 2. Hydrogeological zoning of the Sokh River Basin

A wide groundwater transit zone replaces the natural recharge zone to the north. The replacement of highly permeable deposits by low-permeable loamy deposits creates the spring zone. This zone has a narrow belt with boundaries 3-5 km to the south from the BFC and 5 km to the north. Managed aquifer recharge is focused on these three zones called the management zone to the north of which spread two zones of groundwater discharge to the drainage zones and a wide groundwater dispersion zone. In the paper, these last three zones are together called the discharge zone.

Field studies on groundwater recharge

The flow of the Sokh River is fully withdrawn into canals at the Sarykurgan head works. The head works operation organization uses the following procedure to ensure safe maintenance of the structure:

- When the river flow is less than 40 m^3/s, no flow is released to the floodplain of the river, and the river flow is diverted fully into the irrigation canals.
- When the flow exceeds 40 m^3/s, but less than 80 m^3/s, the organization, which has built a temporary dam 1 km far from the head works across the riverbed, releases the flow exceeding 40 m^3/s into the river floodplain (upper part).
- When the river flow exceeds 80 m^3/s, the flow exceeding 40 m^3/s is released to the river floodplain. The water surface covers a floodplain area of 600 ha up to the second flow-regulating structure, which is 10 km from the head works (main part of the floodplain).

Water budgeting studies were organized in 2009 to estimate water releases to the river floodplain and groundwater recharge. The flow released to the floodplain and diverted to irrigation canals was measured three times per day. Then the groundwater discharge was calculated as the difference between the inflow and the outflow. The long-term groundwater recharge from the river floodplain was estimated using the data on river flow and the operating procedure of the head works. Finally, the correlation between groundwater salinity and volume of the groundwater recharge from the river floodplain was analyzed.

Modeling salinity of water of the Sokh aquifer

A three dimensional (3-D) finite-difference numerical model of the Sokh aquifer was developed using the US Geological Survey (USGS) software: the MODFLOW groundwater code (Harbaugh et al. 2000). The model top is the land surface topography. The area represented by the model covers 54.75 km x 50.25 km in a grid of 335 rows and 365 columns with a fixed cell size of 150 m x 150 m. The aquifer system is represented by three distinct geologic units—QIII, QII and QI. The three geologic strata are represented as five layers in the model:

- Layer 1: Soil surface to 20 m below ground level, at the head of the valley; the layer contains no water, but in the valley the water table is 0.5-3.0 m below ground level.
- Layer 2: From the bottom of layer 1 to an elevation defined by the base stratigraphic layer QIII, typically between 280 and 350 m amsl.
- Layer 3: The elevation of the base of this layer corresponds to the stratigraphic boundary between geologic units QII_1 and QII_2 and varies from 253 to 218 m amsl.
- Layer 4: The base elevation of this layer is marked by the stratigraphic boundary of geologic units QII_2 and QI_1 and varies from 218 to 125 m amsl.
- Layer 5: The base of this layer is set at 50 m amsl and is impermeable.

The boundary condition along the eastern and western edges of the model is no-flow. The northern boundary is taken as a constant head. Groundwater in layer 1 is unconfined while in layer 2 it is unconfined in the recharge but confined in the discharge zone, and in layers 3, 4 and 5 it is confined. The initial values of horizontal hydraulic conductivity and storage parameters were taken from the hydrogeologic surveys conducted in the study area by the GIDROINGEO (Mirzaev 1974; Borisov 1990; Gracheva and Miryusupov, 2006).

There are several sources of the groundwater recharge. The Sokh River and the BFC were included in the model to provide local recharge of the groundwater. There is also deep percolation flows from irrigation and rainfall. The BFC was included in the model as a “River.” Natural surface leakage along branches of the Sokh River was included as linear recharge. In other areas, areal recharge from irrigated lands predominates. Groundwater discharge in the spring zone is represented as the inflow to a 3-m deep surface drain with a constant flow depth of 1 m. During 1992-1996, the groundwater abstraction was 620 Mm^3/yr.; altogether 1,061 wells were in operation, including 661 for irrigation, 195 for drainage and 200 for domestic needs. Other hydrogeological input data used in the model given in Gracheva et. al.(2009).

Initial groundwater salinity data were from the regional database of the GIDROINGEO for 1992-1996 (Gracheva, personnel communication). The salinity of groundwater in the natural recharge zone is in the range of 150-300 mg/l. While moving from the natural recharge zone to the discharge zone, the salinity of groundwater increases in the upper layer to 2,500-5,500 mg/l. Salinity of the groundwater in the second layer varies in a wide range from 200-350 mg/l in the recharge zone to 1,500-2,500 mg/l at the edges of the cone where water moves through the less-permeable deposits and vertical flows dominate over the horizontal flows. In layers 3, 4 and 5, salinity of groundwater is low and varies from 200-300 mg/l in the natural recharge zone to 500 mg/l at the edges of the cone.

Salinity of water of the Sokh River recharging groundwater in the natural recharge zone is below 300 mg/l. More solids enter the groundwater system in the irrigated zone north of the BFC. The salinity of water percolating from the irrigated fields in this zone was calculated based on the concentration of soluble salts in the topsoil and then converted into the ESRI polygon shape file for reading from the MODFLOW. After compiling the model and input data, the model was calibrated using long-term groundwater levels and salinity data for 1992-1996. The results of the calibration are given below.

Calibration of hydraulic parameters of the model

Calibration of the model was undertaken using a combination of a manual method and PEST (Waterloo Hydrogeologic Inc. 1999). PEST software was run to improve values of the storage parameters using monthly observations of water heads for 1992-1996. A comparison between observed and the model-generated monthly water heads during the calibration stage showed a high correlation coefficient (CC) value at 0.969. Salinity change in the groundwater system was similarly estimated. Dispersion parameters were updated to improve convergence between the actual and calculated values of the salinity of groundwater. Final values of the dispersion coefficient of water were found to be 2.46 m for the management zone and 12 m for the discharge zone. A comparison between observed and the model-generated values of groundwater salinity during the calibration stage showed a CC value at 0.723. The obtained values of CC show that the model gives valuable estimates of water heads and salinity.

These calculations were carried out for the current conditions with the groundwater abstraction at the level of 657 Mm^3/yr. Then groundwater salinity predictions were made for conditions when the water abstraction will reach the maximum level (scenario 1: maximum). Scenario maximum (1) reflects the situation with increasing groundwater abstraction from 657 to 750 Mm^3/yr. or at 57% of the groundwater recharges. New wells are considered for the management zone where water quality is good and the hydrogeologic conditions favor groundwater development. Under this scenario irrigation shifts from canal use to conjunctive use. Most of the Naryn River summer flow becomes available for downstream uses.

Since the intensive groundwater abstraction in scenario maximum may affect the quality of groundwater, two different managed aquifer recharge scenarios are tested to maintain salinity levels at the groundwater system:

- Scenario 2: Scenario maximum, water saving in the recharge zone and banking the saved water in the subsurface aquifers and maintaining high water depth at the BFC (water saving and managed aquifer recharge).
- Scenario 3: Water saving, managed aquifer recharge and construction of shallow wells in the discharge zone (managed aquifer recharge and discharge).

The scenario of water saving and managed groundwater recharge (scenario 2) simulates conditions when water saving irrigation technologies is applied to the upper part of the basin and saved water is turned to the river floodplain and recharged into the aquifer in summer and winter. Adoption of the water saving irrigation technologies, such as cut back furrow irrigation, in the upper part of the basin on 48,000 ha of irrigated area allows reducing water diversions from the riverbed to 3,000 m^3/ha in winter and 9,000 m^3/ha in summer against the current 5,000 and 12,000 m^3/ha, respectively. The saved water is forwarded to the subsurface aquifers through the river floodplain to maintain the quality of the groundwater. The scenario of managed aquifer recharge and discharge (scenario 3) simulates increased groundwater abstraction in the management and the discharge zones by adopting 25-30 m deep shallow wells against the currently used 60-100 m deep wells. The groundwater abstraction amounts to 730 Mm^3/yr. The groundwater recharge is the same as in the previous scenario. This scenario is forwarded to reduce blending of the groundwater of different layers by adoption of shallow wells. Results of the modeling for different scenarios of the groundwater management are given below.

Results and discussions

Groundwater recharge from the floodplain of the Sokh River

In 2009, the river flow exceeding 40 m^3/s was released to the river floodplain during June-September. The water budgeting studies found that 31% of the flow released to the floodplain from June to September recharged the groundwater. Then, the long term river flow releasing to the river floodplain was estimated using the flow data monitored by the Sarykurgan Department of the Sokh-Syrdarya Irrigation System Administration at the Sarykurgan head works and given in Figure 3.

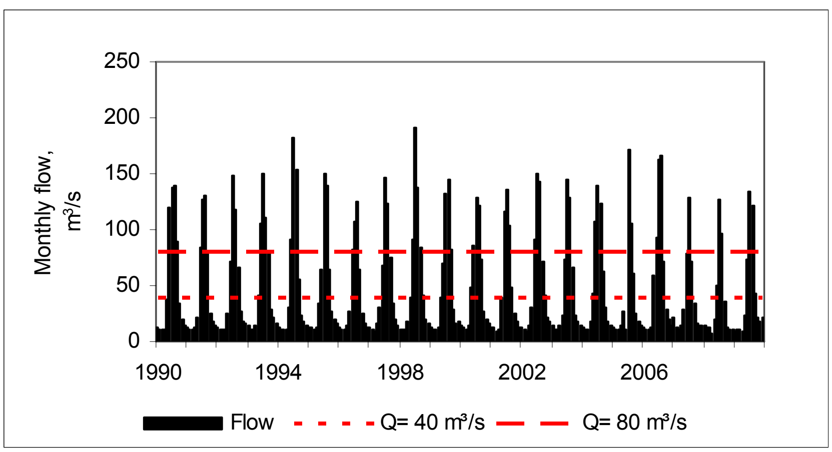


Figure 3. The long-term flow of Sokh River at the Sarykyrgan station (Shokirov, 2010)

The data presented in Figure 3 show that the volume of the river flow at the Sarykurgan head works varies from 650 to 1300 Mm^3/yr. The estimates show that the flow released to the river floodplain is in range of 414 to 584 Mm^3/yr. The estimates of the groundwater recharge from the river floodplain and the salinity of the groundwater are given in Figure 4.

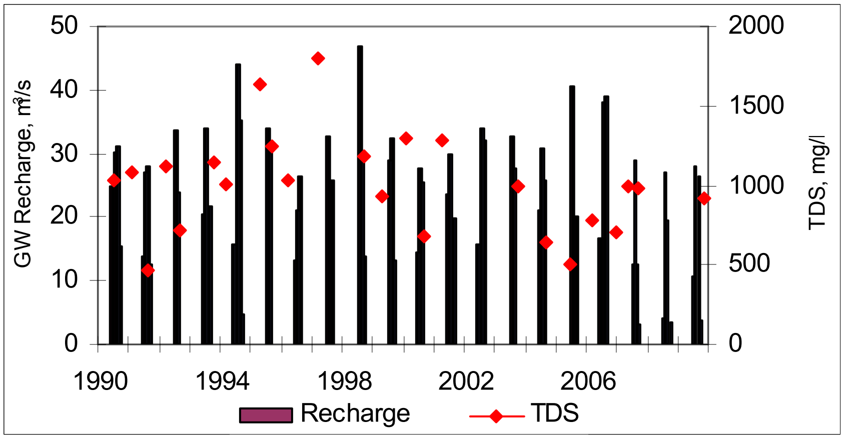


Figure 4. The groundwater recharge from the floodplain of the Sokh River and salinity of the groundwater expressed by total dissolved solids (TDS).

The data given in Figure 4 show that groundwater recharge from the floodplain amounts to 108-200 Mm^3/yr, against a total recharge at 680-750 Mm^3/yr. in the groundwater management zone. These data show that the groundwater recharge from the floodplain is significant and contributes to maintaining high-quality water in the Sokh aquifer. Figure 4 illustrates that the salinity of the groundwater expressed by total dissolved solids (TDS) is lowering in summer. For example, long term TDS of the groundwater for observation well 429 (Figure 2) averages to 1179 mg/l in March – May and to 876 in August – November, which indicates impact of the groundwater summer recharge from the river floodplain. However, data available on groundwater quality did not allow estimating the impact of the groundwater recharge from the floodplain on soil and groundwater salinity. That is why modeling studies were carried out that are presented in the next section.

Current conditions

Calculations for 1992-1996 using actual recharge and water abstraction volumes allowed obtaining a detailed analysis of water budgets for each calculated zone. The total groundwater recharge amounts to 1,114 Mm^3/yr. of which 60% is in the management zone. Leakage from the BFC is estimated at 122 Mm^3/yr. The groundwater recharge from the floodplain varies in the range of 108-200 Mm^3/yr. The groundwater abstraction was 657 Mm^3/yr of which 63% is in the management zone. In this zone, the groundwater abstraction amounts to 50% of the total recharge. The rest of the inflow forms the subsurface outflow to the discharge zone causing waterlogging. The low level of groundwater abstraction in the discharge zone contributes to a mixture of water of different layers in some locations only. In spite of the high drainage outflow from the area at 365 Mm^3/yr, high evaporation from the shallow water table causes accumulation of soluble salts in the topsoil in the discharge zone.

The data obtained show salinity buildup on the 69,557-ha area of the topsoil in the discharge zone. Salinity of the groundwater exceeds 2,000 mg/l on 75% and 5% of the area of the discharge and management zones, respectively, in the first layer. Salinity of the groundwater is in the range of 1,000-2,000 mg/l on 65% and 2% of the area of the discharge and the management zones, respectively, in the second layer. Salinity of the groundwater of the third layer is less than 300 mg/l, which is an indication of high-quality water for drinking water supply on 32% of the management zone area.

Scenario maximum

There are no significant changes in the groundwater recharge under this scenario; however, leakage from the BFC will come to 155-158 Mm^3/yr. The groundwater recharge from the floodplain at the same level is the same as under the current conditions. Increasing groundwater abstraction from 657 Mm^3/yr to 708 Mm^3/yr will decrease the groundwater discharge to the drainage to 350 Mm^3/yr. Evaporation from the water table will be 240 Mm^3/yr. Nevertheless, increasing groundwater abstraction will affect groundwater salinity. Salinity of the groundwater of the first layer will exceed 2,000 mg/l on 73% and 6% of the area of the discharge and the management zones, respectively. These data indicate a gradual increase of the salinity in the groundwater management zone, where groundwater abstraction was increased by 12%. Salinity of the groundwater of the second layer will be in the range of 1,000-2,000 mg/l on 66% and 2% of the area of the discharge and the management zones, respectively. These data also indicate a gradual increase of the salinity in the discharge zone of the second layer due to reduction of the subsurface inflow from the management zone. Salinity of the groundwater of the third layer will be less than 300 mg/l on 22% of the management zone area. Salinity of the groundwater in the third layer will exceed 1,000 mg/l on 18% of the discharge zone which is similar to the other scenarios. Lowering the water table will reduce salinity buildup in the topsoil. Salinity buildup will take place on 29% of the area in the discharge zone.

The data presented prove that attempts to prevent salinity buildup in the topsoil by increasing groundwater abstractions at 708 Mm^3/yr against 657 Mm^3/yr may cause an increase in the area with saline groundwater, with salinity above 2,000 mg/l in the first and second layers. Consequently, using the saline groundwater for irrigation may affect yields of the agricultural crops and correspondingly the incomes of the farms. That is why the managed aquifer recharge was modeled to avoid negative consequences of the intensive groundwater abstraction.

Scenario: Water conservation and managed aquifer recharge

It was assumed that adoption of the water saving technologies will reduce water intake from the riverbed in the upstream on an area of 48,000 ha to 3,000 and 9,000 m^3/ha in winter and summer, respectively, against 5,000 and 12,000 m^3/ha, respectively, of the current average. Adopting the water saving technologies will create conditions for increasing the groundwater recharge from the river floodplain. The groundwater will receive additional 269 Mm^3/yr of water from the floodplain. As a result of the rise in the water table, leakage from the BFC will decline to 127 Mm^3/yr, whereas the discharge into the drainage will be at level of 227 Mm^3/yr and evapotranspiration at 260 Mm^3/yr. The data obtained show that adoption of the water saving technologies will contribute to maintaining salinity of the groundwater that a) in the first layer, will exceed 2,000 mg/l on 73% and 4% of the discharge and the management zones, respectively, b) in the second layer will be in the range of 1,000-2,000 mg/l on 64% and 1% of the area of the discharge and the management zones, respectively, and c) in the third layer will be less than 300 mg/l on 48% of the management zone area. A positive salt balance is noted in the vadoze zone on an area of 54,247, or 22.7% of the total area against 29.1% under the current scenario.

The predictions suggest that water saving and increased groundwater recharge from the river floodplain allow maintaining low salinity of the groundwater. However, increased groundwater outflow to the discharge zone will cause a shallow water table and salinity buildup in the vadoze zone. These data show the need for additional measures to eliminate the salinity buildup in the topsoil. Shallow wells were tested in the groundwater discharge zone to eliminate blending of groundwater of different layers.

Scenario: Water saving, managed aquifer recharge and discharge

Increase in groundwater abstraction is achieved thanks to the construction of the shallow wells in the discharge zone, where it reaches 410 Mm^3/yr. As a consequence, groundwater discharge to the drainage decreases to 177 Mm^3/yr. and evapotranspiration to 191 Mm^3/yr. Because of the groundwater recharge from the river floodplain the groundwater resources could be available for irrigation in the discharge zone. The applied water saving strategy will increase groundwater storages in the management zone by 335 Mm^3/yr. which could be available for use in addition to 731 Mm^3/yr. of the abstracted water. These data show that groundwater can cover over 50% of the irrigation demand in the Sokh River Basin. Salinity of the groundwater a) in the first layer will exceed 2,000 mg/l on 75% and 3% of the area of the discharge and the management zones, respectively, b) in the second layer will be in the range of 1,000-2,000 mg/l on 65% and 2% of the discharge and the management zones, respectively, and c) in the third layer will be less than 300 mg/l on 46% of the management zone area.

Conclusions

The studies indicated the close linkage between groundwater recharge and discharge strategies and water and soil salinity. Adopting water saving irrigation technologies and using saved water for increased groundwater recharge through the floodplain of the river contribute to maintaining good quality of the groundwater. However, this practice may cause a shallow water table and salinity buildup in the groundwater discharge zone in the topsoil. Groundwater development for irrigation may allow avoiding salinity buildup in the topsoil. This approach may not be enough to eliminate the groundwater salinity increase in the discharge zone due to blending of groundwater of the different layers. The shift from deep to shallow wells in the groundwater discharge zone was found to be an effective method to decrease the blending of shallow saline water and deep freshwater.

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Keep it Fresh! Field test results of the fresh keeper concept including disposal of the RO membrane concentrate by deep well injection.

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Abstract
Salinization of fresh water wells is a continuing problem in the Northern parts of the Netherlands, and has led to the abandonment of some well fields. The fresh-keeper concept, i.e. simultaneous abstraction of fresh and upconing brackish water, may provide a remedy against salinization. When treated with reverse osmosis (RO), the intercepted brackish water may form an additional drinking water source.

To test the fresh-keeper concept in practice, a field pilot was started in 2009 at the abandoned well field Noardburgum, the Netherlands. The fresh keeper concept was combined with RO treatment of the brackish water and the deep well injection of RO concentrate. A single well with two extraction filters is used to simultaneously extract 50 m^3 h^{-1} fresh water and 50 m^3 h^{-1} brackish water. This brackish water is fed to the RO installation, rendering 25 m^3 h^{-1} permeate and 25 m^3 h^{-1} concentrate. The concentrate is disposed by deep well injection into a deeper, confined aquifer.

A monitoring program was initiated to study the different aspects of the field pilot, including changes in the fresh-brackish water interface, performance of the RO installation, clogging of the injection well, and water quality changes following concentrate injection. Results after 8 months of operation were promising: upconing of brackish water was reversed by the simultaneous abstraction of fresh and brackish water; scaling of the RO or clogging of the injection well did not take place, even though the concentrate water was supersaturated towards several minerals; and water quality changes in the target aquifer were limited.

The field pilot will run for another 4 months, until October 2010. It will then be decided whether the fresh-keeper concept will be applied at full-scale at the Noardburgum well field. In combination with a full-scale BWRO plant, the now abandoned well field will then produce 3Mm³ of drinking water per year.

Keywords: Brackish groundwater, fresh keeper, reverse osmosis, concentrate disposal, deep well infiltration, well clogging

Introduction
In the province of Friesland, in the Northern part of the Netherlands, salinization of fresh water wells is a serious problem for drinking water abstraction. For example, well fields Ritskebos and Noardburgum, with a total abstraction permit of 20 Mm^3 y^{-1} both suffer from salinization, leading to the abandonment of well field Noardburgum in 1993 and reduction of fresh water abstraction at well field Ritskebos, to a sustainable amount of 7 Mm^3 y^{-1} today. The cause of salinization is upconing in combination with lateral flow of brackish groundwater, which originates from Holocene transgression close to the north western border of the well field Noardburgum (Fig. 1).

Further salinization of still operated fresh wells may be prevented by interception of the upconing brackish groundwater. This so called fresh-keeper concept (Grakist et al. 2002; Kooiman et al., 2004) has been on the drawing table for several years now, but has not yet been applied in practice. The intercepted brackish water may be disposed of or, when treated by reverse osmosis, may serve as an additional drinking water source. A formerly threatened fresh water well may thus deliver more than the original quantity of drinking water, by using both the fresh and brackish groundwater.

Vitens Water Supply Company plans to re-open the Noardburgum well field by applying the fresh-keeper concept in combination with a Brackish Water Reverse Osmosis (BWRO) unit. Aim is to produce an additional 3Mm^3 y^{-1} of drinking water from this well field. It is expected that a number of deep abstraction wells will scavenge the advancing brackish groundwater thereby safeguarding the re-opened fresh wells. The extracted amount of brackish ground water will be treated by a RO installation operated under anaerobic conditions.

Before designing, permitting and constructing a full-scale BWRO plant, a field pilot was started to test and apply the fresh-keeper concept in practice. Issues addressed were, amongst others, the response of the fresh-brackish water interface to simultaneous abstraction of fresh and brackish water and the disposal (deep well injection) of the produced BWRO concentrate. Disposal of this concentrate is a serious problem at inland productions sites, as environmental laws prohibit disposal to the surface waters. Deep well injection into a deeper, more saline aquifer is the best solution for this problem (Stuyfzand et al., 2007; Stuyfzand and Raat, 2010).

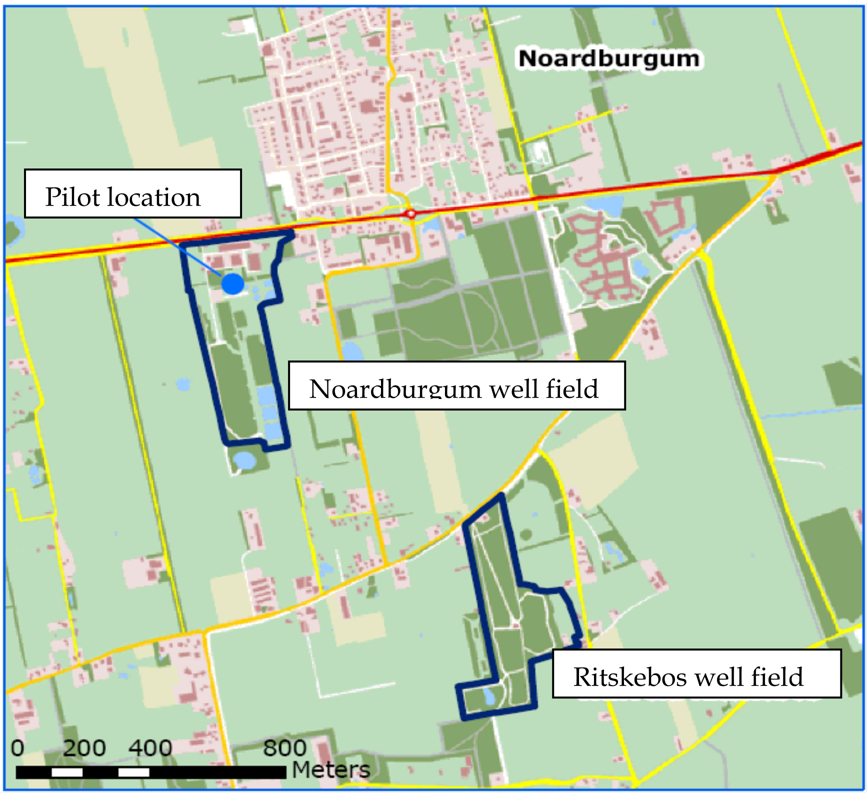

Figure 1: Province of Friesland, with location of the two well fields and production site.

Set up of the field pilot

The field pilot was started in October 2009 at the abandoned well field Noardburgum and combined the fresh keeper concept, anaerobic reverse osmosis and deep-well injection of the concentrate (Fig. 2). A newly drilled single well with two extraction filters at 60-80m and 130 -150m depth, respectively, was used to simultaneously extract 50 $m^3\ h^{-1}$ of fresh water and 50 $m^3\ h^{-1}$ of brackish water. The brackish water was used to feed the BWRO installation, which operated with a 50% recovery and without the use of antiscalants. The BWRO permeate (25 $m^3\ h^{-1}$) and the fresh water were transported separately to the drinking water production plant for further treatment and distribution. The BWRO concentrate was disposed by deep well injection into a confined aquifer at 180 m depth. During the 1 year test period, about 220,000 m^3 of concentrate will be produced and injected.

The aims of the pilot were:

1. To apply and test the fresh-keeper concept in practice, with special focus on the fresh-brackish water interface;
2. To test if anaerobic brackish groundwater is a reliable source for drinking water and if the anaerobic RO treatment process can be operated under stable conditions and cost effectively;
3. To test if concentrate disposal by deep well injection is technically feasible and sustainable from an environmental (water quality) viewpoint; and
4. To test if clogging of the injection well is an operational risk at this site.

A detailed monitoring program was started to study these different aspects of the pilot. Two observation wells, with filter screens in both the source and target aquifers, were installed at a distance of 12 and 26 meters from the injection well, in addition to an existing observation well at 43 meter distance. The fresh-brackish interface in the source aquifer was monitored closely using these observation wells and additional observation screens in both the abstraction and injection well. Water quality changes upon concentrate injection were monitored from the observation wells at 12, 26 and 43 meters. Clogging of the injection well is a serious operational risk and as such special attention was paid to gas clogging, biological clogging and clogging due to mineral precipitation of the supersaturated membrane concentrate. Well performance was monitored by high frequent pressure loggers in the infiltration well and nearby the observation wells.

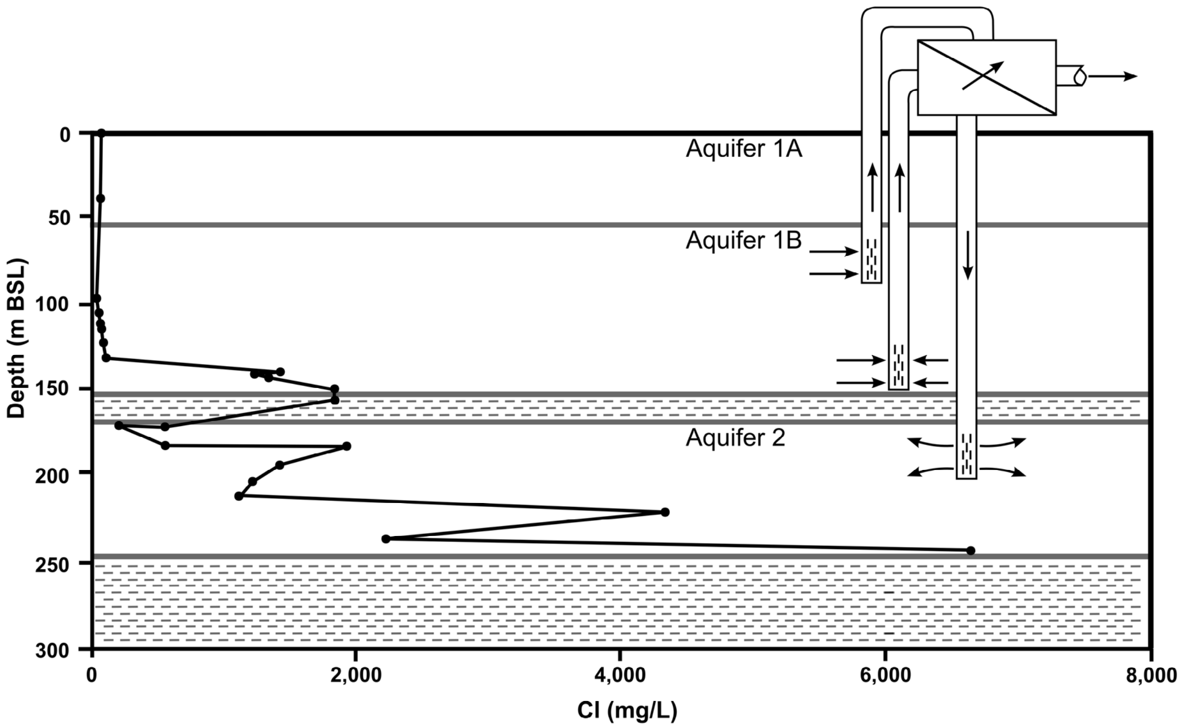

Figure 2.: Setup of the pilot

Results and discussion

Fresh-brackish water interface

The performance of the fresh-keeper and its effect on the fresh-brackish water interface was monitored by measuring the chloride concentration along the abstraction well, using both the abstracted fresh and brackish groundwater and additional observation filters along the well. For a selection of these filters, figure 3 shows the chloride concentrations in the first 8 months of the pilot, i.e. from October 2009 until May 2010. At the start of the pilot, chloride concentrations of the abstracted fresh and brackish groundwater were 42 and 1000 mg L^{-1}, respectively. In the first three months, the chloride concentration of the abstracted brackish water decreased rapidly to about 650 mg L^{-1}. Thereafter, concentrations decreased at a slower pace, to stabilize at a level of about 620 mg L^{-1} in May 2010. The chloride concentration in the fresh well decreased rapidly in the first month of the pilot, and reached a value between 35 and 40 mg L^{-1} in the following months. During the test period, the largest changes in chloride concentrations were observed in the observation well at 42 m distance from the injection well. Filter 3 of this well (Fig. 3), with a screen depth of 140 m, showed a decrease in chloride from 1200 mg L^{-1} at the start of the pilot to 190 mg L^{-1} in May 2010. This indicates that the fresh/brackish water interface dropped below 140 meter depth at this location. In summary, these results indicate that the fresh-keeper concept operates as intended, and that simultaneous abstraction of fresh and brackish water may provide a remedy to salinization of fresh water wells.

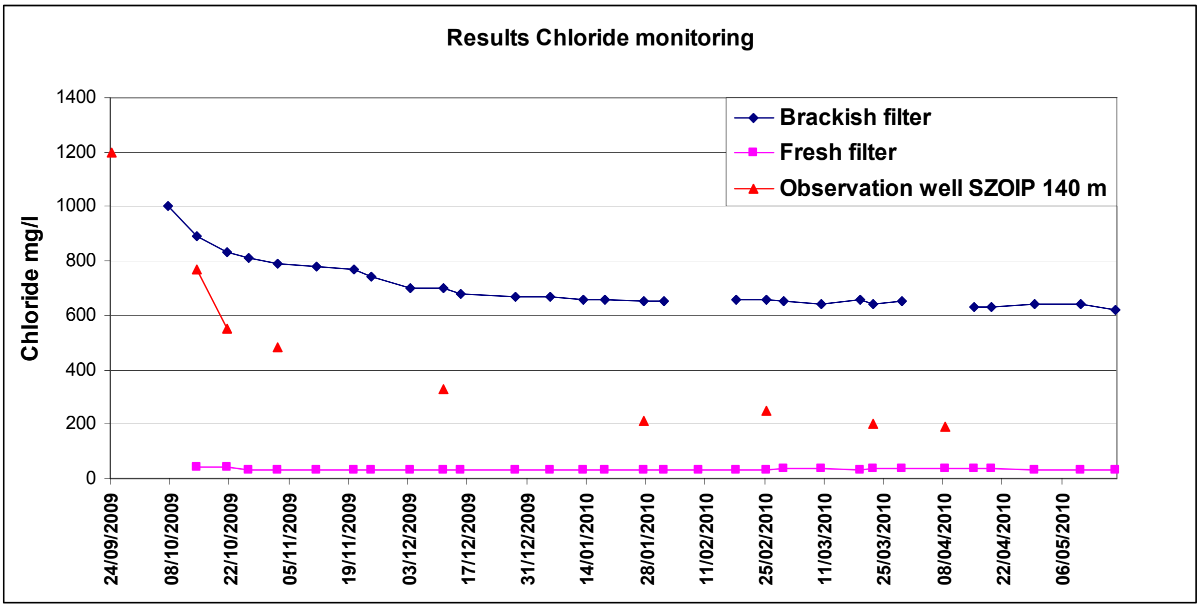

Figure 3: Cloride monitoring result during the first months of the field test period

Performance of the BWRO installation and injection well

Fouling or scaling of the RO membranes due to mineral precipitation forms an operational risk for the BWRO installation. Mineral precipitation may be prevented by using antiscalants like polyphosphates, but this was not applied in the pilot as to allow the injection of the BWRO concentrate in the subsoil. Instead, the RO recovery was kept at a relatively low level of 50%, thus preventing severe oversaturation of the concentrate.

Analysis of the BWRO concentrate indicated that the concentrate was supersaturated towards carbonate (calcite, dolomite, siderite) and phosphate (vivianite) minerals. Despite this oversaturation, scaling of the membranes did not occur, as indicated by the pressure drop over the membranes, which remained stable at $0.85 \cdot 10^{-8}$ m s^{-1} kPa^{-1} throughout the pilot. In addition, this result showed that the BWRO was indeed operated under strict anaerobic conditions, as intrusion of oxygen would have immediately resulted in the formation of iron flocs in the iron(II) rich feed water.

Similar to scaling of the RO membranes, clogging of the injection well is a serious risk for injection wells. Injection well clogging was monitored with use of high-frequency pressure loggers both in the well itself, the gravel pack and the nearby observation well. Results from these loggers showed that the resistance against water flow did not increase during the pilot, indicating an absence of clogging both at the filter screen as in the target aquifer. As such, like in the RO installation, mineral oversaturation did not lead to precipitation and subsequent operational problems. In addition, this result shows that clogging due gas bubble formation (high CH_4 concentrations) or microbial growth seems absent.

Water quality changes in target aquifer

Water quality changes in the target aquifer were monitored from the three observation wells, at 12, 26 and 43 m distance from the injection well. The injected water moved through the aquifer unhindered, without clear interactions with the aquifer matrix or the native groundwater (Table 1). Chloride and electrical conductivity were excellent indicators of breakthrough of the injectate in the observation wells. Cations were only slightly retarded, indicating that exchange reactions were limited. Shortly after breakthrough, mineral saturation levels increased rapidly to values equalling those of the concentrate, again indicating that mineral precipitation is absent, despite oversaturation towards carbonates and phosphates. Possible causes are, among others, kinetic hindrances and unaccounted complexation by dissolved organic matter.

		Abstracted water		Concentrate	Observation well at 26 m distance	
		Fresh water	Brackish water		Initial	After breakthrough
pH		6.98	6.8	7.01	6.85	6.95
EC	mS/m	61.4	242	453	226	481
Cl	mg/L	34	650	1200	670	1400
HCO3	mg/L	383	345	678	331	666
TOC	mg/L	7.2	4.3	9	4.7	9.1
Na	mg/L	21.4	87.7	170	71.4	190
K	mg/L	1.7	3.4	6.2	3.28	6.13
Ca	mg/L	110	359	710	324	737
Mg	mg/L	9.38	30.1	58.8	27.3	57.3
Fe	mg/L	13.7	39.6	75.6	29.7	82.8
Mn	mg/L	0.225	0.82	1.62	0.738	1.61
Ba	mg/L	0.047	0.251	0.504	0.226	0.535
Sr	mg/L	0.33	1.26	2.49	1.11	2.47
SiO2	mg-Si/L	16	16	30	14	25

Table 1. Water quality of the abstracted fresh and brackish water, the RO concentrate, and at 26 m distance from the injection well, both before and after breakthrough of the concentrate.

The absence of mineral precipitation both in the BWRO installation, the injection well and the target aquifer, indicates that the current RO recovery could be increased above the current 50%, without introducing high risks on membrane scaling or injection well clogging. To which extent the recovery may be increased, will be tested in the near future.

Conclusions

The pilot at the Noardburgum well field is the first on-site application of the fresh-keeper concept. Results after 8 months of operation are promising and lead to following, preliminary conclusions:

1. The fresh-keeper concept is a successful remedy against salinizaton of fresh water wells, leading to stabilization or even lowering of the fresh-brackish water interface;
2. Direct RO treatment of anaerobic groundwater is applicable (i) for feed water that is high in iron, and (ii) without the use of antiscalants;
3. Deep well injection of BWRO concentrate is technically feasible and results in moderate water quality changes in the target aquifer. It thus provides a sustainable solution for RO concentrate disposal.
4. RO recovery levels can be increased above 50%, without causing membrane fouling or injection well clogging.

The field pilot will run for another 4 months, until October 2010. After the pilot, Vitens Water Supply Company will decide whether or not to re-open the Noardburgum well field by applying the fresh-keeper concept at full scale. In combination with a full-scale BWRO plant, the now abandoned well field will then produce 3Mm³ of drinking water per year.

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Numerical Innovations for Improved Computational Efficiency in the Simulation of Basin-Wide Integrated Surface-Subsurface Flow: Testing of Sub-gridding and Sub-timing Methods

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Abstract
Sub-gridding and sub-timing techniques are a recent innovation in numerical modeling. The benefit of these advanced methods lies in their ability to provide computational advantage for simulation of challenging hydrologic problems. These methods are particularly adept for problems that require simulation across large and multiple scales and/or over large simulation periods of time. Therefore problems such as the simulation of basin-wide integrated surface and subsurface flow, including managed aquifer recovery are well suited for sub-gridding and sub-timing application.

Integrated surface-subsurface flow simulations typically can be fraught with temporal over-discretization problems because the characteristic response times for stream flow, overland runoff and subsurface flow can differ by orders of magnitude. The sub-timing approach can apply different time step sizes to the sub-domains that have different response times. Sub-timing is well suited for locations where the movement of water is rapid, such as within the overland flow domain, near pumping well screens or within discrete fractures.

The local and regional scale aspects of integrated problems present spatial discretization challenges. The sub-gridding approach permits high resolution detailing in localized sub-regions of the model without the computational burden of extending the refinement to the model boundaries, whether vertical or horizontal. Sub-gridding in the vicinity of managed aquifer recharge facilities, where rates of piezometric head change can be significantly higher than elsewhere in the aquifer, is considered a valuable contribution to assessing or designing managed aquifer recovery operations.

Sub-gridding and sub-timing methods were tested using the HydroGeoSphere model for a regional scale, simplified integrated surface-subsurface model of the San Joaquin Valley and Tulare Basin of Central Valley of California. Results indicate the sub-timing approach can significantly improve the solution efficiency for solving integrated surface-subsurface water systems. Additional efficiency can be achieved by dynamically adjusting the sub-timed nodes and the number of sub-time steps over the simulation period for a given problem. With respect to sub-gridding, the method can achieve localized high resolution results at a computational savings, compared to cases of more extensive mesh refinement. Sub-gridding is most suitable when the regions to be sub-gridded are localized portions of the overall domain.

Keywords: California, HydroGeoSphere, integrated, modeling, sub-gridding, sub-timing.

INTRODUCTION

Sub-gridding and sub-timing techniques are a recent innovation in numerical modeling. The benefit of these advanced methods lies in their ability to provide computational efficiency for simulation of complex flow and transport processes in fully-integrated surface and subsurface water systems. Benefits of these techniques have been demonstrated in small-scale flow domain study cases, although their utility for application to large scale flow domains has not been thoroughly evaluated. These techniques are particularly adept for problems that require simulation of flow and transport processes over basin-wide large and multiple spatial scales of and/or large simulation periods of time; therefore basin-wide integrated surface-subsurface flow systems, including managed aquifer recovery problems, are well suited for sub-gridding and sub-timing application. Integrated surface-subsurface flow systems have capability to account for flow within each surface and subsurface water regimes and across the interface of the water regimes. It is noted that surface-water flow rate is faster than flow rate through the subsurface water regime is much slower than that in the surface-water flow rate.

The objective of this research is to investigate the scalability of the benefits of sub-gridding and sub-timing methods in their application to basin wide integrated surface-subsurface flow systems. The results of this work will provide insight into the degree to which these methods may help alleviate the computational burden of applying sub-gridding and sub-timing algorithms to field problems, and thereby provide advantage by increasing the utility of integrated models for managed aquifer recovery, and addressing water management issues related to water supply reliability, water quality and ecosystem health in an integrated manner.

This application example described herein is directed towards the fully-integrated surface and subsurface conditions in the San Joaquin Valley. A preliminary basin-wide model was developed. Basin-wide model development and testing focused on the San Joaquin Valley (SJV) and the northern portion of the Tulare Basin as shown in Figure 1. This basin-wide model testing is based on the conceptual model development provided herein. The fully-integrated surface-subsurface simulator, HydroGeoSphere (HGS), was used for all simulations.

HydroGeoSphere Development

HGS is the product of collaborative research work among BOR, University of Waterloo, Université Laval, and HydroGeoLogic, Inc. (HGL) (Panday et al, 1993, Sudicky et al, 1995, Therrien and Sudicky, 1996, Panday and Huyakorn, 2004). HGS is a fully-coupled surface/subsurface numerical flow and transport computer program developed as a numerical tool for water-resource, managed aquifer recovery (MAR) and ecosystem analyses and management. It is a physically-based, spatially distributed model designed to address surface-subsurface flow and their interactions.

HGS provides simulation options for either finite difference or the control volume finite element approaches to simulate coupled surface and subsurface flow and transport. Fully 3-D simulations of variably-saturated fractured or granular aquifers may be performed. HGS provides several discretization options ranging from simple rectangular and axisymmetric domains to irregular domains with complex geometry and layering. Mixed element types provide an efficient mechanism for simulating flow and transport processes in fractures (2-D rectangular or triangular elements), pumping/injection wells, and streams or tile drains (1-D line elements). External flow stresses can include specified rainfall rates, hydraulic head and flux, infiltration and evapotranspiration, drains, wells, streams and seepage faces.

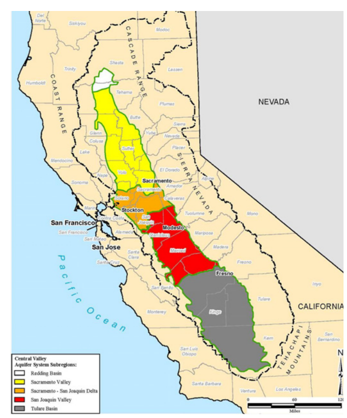

Figure 1. San Joaquin Valley (SJV) and the northern portion of the Tulare Basin.

Development of HGS formally commenced in 2003, under funding by the BOR. During this initial work, HGS emerged as a linkage between the subsurface model FRAC3DVS (Therrien et al., 1993) and the surface water module from MODHMS (Huyakorn and Panday, 2004). Furthermore, the HGS model was enhanced to include capability to account for hydrologic processes of tile drains and evapotranspiration

(ET). In 2005, sub-gridding and sub-timing algorithms were implemented within HGS to further improve its computational efficiency and, thereby, broaden its utility.

Multiple HGS training workshops have been held to provide training to model users and stakeholders. The workshops have featured lectures and hands-on modeling demonstration on topics including surface and subsurface water interaction, approaches for coupling surface and subsurface water regimes and more. In a 2008 workshop in Sacramento, recent enhancements to HGS were introduced, including thermal process simulation and plans for a linkage to a water allocation optimization and simulation model.

Water Issues and Modeling in the Central Valley

Water supply reliability, water quality, and ecosystem health are key issues impacting water resources in the San Joaquin Valley and pose challenges for sustainable water-resource management. To address these issues, numerous projects and programs have been implemented by Federal and State agencies such as the BOR, the California Department of Water Resources (DWR) and the U.S. Geological Survey (USGS).

The challenges of sustainable water-resource management are further exacerbated by climate change and growing water demands. Sustainability and vulnerability of surface and subsurface water supplies, given increasing demand and ageing infrastructure, are of interest to many water professionals. In addition, water quality and ecosystem health will continue to be threatened by current and future natural and anthropogenic activities.

Many Federal and State programs have utilized numerical modeling to gain insight into surface and sub-surface flow system dynamics. These modeling studies include the USGS National Water-Quality Assessment Program–SJV Model (Phillips et al., 2007), USGS's Regional Aquifer-System Analysis (Williamson et al., 1989; Phillips and Belitz, 1991), BOR's WESTSIM model and S.S. Papadopulos and Associates' MODFLOW model of the SJV. However, these previous modeling studies presented overly simplified representation of key physical processes, including surface-subsurface flow interactions. HGS overcomes the oversimplications described above by accounting for 3-dimensional variably-saturated subsurface flow and solute transport, and 2-dimensional overland (surface) flow and transport.. The key feature of HGS is that the governing equations for surface and subsurface flow and transport are solved simultaneously so that dynamic interactions between surface and subsurface flows are accurately simulated. This provides a fundamental distinction between HGS and the models that treat surface and subsurface processes separately. Moreover, with enhanced sub-gridding and sub-timing capability, HGS permits basin-wide evaluation of integrated surface-subsurface flow systems, without compromising solution accuracy and detail in localized domains of interest.

METHODS

A simplified representation of the basin-wide surface-subsurface integrated flow system within the SJV and northern portion of the Tulare Basin was developed to test the sub-gridding and sub-timing methods. The numerical representation of the basin-wide model is somewhat simplified to maintain emphasis on the testing of sub-gridding and sub-timing. A more detailed treatment of physical processes, input parameters and model application is reserved for a future model development initiative.

Model Domain and Discretization

The areal extent of the basin-wide integrated flow model coincides with the SJV boundary on the north side and a significant portion of the east and west sides; the Tulare Basin on the southern portions of the east and west side; the groundwater basin boundary; and the Kings River on the south side. The model boundary encompasses a combined area of 6,645 square miles (17,212 square kilometers). Vertically, the model extends from the ground surface through to the top of the marine rocks and consolidated continental deposits.

A 30-minute USGS digital elevation model described defines the ground surface in the model. A smoothed representation was used to enhance computational performance. The smoothed surface was attained through averaging nodal property elevations at surrounding connected nodes to create a single nodal elevation. Because of loss of detail along major river courses in the smoothing process, the surface elevation of the nodes associated with six major rivers are lowered by 3 meters in the smoothed model. The bottom surface of the model is set below the smoothed ground surface by the depth of the unconsolidated continental deposits delineated by Williamson et al (1989).

The two-dimensional overland flow domain is discretized into 27,373 nodes and 26,718 rectangular elements with a uniform grid spacing of 800 by 800 meters. The three-dimensional subsurface domain is discretized into 218,984 nodes and 187,026 block elements with the plani-metric dimensions of the overland flow domain.

The model consists of seven evenly-spaced layers extending from the ground surface to the bottom of the model, with an average layer thickness of approximately 130 meters. This average thickness results in an average cell aspect ratio of six.

The Corcoran clay is not explicitly represented as a distinct layer separating the Upper Unconfined Aquifer and Lower Semi-Confined Aquifers. This is consistent with the conceptual model of the San Joaquin Valley, whereby the distinct Upper and Lower aquifers in the SJV are believed to be a single heterogeneous aquifer with unconfined conditions near ground surface and increasing confinement with depth. Based on this conceptual understanding, the confining conditions are not due to one main confining unit in the Corcoran Clay, but are the result of multiple overlapping fine grained lenses and beds.

Model Domain and Discretization

According to previous modeling efforts, the unconsolidated overburden includes varying amounts of sand and gravel, clay, sandy clay, silt, and silty sand (Williamson et al., 1989; Phillips et al., 2007). For the purposes of testing sub-timing and sub-gridding, the subsurface is considered to be a single homogeneous, anisotropic aquifer system.

For subsurface layers the equivalent homogenous horizontal hydraulic conductivity is selected to be 0.28 centimeters per day with a vertical value equal to 3.7×10-3 centimeters per day. Storativity and porosity are estimated to be 1.0 x 10-7 m-1 and 0.34, respectively. Storativity is the volume of water released from storage per unit decline in hydraulic head in the aquifer, per unit area of the aquifer. The van Genuchten retention properties are based on default HGS values representing the Borden sand used for the simulation of the Abdul (1985) rainfall-runoff problem.

Surface Properties

A Manning's roughness coefficient of 0.02 was assigned throughout the surface-water flow domain with a reduced value of 0.002 along the river elements. These are deemed to be reasonable values inferred from literature. Rill and obstruction storage heights of 0.037 and 0.085 meters respectively are used. A coupling length of 0.1 meters was used.

Boundary Conditions

The areal extent of the aquifer system in the unconsolidated overburden is assigned based on groundwater divides and potentiometric surface contours described in Section 2. The aquifer system is bounded by no-flow boundaries consisting of consolidated and crystalline rocks that allow minimal groundwater flow.

Lateral boundary conditions for the surface water component are required to establish upstream and downstream surface flow conditions for rivers. The upstream boundaries are established using specified volumetric flow rates while the downstream boundary is governed by a critical depth boundary condition. Upstream boundaries are applied for the following rivers: Stanislaus River, Tuolumne River, Merced River, San Joaquin River, Chowchilla River, and Fresno River. The average annual flow rates are based on data provided by the USGS National Water Information System.

An annually averaged value of 284 millimeters per year is applied uniformly to the top of the surface water as specified rainfall. This value is based on the average monthly 1961-1990 PRISM data set that provides the temporal and spatial distribution of precipitation. This data set exists in raster format with a resolution of 1.2 miles (2 kilometers).

RESULTS AND DISCUSSION

The methodology and results for obtaining a steady-state simulation from the regional model is detailed in this section. Given the lack of representation of subsurface water injection/extraction, agricultural return flow, and surface water structures (i.e. canals and reservoirs), the model simulated can be considered to represent pre-development conditions. Analyses and comparisons of the models facilitate the diagnosis of errors that may arise during implementations, provide an understanding of hydrological system responses, and provide a basis of comparison for sub-gridded and sub-timed models.

As a starting point for subsequent transient simulations, steady-state solutions are obtained given constant initial heads/depths through long-term transient simulations. The simulations are implemented using a staged approach. All simulations are performed using a variable time-stepping procedure, and steady-state conditions are assumed based on changes in the rates of fluid exchange and accumulation. As a starting point for a subsequent transient simulation, a steady-state solution is obtained in stages.

This involves three steps:

- A steady-state solution for surface water flow using an estimated value of the effective precipitation rate equal to 284 millimeters per year;
- A second steady solution for groundwater flow using the same infiltration rate as the effective precipitation rate and with specified head values at the river nodes equal to 2 meters above the bottom elevation of the river nodes, and;
- An integrated surface water and variably-saturated subsurface flow solution with the estimated effective precipitation rate being applied to the surface domain. The effective precipitation applied is assumed to equal the mean rainfall minus the mean ET.
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The purpose of the steady-state solutions based on transient simulations (see previous subsection) is two fold: to compare the integrated simulation results (both heads and flows) to the results of surface-only and subsurface-only simulations, and to illustrate the characteristic hydraulic response times in the different flow regimes.

For the surface water flow simulation, the estimated effective precipitation is applied to the entire overland domain, and the average measured stream flow rates are applied as inputs to the upstream ends of the six rivers. The surface of the domain is assumed to be completely dry with zero water depth at the initial time (t = 0). Simulated stream flows take approximately 10 days to transmit water from the inlets to the outlet and the overland flow component requires approximately 100 days to run off the system.

For the subsurface flow simulation, the estimated net precipitation is applied to the top surface of the three-dimensional simulation domain as an effective infiltration. A specified head boundary condition is applied to the river nodes with the head specified at 2 meters above the streambed elevation. The initial head is set to 5 meters AMSL throughout the entire simulation domain. Steady state is reached after about 105 days.

For the coupled surface and subsurface flow simulation, the same boundary conditions as applied to the surface-only simulation (the estimated effective precipitation and average measured stream inflow rates) were applied to the surface domain. The final steady-state simulation results from the surface-only and subsurface-only transient simulations were used as an initial condition. Figure 2 shows the distribution of the final steady-state surface water fluxes, fluid exchange rates between surface and subsurface, and the subsurface water saturations.

In the coupled simulation, a portion of the net precipitation applied to the surface infiltrates into the subsurface and the remainder of water flows over the land surface, depending on the degree of saturation in the subsurface. As a result, the saturation in the subsurface-only simulation is overestimated compared to the results of the coupled simulation.

For the first few tens of days, the total exfiltration rate is greater than the total infiltration rate because the steady-simulation results from the subsurface-only simulation overestimated the water saturations in the subsurface, resulting in excess outflow from the system. At steady state, the infiltration and exfiltration rates are similar, and slightly more than one half of the total inflow applied to the land surface infiltrates into the subsurface. This implies that the remainder of water flows over the land surface.

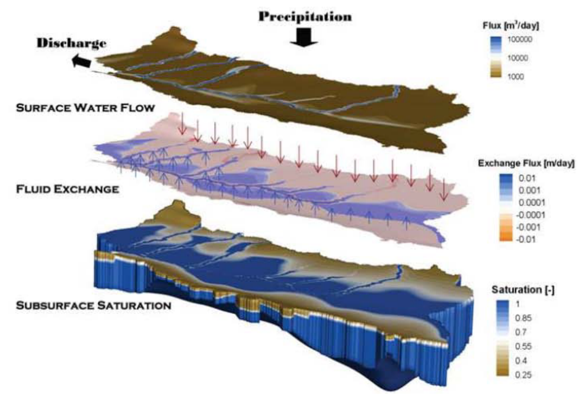

Figure 2. Distribution of the final steady-state surface water fluxes.

From the three transient simulations to steady-state, it was illustrated that the characteristic response time for river drainage, overland flow, and subsurface flow ranges over four orders of magnitude from tens of days to about 105 days. It is also evident that important features in the integrated flow regime characteristics cannot be captured by performing a surface-only or subsurface-only simulation.

The time frame required to reach steady state for the case of integrated flow may be subject to uncertainty as the initial guess for the last simulation is not the same as those of the surface water only and groundwater only simulations. Even starting with the respective equilibrium conditions, it took close to 100,000 days to achieve the integrated equilibrium.

Overview of Sub-gridding and Sub-Timing
Many numerical models of groundwater flow and transport use finite-difference or finite-element methods to discretize and solve the governing flow and transport equations. These models often require highly refined meshes in areas of interest where gradients vary rapidly in space to obtain adequate spatial resolution and accuracy. The need for a locally refined mesh is based on practical requirements:

- To accurately represent steep hydraulic gradients near river banks, pumping or injection wells;
- To represent local topography, hydrogeologic features or hydraulic property distributions as accurately as practicable; and
- To accurately represent sharp fronts in contaminant transport.

Use of a fine mesh over the entire domain can be computationally intensive, and in some cases intractable (Mehl and Hill 2002). Variably spaced meshes can lead to elements with large aspect ratios and refinement in areas where such detail is not needed. In addition, fine discretization is advantageous within models in situations where previously constructed mesh geometries render redesign of the entire mesh impractical or not feasible. An alternative is to use localized grid refinements limited only to the area of interest, and there are two general approaches in this regard: model-in-model and direct embedment.

A model-in-model or telescopic approach, entails the use of successively smaller-scale models, e.g. basin-wide, regional, local, and site-specific models. In Ward et al. (1987), the approach for inter-scale information transfer has involved linear interpolation, which is associated with several disadvantages. Firstly, coupling between two model meshes occurs only in one direction: from the large mesh to the small mesh. Because there is no feedback from the small mesh to the large mesh, nonlinear analyses based on iterative solution techniques are not possible, and significant discrepancies can occur in fluxes or state variables (whichever are not used to couple the meshes) at the model interface.

The direct embedment approach is more appropriate for finite element methods than for finite-difference methods. For finite-difference methods, the choice of neighboring nodes for finite-difference stencils (5 nodes for two dimensions and 7 nodes for three dimensions) becomes less obvious. Some techniques (e.g., von Rosenberg 1982; Edwards 1996) resulted in poor matrix properties (e.g., asymmetric matrices for the flow equation) and conditional diagonal dominance. These poor properties may not help in reducing computational effort and may even result in divergence of numerical solutions. Assumptions to circumvent these problems may not be realistic (Mehl and Hill 2002). To overcome these problems, some researchers (e.g., Szekely 1998; and Mehl and Hill 2002) developed a variety of hybrid approaches (combinations of model-in-model and direct embedment approaches). However, these approaches require iterative coupling of boundary conditions at model interfaces in an explicit manner (Guvanasen, 2007).

The sub-gridding in HGS is based on the direct embedment approach. Thus sub-gridding eliminates the need for nesting multi-scale models, while providing all the advantages of a locally-refined model. HGS handles spatial sub-gridding for 3-dimensional and 2-dimensional planar elements. Theoretical basis for sub-gridding these elements are described in Guvanasen (2005). Illustrations of sub-gridding are shown in Figure 3.

Sub-gridding and sub-timing may be used for either finite difference of finite element schemes. Matrix assembly and connectivity issues are related to the numerical method selected for spatial discretization and can greatly affect simulation times. An integrated finite-difference discretization of the governing equations generates a 7-point spatial connectivity for a 3-D system (only principal direction connections). A finite-element discretization of the governing equations generates a 27-point matrix connectivity (since it includes the diagonal connections). Other things being the same, a finite-difference computer program will therefore have almost 3-times fewer non-zero columns in the matrix than a finite-element computer program, with associated 3-fold reduction in computations for assembly and solution – possibly even more savings when optimized matrix ordering schemes are used to reduce the system of equations. Furthermore, some finite-element computer programs implement numerical integration for matrix assembly, which can be extremely time consuming. In addition, there are negative transmissibility issues that can arise from the finite-element connectivity which may hinder convergence. Granted that finite-element methods allow for a higher order of spatial discretization, however, sub-gridding negates this advantage, while providing the same accuracy – yet faster run times with less random access memory consumption.

Hydrologic responses in surface domains, especially in channels, of integrated hydrological systems are quick relative to subsurface domains. The difference in responses is generally orders of magnitude and makes problems difficult, if not impossible, to solve. In the integrated simulation of transient surface and subsurface flow and transport, the accuracy of the solution is controlled by the time step size applied to a spatially-discretized domain. In general, smaller time step sizes result in more accurate solutions. The traditional finite difference approximation for temporal derivative in mass accumulation term in flow or solute transport equations utilizes a fixed value of time step size at a given time and in the entire computational domain. Therefore, in order to achieve a certain level of accuracy in any part of the domain, all the spatially distributed finite difference grids or finite element meshes need to be adjusted by one time step size value that can satisfy the least tolerant accuracy requirement. Therefore, for integrated flow and transport simulations, time stepping is dominated by surface domains, where the response is quick relative to the other domains. This leads to temporal over-discretization in the domain of relatively slow change (for example, low permeability subsurface materials), where the accuracy requirements are more tolerant.

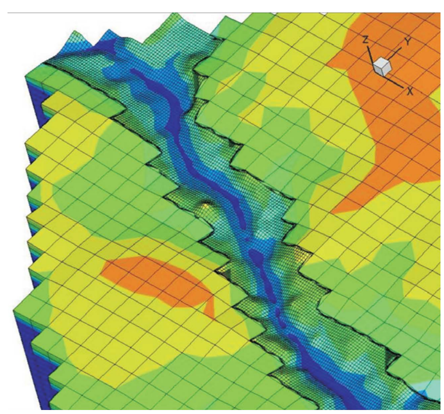

Figure 3. Illustration of sub-gridding.

Sub-timing is a strategy to make use of different time-step sizes for the sub-domains, where the rates of temporal change are different and thus the different temporal discretizations are required for a certain level of simulation accuracy [Singh and Bhallamudi, 1996; Gravouil and Combescure, 2003; Bhallamudi et al., 2003]. In the sub-timing approach, if the response in head/water depth or concentration is quicker in a certain part of the computational domain (for instance, in the channels), smaller sub-time step size can be assigned only to the sub-domain, keeping the larger time step size in the other part of the domain to circumvent the temporal over-discretization problems. Thus the sub-timing strategy is efficient for integrated surface and subsurface flow and transport.

A method of sub-timing was suggested by Singh and Bhallamudi [1996] for explicit time marching procedure. In this approach, time step size in each spatial element is constrained by a certain stability condition because of its explicit nature (e.g. diffusion or Courant number constraint, depending on the problem) as well as the accuracy requirements. To overcome the strict stability constraints in a portion of the simulation domain, a hybrid implicit-explicit type approach was suggested by VanderKwaak [1999]. In this hybrid approach, the implicit time marching is applied only to the part of the domain where the stability constraints are dominant over accuracy requirements, while the explicit scheme is applied to the other portion of the domain. As a result, it is demonstrated that the hybrid approach could alleviate the computational efforts in each time step by reducing the matrix size to be solved, but it still requires the same time step size for the whole domain.

The implicit sub-time stepping approach, suggested by Bhallamudi et al. [2003], is unconditionally stable because of its implicit nature and can handle the temporal over-discretization in the sub-domain of relatively slow response. While the implicit sub-timing strategy can directly address the temporal over-discretization problem, the numbers of unknowns to be solved increases in each global time step, as more unknowns arise in the sub-timed nodes (Bhallamudi et al., 2003). Thus, the efficiency of implicit sub-time stepping strategy depends on the number of sub-timed nodes, compared to the total number of nodes and the characteristics of the matrix solver.

As suggested by Bhallamudi et al. [2003], the implicit sub-time stepping approach is most suitable for the problems where activity and interest is high in only a small portion of the computational domain such as integrated surface and subsurface hydrologic simulations and may generally apply in simulations involving intermittent MAR sites. HGS contains this implicit sub-time stepping approach (Park et al., 2009), suggested by Bhallamudi et al. [2003], and includes an automated adaptive time-stepping strategy to improve its efficiency (Therrien et al., 2009).

Basin-Wide Scale Testing: San Joaquin Valley and Tulare Basin Model

The general model grid includes an identical horizontal grid for all model layers, whereas the use of sub-gridding permits additional localized horizontal grid refinement in selected model layers. To evaluate the sub-gridding methodology in the SJV and northern portion of the Tulare Basin model, the San Joaquin River and other major rivers are further discretized by sub-gridded cells to provide enhanced localized resolution of the interface between surface and subsurface flow systems. This results in more accurate representations; as an example, the sub-gridded rectangular cells of 82 feet (25 meters), to provide the capability to capture the meandering river paths within the randomly selected portion of the SJR.

The demonstration of the sub-gridding scheme is performed by comparing the numerical model without sub-gridded elements, referred to here as the non-sub-gridded (NSG) model, to a sub-gridded version of the model, referred to here as the sub-gridded model. With the original NSG model, sub-gridding is used to provide additional resolution in the vicinity of major river courses. The non-sub-gridded model utilized nearly 12,000 elements and less than 1 hour to reach a steady-state condition. In comparison, the sub-gridded model utilized 55,000 elements and achieved steady-state in approximately 10.5 hours.

In general, the sub-gridded and non-sub-gridded models produce similar trends in terms of water balances. Given the same inflow, the differences in outflow rates are negligible. However, the effects of sub-gridding are pronounced in infiltration/exfiltration rates. In the given application, the sub-gridded model simulated a lower rate of fluid exchange between the surface and subsurface domains. The difference at steady-state is on the order of 106 m3/day, approximately 10 percent of the infiltration/exfiltration rates at steady-state. Given that the sub-gridding scheme is only applied along rivers and the improved connectivity of the stream achieved through the use of sub-gridded elements, sub-gridding is likely to be improving the representation of surface and subsurface water flow interactions. A comparison to a model discretized to the level of the sub-grids can demonstrate any accuracy improvements. However, the computational requirement for the non-subgridded model refined to the level of sub-grids is prohibitively increased. It is evident, therefore, sub-gridding allows for a more detailed simulation which would not be feasible with constraints in available computing resources. Near steady-state conditions were achieved after approximately 40,000 days of simulated time. The subsurface heads at this near steady-state condition compare favorably with the non-sub-gridded case. This is to be somewhat expected because the sub-gridded areas are focused in the surface-water layers, and sub-surface effects may not be as discernible on the regional scale presented. Simulated surface water depths at a near steady-state condition shown in Figure 4 illustrate water accumulation along the water courses specified in the domain. The sub-gridded model achieves higher resolution in critical regions near San Joaquin River and tributaries.

Since the response of an integrated surface and subsurface system is relatively rapid in only a small portion of the domain such as within the overland flow regime or the rivers, the sub-time stepping approach can significantly enhance simulation efficiency. In order to demonstrate the enhancement in efficiency, the response of the SJV to a single precipitation event was simulated using standard time stepping and sub-time stepping approaches and the results and the cost of simulations will be compared.

Prior to the application of the rainfall, the results from the steady-state integrated simulation were taken as the initial condition. The rainfall event was applied at and the simulation proceeded for an additional 8 days after the precipitation ceased (total simulation period of 10 days).
Two standard time-stepping simulations were performed using time step sizes of 0.1 and 0.5 days. For the sub-time stepping simulations, two cases were considered: one in which the entire surface was sub-timed and another in which only the river nodes and their neighboring nodes were sub-timed. The accuracy of the simulations is strongly influenced by the time step size when using a standard time stepping approach. The sub-time stepping simulations yield the same peak flow rate and peak arrival time, while the peak flow is underestimated by more than 10 percent for the case of a large standard time stepping of 0.5 days.

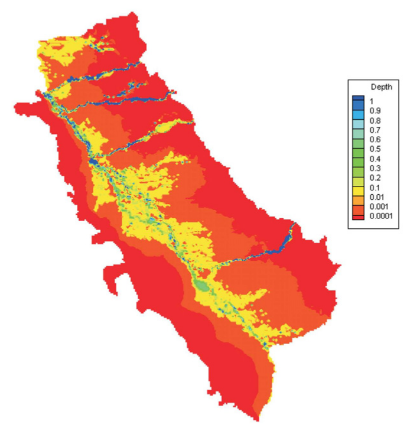

Figure 4. Water accumulation along the water courses specified in the domain.

The CPU costs for the standard time stepping simulations were 1,976 and 680 seconds for $\Delta t = 0.1$ and 0.5 days, respectively, while the number of time steps increases by a factor of five. When $\Delta t = 0.1$ days, during each time step, the matrix solver converges with fewer iterations, resulting in less than five times the CPU cost. For the sub-time stepping simulations, the CPU time was 1,517 seconds when the entire surface was sub-timed, while it was 1,106 seconds when about 20 percent of the active surface nodes were sub-timed. Thus, to achieve a similar level of accuracy using standard time stepping compared to the most efficient sub-timing case, almost twice as much CPU time is required.

CONCLUSIONS
Integrated surface-subsurface flow and transport numerical models typically can be fraught with temporal over-discretization problems because the characteristic response times for stream, overland and subsurface flow can differ by orders of magnitude. Moreover, the local and basin-wide scale aspects of integrated problems present spatial discretization challenges. The sub-time stepping approach can apply different time step sizes to the sub-domains that have different response times. It is most suitable for problems where the hydraulic activity is high in only a small portion of the domain such within or near active stream channels. The sub-gridding approach permits high resolution detailing in localized sub-basins of the model without the computational burden of extending the refinement to the flow-domain boundaries, whether vertical or horizontal. Sub-gridding is most suitable when the regions to be sub-gridded are localized portions of the overall domain.

Sub-gridding and sub-timing methods incorporated into HGS were tested using three flow domains: two local-scale and one basin-wide scale. The local-scale domains were adopted from the Smith-Wolhiser (1971) and Abdul (1985) study cases. The basin-wide flow domain was based on a simplified integrated surface-subsurface model of the SJV and the northern portion of the Tulare Basin. A conceptual model was developed for the SJV and Tulare Basin.

It was shown that the sub-timing approach can significantly improve the solution efficiency for solving coupled surface-subsurface problems. Additional efficiency could be achieved by dynamically adjusting the sub-timed nodes and the number of sub-time steps over the simulation period for a given problem. With respect to sub-gridding, it was shown that the method can achieve localized high resolution results at a computational savings, compared to study cases of more extensive mesh refinement. For some model applications, sub-gridding may permit simulation of a preferred level of detail in localized regions of a basin-wide model that may have otherwise been computationally prohibitive. Sub-gridding comes at a higher computational cost than regular mesh refinement, and thus there are diminishing returns on sub-gridding when the region to be sub-gridded represents a larger fraction of the flow domain.

Sub-gridding has been demonstrated to provide higher resolution accuracy in critical near-surface areas, such as surface water flow features. This accuracy comes at a higher computational cost, but is significantly less than the computational expense of discretizing all layers or the entire domain. Sub-gridding requires additional efforts in model set-up, parameterization and simulation. Particular care is required to obtain the optimal level of DEM resolution in both sub-gridded and non-sub-gridded regions. At the time of this research the tools to import DEM over the sub-gridded region only were in development. Furthermore, sub-gridding also permits grid coarsening where high resolution is not needed. Additional potentially significant saving in computational effort may be realized by coarsening of the grid in areas where high resolution is not required. In the SJV-TB model, potential areas for grid coarsening include flat-lying valley regions distant from rivers or other surface water features.

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Can man-made aquifers provide a solution for a thirsty mining industry?

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Abstract

The paper presents results of a case study whereby an existing backfilled mining void is used as an artificial aquifer to enhance water security, improve water quality and reduce water purification costs. Eland Platinum Mine is located in the northern central parts of South Africa, in close proximity of the capital, Pretoria. South Africa counts among the 40 driest counties in the word, has quite extreme weather conditions, and therefore needs to cater for both extreme drought and flood conditions. As a result the Mine, as part of an integrated water management plan, developed an artificial aquifer using an existing backfilled mining void, which is able to store up to 1 million cubic meters of raw water and also has the ability to capture large flood events and therefore buffering the Mine from extreme floods.

The project was initiated using limited field and laboratory results and continues to be carefully monitored and managed. Raw water is pumped into a semi-filled portion of an old mining void, water seeps into the backfilled portion and five ODEX boreholes are used to supply an on-site water treatment plant with water. The system started out in December 2007 and now supplies the Mine with all water requirements. Major advantages are cleaner water which requires no physical and biophysical filtering, thereby reducing electricity costs and lessening our carbon footprint. Furthermore, it reduces any risks of the Mine to lose production as a result of water shortages. Major challenges, e.g. escalating salts, were overcome with simple operating rules. Two years of results conclusively shows that mining voids within the platinum limbs of South Africa presents significant man-made aquifers and if properly managed it can become a major water source. This could ease the pressure on a diminishing natural resource.

Keywords: sustainable, integrated water resource management, artificial recharge, man-made aquifers, South Africa

INTRODUCTION

Eland Platinum Mine is situated approximately 10 km east of the town of Brits in the North West Province, South Africa and is within the western portion of the platinum-rich Bushveld Igneous Complex (BIC). The area is covered with mining developments and stretch over a distance of 80 km. To date, most new mining developments received their raw water from existing water resources, and as a result water security was becoming one of the major concerns for sustainable mining developments. Climate variability also contributed to water insecurity and called for new innovative water resource development and management strategies.

The Department of Water Affairs (DWA) developed the National Water Conservation and Demand Management Strategy (NWCDMS), which defines water conservation as “the minimization of loss or waste, care and protection of water resources and the efficient and effective use of water.” Eland Platinum is committed to the NWCDMS and aimed to change the way the mine manage its groundwater by bringing groundwater into mining as a sustainable partner, rather than as a risk to sustainability. Further to this, South Africa also developed the Artificial Recharge Strategy to introduce Artificial Recharge (AR) as a water management option and give some guidance on how AR can be applied in South African conditions (DWAF, 2007/1).

BACKGROUND TO INTEGRATED WATER MANAGEMENT AT ELAND PLATINUM MINE

An Integrated Water and Waste Management Plan (IWMP) is part of the South African water management legislation and when submitting a water uses license application technical information must be provided in the form of an IWMP (DWAF, 2007/2). The IWMP should provide details not only of the impact assessment, but also on the short and long-term management strategies, including details of a monitoring plan, measurement whether management objectives are achieved, and reporting. Measures to optimise and re-use water should be included in the document. Xstrata, Eland Platinum Mine, independently from IWMP, developed an IGRMP. The aim of the IWMP was to develop groundwater resources in advance of mining and optimize the use of groundwater resources (Botha, 2008). During the course of 2008 and 2009 the IGRMP was assimilated as part of the IWMP, making the it a continuous process to improve and optimise water resources at Eland Platinum Mine (Botha, 2008/2). As part of the IWMP, the Mine developed an hourly water balance simulation model to test different water management scenarios in terms of more effective management of mine water management (Simx Consulting, 2009). Different scenarios were tested, e.g. the storage of water underground, harvesting of open pit water or effective management of tailings storage return flows. The mode provides a scematic view of the Mine on a background of a Google image showing all the dirty and clean water flows and storage facilities (Figure 1).

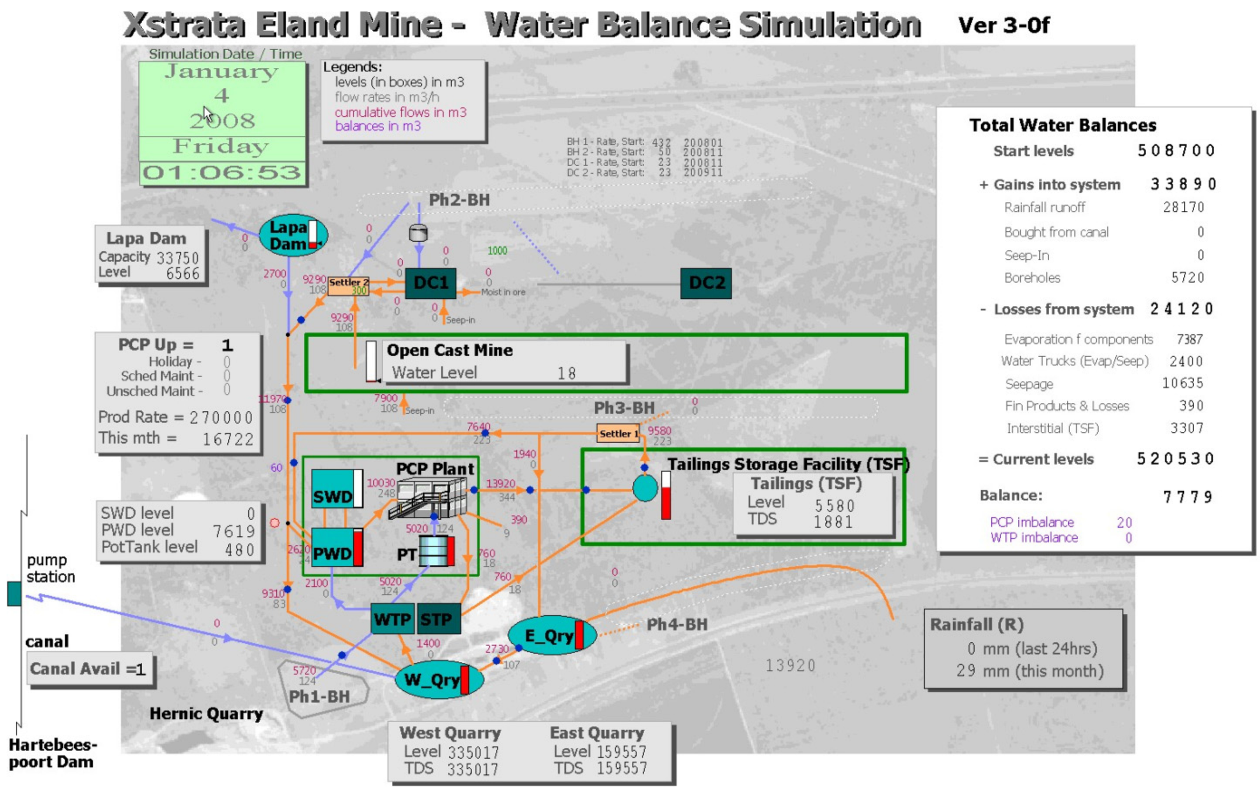

Figure 1 Screen shot of water simulation model used at Eland Platinum Mine.

Previous work

The area is underlain by mafic rocks of the Rustenburg Layered Suite (RLS) and forms part of the BIC. The RLS comprises of a basal Marginal Zone (norite), the Lower Zone (norite), the Critical Zone (pyroxenite, norite, anorthosite and chromititite), the Main Zone (gabbro-norite) and Upper Zone (magnetite-gabbro). The UG2 chromitite layers occur within the upper Critical Zone and are the primary mining target. In the Brits area the BIC intrudes into the Pretoria Group, and the Magaliesberg Formation forms the base of the BIC. Within the Brits area the strata strikes NE-SW, and dips towards the NW. At

Eland Platinum the mining reef dips at 18º and will be mined to a depth of ~1200 mbgl (Praxos 741, 2008).

The groundwater specialist investigation for the Environmental Impact Assessment (EIA) was conducted by Africa Geo-Environmental Services (Pty) Ltd (AGES) and findings from the specialist investigation were used as the point of departure (AGES, 2006). Improved hydrogeological understanding was gained by adapting mineral resource exploration data; as a result hydrogeological exploration was done with great success at the Mine (Praxos 741, 2008) The local hydrogeological conditions can be classified in three aquifer types upper perched, middle weathered and fractures and lower fractured (AGES, 2006).

The upper soil zone forms a rainfall dependent perched aquifer with a thickness between 1-5 m and blow yields less than 0.1 L/s and not really used. The middle aquifer can be classed as a semi-confined, shallow weathered aquifer with a thickness between 5-30 m. Blow yields are between 1-5 L/s and water quality is generally poor and high in nitrates. Fault zone fractured rock aquifers forms preferential flow pathways aquifers, having a variable spatial distribution or secondary fault zone aquifers.

Water quality is poor as a result of high nitrate concentrations in borehole water samples (>25 mg/L). The upper limit for domestic water supply for nitrate is 20 mg/L (DWAF, 1996). The average TDS concentration is 740 mg/L and the average EC value is 100 mg/L. The upper limit for domestic water supply is 2400 mg/L.

Based on the aquifer conditions, a conceptual model was derived and nine stages were simulated as scenarios to determine the groundwater flow and impacts (AGES, 2006). Simulated inflow rates into the open cast workings at the final mining depth (60 m) and calculated across the length of the open cast, were between 300 and 700 m^3/d and de-watering of the open cast mine for 5 years will lower the existing groundwater between 5 m and 15 m and might be evident up to 2 km from the open cast workings. Simulated inflow rates into the underground mine workings at a 1000 m were between 800 and 1000 m^3/d.

During the deep groundwater assessment (Praxos 741, 2008), ten boreholes were drilled to depths ranging from 150 m to 198 m. The highest blow yield recorded was 30 L/s with major water strike at 148 m. and has a potential long-term yield of 5 L/s. The combined potential long-term yield was estimated at 11.5 L/s for a 24-hour pump cycle.

Borehole water quality ranged between Class 0 and Class 3. High nitrate levels in top aquifers may have contaminated deep-seated borehole water quality and therefore some deep borehole may seem to have moderately high nitrate levels.

Continuous water level and temperature monitoring indicated definite differences between deep water strikes and shallow water strikes. Two permanent data loggers were installed in ELW 2 and 5 and are used to gather long-term time-series groundwater level and temperature fluctuations.

An additional exploration odex borehole (ELW 15) was drilled into the old Hernic quarry (OHQ) and was tested at ±25 L/s with a maximum recorded water level drawdown during the step tests of 0.05 m. The borehole can be used as an emergency abstraction point in the quarry.

INTRODUCTION OF MAN-MADE AQUIFERS

The Mine gets its raw water from the eastern channel of the Hartbees Irrigation Board (HIB) channel and stores it in the OHQ which serves as a water storage dam. The OHQ consists of an old open pit 40 m deep and partial filled to a depth of 28 mbgl and filled with water till roughly 21 mbgl. The quarry material consisted of waste rock form the open pit, basically anorthosite and norite. The OHQ is divided by a dolerite dyke to form a western and eastern portion and modeled as W_Qry and E_Qyr (Figure 1). During the groundwater exploration phase an exploration ODEX borehole (ELW 15) was drilled into a backfilled portion on the western side of the OHQ. Prior to the AR project, water was pumped from the OHQ via a floating barge fitted with four pumps to a water treatment plant where it was cleaned for both potable and process water.

The obvious advantages are that the natural rock filter lowers source water turbidity. Lower turbidity results in lower operational costs, for example longer periods between filter replacements, less chemicals required and less pressure on activated carbon cells. ELW 15 measured some12 m below barge pump intake and able to access water from the quarry for a longer period, for example in case of a major canal breakdown and much more storage is accessed by using deeper borehole water. Water in storage went up from an approximate 80 000 m^3 to 330 000m^3 and with variable availability of irrigation water the simulation model showed the Mine to run for 160 days without make-up water, whereas with only the barge it can run for some 39 days without make-up water.

Water quality and specifically nitrates were a bit of a concern; initial measurements for ELW 15 were measured at 18.2mg/L, similar to the regional groundwater measurements and close to the allowable domestic water limit of 20 mg/L. However, storage of water and evaporation in the OHQ creates a salt sink and over time the backfill portion would have become more saline. The boreholes provided the Mine with the flexibility to dewater the OHQ and dilute it with low TDS and low nitrate source water. Further to this, if the nitrate is too high within the Quarry then the cannel water can be used directly within the water treatment plant.

Development of additional boreholes

At the OHQ the topography dips slightly towards the west and water flows from east to west. The western portion of the OHQ is already rehabilitated and there was enough space available to develop additional boreholes. As a result the western portion of the OHQ was selected to develop a new well-field. An Astersat images where used to identify where the backfilled portion of the OHQ is situated (**Error! Reference source not found.**). Once the position was established, a series of multi-resistivity profiling were done to identify where the high-wall ends and where the deepest part of the OHQ was (Figure 2). Based on the results six sites were selected and drilled.

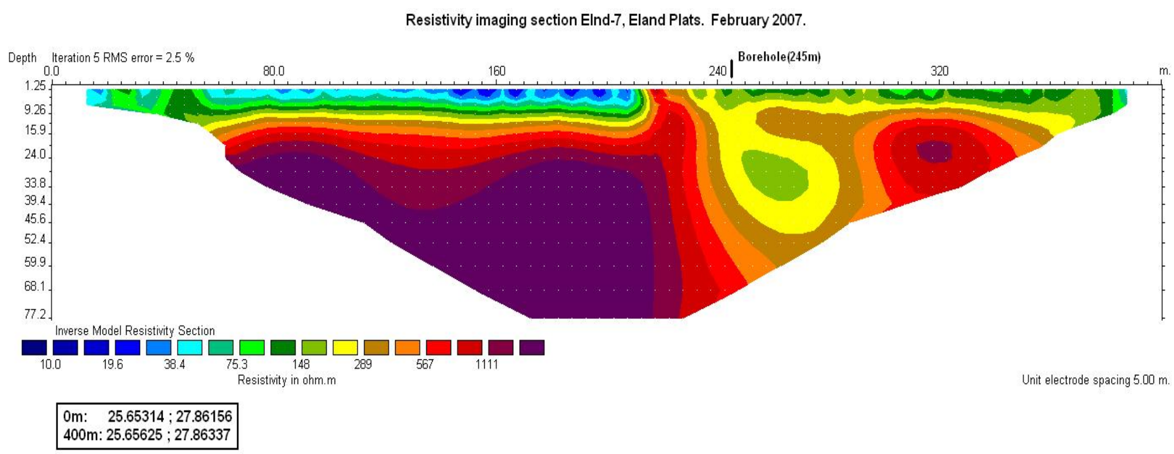

Figure 2 Geophysical profile 7 – positioning of high-wall ends and boreholes.

Boreholes were developed within the backfill material using an ODEX drill-and-drive method. Boreholes were drilled six meters into the quarry floor using the normal air-percussion drilling method. All ODEX sections have steel casings with inside diameters of 194 mm and the air percussion sections have inside diameters of 165 mm. ELW 16 and ELW 17 were drilled to confirm results from the geophysics and ELW 18 to ELW 21 were drilled as production boreholes with recommended yields between 19.5 and 28 L/s.

All boreholes were tested for macro elements. All boreholes were evaluated for Total Dissolved Solids (TDS), Electric Conductivity (EC), Magnesium (Mg) and Nitrogen (N) values and were classified as Class 3, except ELW 15 having much lower TDS values. This might be as a result of water intercepted by ELW 18 -21, being more stagnant, whereas ELW 15 is located 10 m from the OHQ and therefore much closer to the recently recharged surface water.

The boreholes where equipped with submersible pumps and connected via separate pipeline to the process water tank at the WTP. ELW 15 was connected the potable water line and supplies the potable water with all potable water.

One of the major concerns was water loses from the OHQ to the surrounding aquifers. This was constantly checked by measuring input and abstraction volumes and also comparing water level data of two monitoring boreholes within the in-situ aquifer next to OHQ. The boreholes are within 50 m from the quarry and both the boreholes show stable water levels some 6 m above the quarry water levels. Leakage into the aquifer is unlikely and it is more likely for the aquifer to decant into the quarry. Further to this, the volumetric measurements over a period of 15 months showed some 30 000 m^3 more water abstracted from the OHQ compared to water pumped into the quarry. Some of this water might be attributed to rainfall and runoff water, but a portion might be attributed to the aquifer slightly decanting into the OHQ. Personal communication with the previous owners suggested that the there was water in the aquifer and limited decanting took place from the aquifer.

On-site weekly data is taken to check water quality; critical to this is the nitrate values. The source-water has nitrate values of some 3 mg/L and the borehole water continues to have nitrate values of between 10 and 20 mg/L. The stagnant water in the quarry and the waste rock might be the source of salt load. To remove stagnant layered water and also to flash the system from excess salts the quarry was dewatered to levels below the backfilled area. During both dewatering attempts the EC and nitrate levels showed significant increase and correlates well with periods when no water was pumped in the quarry (Figure 3). Following the final attempt nitrate levels recovered to below 10 mg/L and EC values below 60 mS/m.

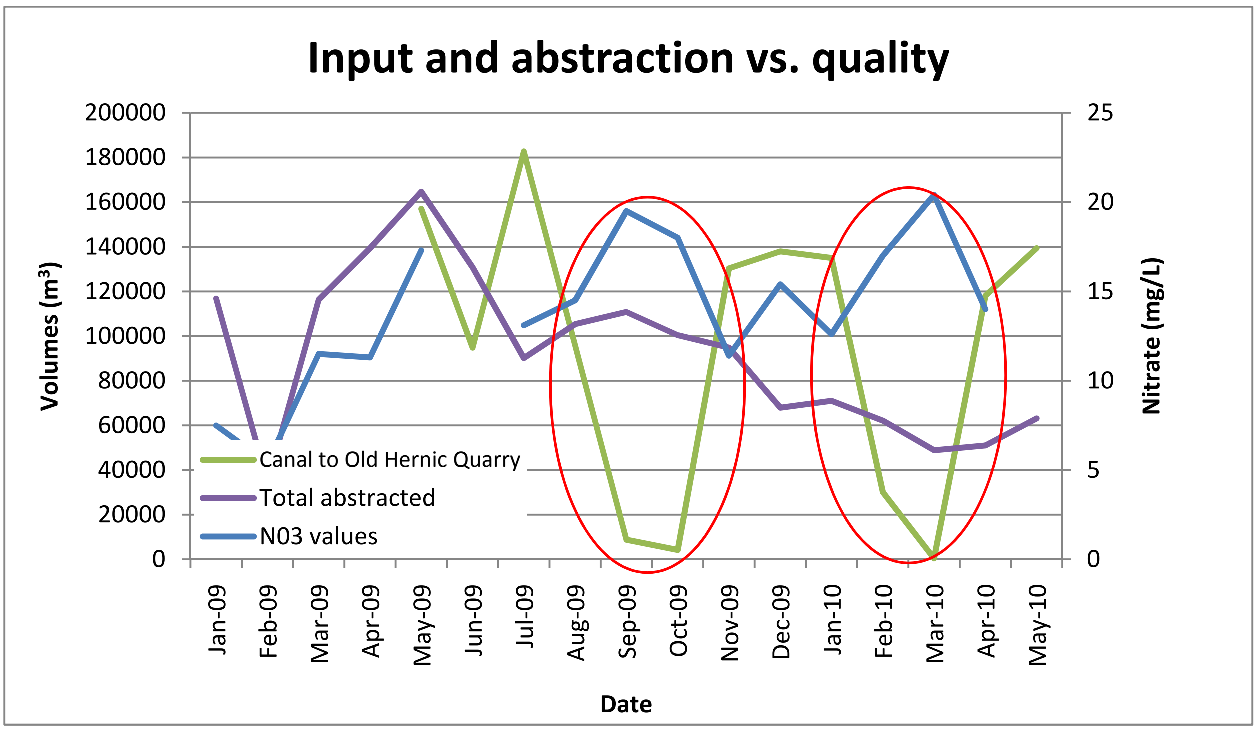

Figure 3 Periods with no input water associated with high nitrate values

CONCLUSIONS

The case study indicated that it is possible to store water within old mining works within the platinum mining areas in South Africa. If carefully managed, old mining works can become quite useful allies in assisting the mining industry to increase these assurance of supply, making more raw and potable water available. This OHQ is a small example compared to the many backfilled quarries available within the area and probably represents only 0.5 to 1% of the available backfilled quarries within the western platinum belt of South Africa. Roughly estimated, some 50 Mm^3 of water can currently be in storage and some backfilled pits continually leak water into underground workings, where is it mixed with dirty water and become and at huge costs pumped from depth and disposed as dirty water, containing hydrocarbons, high suspended solids and niters in excess of 100 mg/L. Even if current water in storage has elevated nitrates it can still be used as process water and gland service water and lift the heavy burden carried by potable water supplies.

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A EUROPEAN INITIATIVE TO DEFINE CURRENT RESEARCH NEEDS IN MANAGED AQUIFER RECHARGE

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Abstract The Water Supply and Sanitation Technology Platform (WssTP) was initiated by the European commission in 2004 and developed by the European Water Industry, open to all stakeholders. The objective is to stimulate a collaborative, innovative, visionary and integrated research and technology development strategy for the European water sector. Within different pilot programmes of the WssTP Managed Aquifer Recharge (MAR) was identified as a topic of interest and area relevant for further research. For this reason a Task Force on MAR was initiated in 2009 with 36 representatives from European research institutes, industry partners and with participation of international experts. During a workshop conducted in Graz in June 2009 these experts developed the basis for a report that has now been submitted to the European Commission for consideration in future research calls. In this report MAR was identified as a possible countermeasure against degradation of groundwater resources in Europe, that has a history of more than 150 years of practical implementation in Europe. Although not generating "new" water resources, it enables the use of alternative resources that would not be used otherwise (e.g. storm-water, seasonal high water flow, re-cycled water) for drinking water and irrigation by buffering high variations in availability and demand. MAR also provides an additional purification step in the regional water cycle. Recharged water can also act as an hydraulic barrier to prevent saltwater intrusion or the spreading of contaminated groundwater and inhibit a regional decrease of groundwater tables. This is particularly important in the scope of achieving the goals of the EU water framework directive. Research needs were identified in the field of defining "Best Management Practices" and standards for MAR in Europe, modelling for transparent feasibility assessment and the investigation of MAR in karstic aquifers.

Keywords: Europe, groundwater management, managed aquifer recharge, research needs, state of the art

INTRODUCTION

WSSTP Objectives and Structures

The Water Supply and Sanitation Technology Platform (WssTP, http://www.wsstp.eu) was developed by the European Water Industry and is open to all stakeholders. The objectives are to:

- enhance the competitiveness of the European Water Industry;
- contribute to the sustainability of Europe;
- address the challenges of water management in Europe;
- support initiatives relevant to the Millennium Development Goals.

The participants in the platform have produced a common vision document for 2030 concerning the whole European Water Industry as well as a Strategic Research Agenda (SRA) and an Implementation Plan (all three documents are available for download at WSSTP website). The SRA identifies research topics, which have been grouped into six pilot programmes:

1. Mitigation of water stress in coastal zones;
2. Sustainable water management inside and around large urban areas;
3. Sustainable water management for agriculture;
4. Sustainable water management for industry;
5. Reclamation of degraded water zones (surface and groundwater);
6. Proactive and corrective management of extreme hydro-climatic events;

WSSTP Task Force Managed Aquifer Recharge (MAR)

During the Vienna Workshop of the WSSTP (2008) the topic "managed aquifer recharge" (MAR) was identified as a cross-cutting theme covering issues in different pilot programs (urban areas, coastal zones, agriculture, degraded areas). In order to develop a common view of the current state-of-the-art and research needs, a task force on MAR was initiated that should involve major research institutions in this field and representatives of the different pilot programs (figure 1).

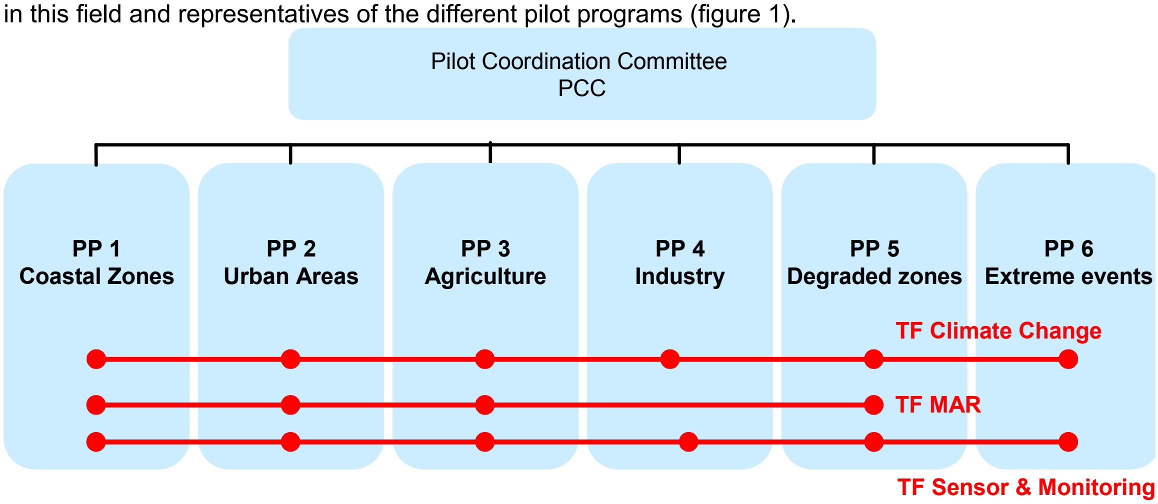

Figure 1: Task force MAR as a cross-cutting topic involving the WSSTP pilot programmes (modified from WssTP 2010).

Members of the Task Force were recruited from interested companies and institutions organized within the WssTP. In addition, renown researchers in the field of MAR, especially those that had participated in previous EU projects on MAR were asked to join. A Task Force workshop was held in Graz (29-30 June 2009, organised by the Berlin Centre of Competence for Water (KWB) and Waterpool, Graz). During this workshop the task force members agreed that further implementation of MAR offers great opportunities to face future challenges by urbanization, industrialization and climate change. However, research on MAR is necessary to enhance the implementation of these systems on a large scale in compliance with the regulatory framework and considering recent technological developments. The task force agreed to contribute to the WSSTP report on “state-of-the-art & research needs”, to support the update of the WSSTP strategic research agenda (SRA) and to develop a common vision of research needs to be proposed to the EU as recommendation for the work programme 2012 topic “Environment (including Climate Change)”.

STATE OF THE ART OF MANAGED AQUIFER RECHARGE (MAR) IN EUROPE AND ABROAD

Managed Aquifer Recharge (MAR) comprises a wide variety of systems in which water is intentionally introduced into an aquifer. The objective is i) to store excess water for times of less water availability (especially in arid and semiarid regions), ii) to introduce an (additional) barrier for purification of water for a specific use or iii) to reduce the risk of intrusion of impaired water (e.g. in coastal aquifers). In the frame of alternative water resources MAR can be seen as a technique which could enable the efficient use of water resources that are currently not captured (e.g. storm water, high flows from springs or rivers, flood waters, treated sewage effluent).

History and Types of MAR

Common MAR techniques used in Europe are: i) riverbank filtration (RBF), ii) artificial groundwater recharge – usually via pond infiltration (AR) and iii) aquifer storage and recovery (ASR) via injection wells.

Riverbank filtration (RBF) has a long history as a process for generating safe water for human consumption in Europe. During industrialization in the 19th century drinking water facilities in England, the Netherlands and Germany started using bank filtered water due to the increasing pollution of the rivers. The systematic production of bank filtrate started around 1870-1890 (BMI 1985). Since then RBF and, in case of insufficient quantity, artificial groundwater recharge (AR) have been generally applied as a first barrier within the drinking water treatment chain. Advanced post-treatment of the bank filtrate became necessary after the 1960’s as the quality of the water in the catchment areas further decreased.

Today the water supply of many European cities and densely populated areas relies on riverbank filtration (Schlosser, 1991). In France the proportion of bank-filtered water was estimated to reach approximately 50% of the total drinking water production (Castany, 1985: from Doussan et al., 1997). In the Netherlands 13% of drinking water is produced from infiltration of surface water, such as bank filtration and dune infiltration (Hiemstra et al., 2003). In Germany riverbank filtration and artificial groundwater recharge are used in the valleys of the rivers Rhein, Main, Elbe, Neckar, Ruhr, and in Berlin along the Havel and Spree (Grischek et al., 2002). In Berlin 60% of the drinking water is derived from riverbank filtration and artificially recharged groundwater (Zippel & Hannappel). In the US riverbank filtration is applied as an efficient and low cost water pre-treatment technology (Ray et al., 2002), mainly to improve the removal of surface water contaminating protozoa.

Infiltration ponds are surface spreading basins providing added benefits of treatment in the vadoze zone and subsequently in the aquifer. Surface spreading is the simplest and most widely applied artificial recharge method (Asano et al., 2007). Infiltration trenches are also used to infiltrate water from a different source along a river, if the river is too polluted and river bank filtration has to be avoided (performed since about 30 years for the protection of Graz water supply in Mur river basin).

Injection wells are often used for aquifer recharge in case of deep aquifers, insufficient area available or for sanitary aspects. For recovery the water is then either pumped from the same well (aquifer storage and recovery – ASR) or from adjacent wells subsequent to a defined transport time (aquifer storage, transport and recovery – ASTR). These techniques are commonly applied in Australia and the USA, but also in Europe (Netherlands, Spain). First ASR systems were operated for means of alleviating the effects of groundwater abstraction or saltwater intrusion. In the USA the first ASR system came into operation in 1968 (Wildwood, NJ); until 2002 a total number of 56 had been operational with more than 100 additional systems in planning including the Comprehensive Everglades Restoration Program in Florida (CERP) and a large ASR program for New York City (Pyne, 2005).

The common way of capturing infiltrated water is via wells in the influenced aquifer (physical or bio-chemical clogging is a frequent issue here, met by a wide variety of regeneration methods). In arid or semi-arid regions, however, MAR may be utilized to support the discharge from groundwater well fields and springs that may not supply sufficient water throughout the year.

MAR systems are also classified with respect to the origin of the infiltrated water and the subsequent use of the recovered water:

- traditional BF and AR systems for drinking water production using surface water from rivers (e.g. Berlin, Germany) or springs (e.g. Graz, Austria),
- infiltrated surplus water (e.g. from drainages or storm-water) is often used for irrigation (e.g. Vergel aquifer, Spain),
- during soil aquifer treatment (SAT) wastewater of varying quality experiences additional purification during flow through unsaturated soils and the aquifer. Artificial recharge of secondary effluent to obtain water for unrestricted irrigation is mostly used in arid and semi-arid regions in USA, Israel, Australia and other countries around the world (AQUAREC Project, 2003-2006). On the other hand, infiltrated waste water treatment plant (WWTP) effluent can serve as alternative water source for industrial use,
- water from desalination plants is increasingly being used for ASR in arid countries,
- infiltration of effluent industry water (process water or cooling water) for storage and reuse with the aim to achieve a certain quality and temperature (hot or cold).
- increasingly, ASTR-like systems are used for geothermal heating/cooling and for aquifer thermal energy storage (ATES, Pyne 2005)

Buffering water quantity and quality

In most applications, MAR acts as a buffer in terms of water availability (quantity) and water quality. The key parameters that determine the extent the system can act as a storage are the specific hydrogeology of the aquifer (e.g. hydraulic conductivity) and the clogging at the entry point of the recharge water (infiltration pond or well). Clogging occurs due to physical, chemical or biochemical processes and needs to be regarded carefully as it may reduce the systems performance substantially. In practice infiltration ponds can be cleaned regularly by scraping off the uppermost sand layer or by wet-dry-cycling whereas the rehabilitation of clogged injection wells requires more effort. For this reason pre-treatment of the injected water is usually necessary and periodic backwashing is used to maintain the conductivity. On the other hand, re-contamination by micro-organisms, which is an issue in surface spreading basins, is avoided. In other cases infiltration water originates from karst springs (Zojer 2008; Trcek & Zojer 2010) at times of high flow, taking into account that their water quality generally fulfils the requirements for subsurface passage without pre-treatment.

Water quality aspects of MAR are governed by i) the quality of the infiltrated / injected water ii) physical straining of particulate and particle-bound substances, iii) biogeochemical degradation / deactivation processes within the aquifer, iv) aquifer mineralogy and water-rock interactions and v) the quality of the ambient groundwater. The most encountered process for RBF applications is usually the physical straining of particulate and particle-bound substances, lessening the effort for subsequent drinking water treatment. In Berlin, e.g. disinfection of drinking water can usually be avoided due to complete removal of pathogens during underground passage of up to 6 months. Organic substances and heavy metals can be adsorbed to the aquifer matrix. Although this does not remove the substances completely, peak loads – e.g. from oil spills – are retarded and maximum concentrations reduced. Dissolved organic carbon and organic trace substances are biologically degraded in the subsurface to varying extent. Recent investigations have shown that the redox potential in the aquifer is decisive for the degree of elimination (Stuyfzand, 1998 , Massmann et al. 2007). On the other hand, redox sensitive trace elements may be released from the aquifer matrix or sorbed inorganic pollutants may become remobilised when redox conditions change (Oren et al., 2007).
The level of knowledge of natural treatment systems, notably in aquifers, is not as strong as in engineered systems, because the biogeochemical environment present in any aquifer where water quality is modified will vary spatially and usually also temporally within the aquifer (Dillon et al. 2008). The heterogeneity of the system, which strengthens its buffer potential on the one hand, makes it more difficult to describe and control on the other hand.

Hydraulic barrier effect

In coastal aquifers saltwater intrusion and mixing processes decrease the reserve of fresh water resources. The resulting brackish water is not usable for drinking purposes and in the most cases even not for irrigation. These processes are not limited to the coastal belt, they occur also in the inland part of coastal aquifers or in regions with saline groundwater originating from deep salt deposits.
According to the seasonal changes of salinity, coastal aquifers can be only utilized in time periods when the static pressure of the fresh water component is high and the fresh/sea water interface in the aquifer is forced very much towards the sea. Overexploitations result in an increasing migration of sea water especially during the dry period.
Recharge of freshwater from different sources through well galleries or infiltration ponds along the shoreline will create a piezometric high (hydraulic barrier) protecting wells further inland from saline intrusion. The same MAR techniques have been used to protect drinking water wells in alluvial aquifers from accidental short-term pollution in the rivers hydraulically connected to the aquifer (e.g. the Crépieux-Charmy site providing Lyon (F) with drinking water).

PERSPECTIVES FOR MAR IN THE EUROPEAN UNION

Current trends and predicted future developments in freshwater consumption and environmental legislation

Developments in water demand. According to figures dated 2004 (UNEP 2004) the 42 % of the total water abstracted in the EU is used for irrigation – in southwestern European countries and EECCA (Eastern Europe, Caucasus and Central Asia) it accounts to 50 to 70 % of the total water abstraction. In central and northern Europe, industry is the main abstracting sector with most of the water used for electricity production (e.g. 64 % in France and Germany). Although the per capita consumption in household and the abstraction for industrial purposes has been decreasing since 1980 there is a clear upward trend in irrigable land in the EU member states (UNEP 2004). Between 1961 and 1996 the irrigable surfaces have risen by > 5 Mio ha across the 15 EU member states. Although water use efficiency measures have been proposed (e.g. planting crops with lower water requirements) it remains uncertain if these measures can compensate the ongoing increase in irrigated land. A second important factor to consider is tourism along the coasts of the Mediterranean Sea, which doubles the coastal population during summer and leads to high pressure on water resources in these regions at times during which water resources are scarce.

Developments in Water Availability. In the decades to come the EU will face drastic water related issues due to environmental changes (i.e. global warming). Already in the past years, the topic of water scarcity and drought has grown up to the top of the European agenda: in February 2008 the EU Report ‘Water Scarcity and Drought’ stated that water stress affects 130 millions inhabitants (30% of population in Europe) in Southern Europe but also in Northern countries such as Belgium, Denmark, Germany, Hungary and UK (EEA 2009). The report concluded that in the past 30 years, water scarcity and drought had a cost of €100 billion to the European economy (€8.7 billion only for the drought of 2003). Without countermeasures this is likely to increase in the future.

Implementation of the EU Water Framework Directive (WFD). In 2000, the European Commission endorsed the Water Framework Directive (2000/60/EC). The WFD commits European Union member states to achieve good qualitative and quantitative status of all water bodies (including near coastal waters) by 2015. It is a framework in the sense that it prescribes steps to reach the common goal based on integrated risk assessment and management rather than adopting the more traditional limit value approach The WFD calls for integrated water resource management at the scale of river basins. The overall goal of the WFD represents a shift from a paradigm focused on the exclusive uses of water. The goal is to ensure that the water demands of natural systems are environmentally balanced with the agricultural, industrial and domestic needs of societies. In particular, the WFD requires “the promotion of sustainable water use based on a long term protection of available water resources”, controlling the negative environmental impacts that water users can have upon the water cycle. At different steps in the cycle, water will be considered as a valuable finite natural resource while wastewater can be considered as a source of beneficial compounds.

Managed Aquifer Recharge as countermeasure

MAR is a possible countermeasure against water scarcity and aquifer over-exploitation, that is explicitly mentioned in the WFD and is already now implemented world wide. Although not generating “new” water resources, it enables the use of alternative resources that would not be used otherwise (e.g. storm-water, re-cycled water) for drinking water and irrigation by buffering high variations in availability and demand and by acting as an additional purification step. Recharged water can also act as an hydraulic barrier to prevent saltwater intrusion or the spreading of contaminated groundwater and inhibit a regional decrease of groundwater tables. Thus MAR should be regarded as a management tool to ensure water supplies under changing climatic and geographical regimes AND as a means to achieve the goals of the EU directives - by interfering with and controlling the water cycle on basin scale, and thereby mitigating economic losses and facilitating increased economic productivity.

CHALLENGES AND RESEARCH NEEDS

Legislation. Although artificial recharge of groundwater is mentioned as possible supplementary measure in the WFD to meet groundwater over-abstraction, “controls, including a requirement for prior authorisation of artificial recharge or augmentation of groundwater bodies” are mandatory. A basic requirement is that no deterioration of water status may take place. The member states have adopted different approaches towards authorizing artificial recharge, especially within re-use schemes. Whereas in Spain recharge with treated wastewater is regulated within a legal regulation for the reuse of treated wastewater (Hochstrat et al. 2008) authorization of these systems in Italy is usually not possible. One outcome of the EU project RECLAIM WATER (Kazner et al. 2010) was therefore that the existing European water directives provide only little guidance for the authorization of aquifer recharge schemes and the recommendation to develop a supportive policy frame (Hochstrat et al. 2008).

Health related risks. Especially for MAR systems within re-use schemes health related issues need to be addressed (Wintgens et al. 2008, Sharma et al. 2007, 2008). This relates to pathogens as well as trace organic substances (such as endocrine disruptor compounds and antibiotics). Previous investigations have shown that there is a high potential for these compounds to be removed during subsurface passage (Böckelmann et al. 2009; Dillon et al. 2008; La Mantia et al. 2008), however, repeated cases of failure make the definition of “best management practices” indispensable (Page et al. 2010; Toze et al. 2010). Micropollutants are also not completely removed and may show up in low trace concentrations being only partly degraded (Schulz et al. 2008).

Sustainability of MAR. A key challenge is to demonstrate that the impact of MAR on the subsurface is sustainable or reversible especially with respect to future policies on MAR including regulations on the quality of water acceptable for recharge. In addition the application of MAR has to be considered in relation to other subsurface uses (such as cold/heat storage, industrial water uses etc.) and factors (soil and groundwater contamination, salinisation). MAR could be tied with efforts to improve recharge in cities to e.g. reduce flood risk (Pilot Extreme Events) or elevate groundwater levels in overexploited aquifers (Pilot Urban). Within the Pilot Coastal the main focus is directed towards avoiding salt water intrusion. In addition, MAR can also act as a balancing tool between periods of high precipitation and seasons of high water demand (for irrigation or drinking water supply) as in some Mediterranean countries (e.g. Montenegro, Spain, France, Italy, Greece…).

Technical and financial risks. Due to the heterogeneity of the subsurface and its limited accessibility the performance of MAR systems is difficult to predict and possibilities for control during operation are few. As for example in case of river bank filtration, the investment costs for water abstraction are substantially higher than for using surface water directly (Rustler et al. 2010) the benefits (higher water quality and supply security) need to be quantified and certain limiting conditions defined. This could prevent unnecessary investments and yield higher sustainability of the systems.

CONCLUSIONS

A wider uptake of the economic benefits of MAR in Europe is currently being impeded by lack of certainty among regulators and the wider community concerning (1) quantification of the actual benefits and costs of MAR projects; (2) potential environmental impacts of MAR projects, (3) policies to ensure their integrated contribution into catchment and basin water resources management plans, taking account of climate change. These issues need to be addressed through appropriate research, including assimilation of past projects, and establishing good communications and dissemination activities.

Research is needed to support formation of effective scientifically based regulation of MAR in order to achieve its benefits while ensuring protection of groundwater and environmental flows. An EU-wide research project with the aim to facilitate the implementation of MAR integrating the needs of the different WSSTP pilot programs should therefore address both issues related to technical and managerial evaluation and optimization of the single plants and related to the use of MAR in a wider IWRM context.

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Managed aquifer recharge in Austria as central element of integrated water resources management

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Abstract
During the last 5 years the artificial groundwater recharge system of the municipal works Graz, Austria, which is in operation for almost 30 years, has been thoroughly examined. About 180 l/s of high quality water from pristine creeks (i.e. no pre-treatment necessary) are infiltrated at two sites via sand and lawn basins and infiltration trenches to allow for the extraction of approximately 400 l/s for drinking water supply. The investigations included (i) field experiments and laboratory analyses to improve the trade off between infiltration rate and elimination capacities of the sand filter basins' top layer, (ii) numerical groundwater modelling to compute the recovery rate of the recharged water, the composition of the origin of the pumped water, the transient capture zones of the withdrawal wells and the optimization of recharge and withdrawal coordination and (iii) development of an online monitoring setup to guarantee reliable functioning of the entire structure. In addition, the depreciation, maintenance and operation costs of the managed aquifer recharge system have been evaluated. It turned out that next to personal and housing infrastructure expenditures the maintenance costs of the water intakes at the creeks and of the preservation measures within the protection areas around the production wells are the most relevant cost items. Overall, total costs of artificial groundwater recharge amount to 0.20 €/m³ excluding property, pumping and distribution costs compared to a water price of about 1.5 €/m³ charged to consumers. Thus, also in predominantly humid regions managed aquifer recharge represents a viable and sustainable solution to overcome local shortages of drinking water supply which is expected to increase due to projected climate change impacts.

Keywords

costs of MAR, infiltration experiments, managed aquifer recharge, numerical groundwater modelling

INTRODUCTION

Groundwater resources in almost all countries are subject to increasing strains related to various land uses (e.g. agriculture, housing and infrastructure projects, mineral resources mining, regulation of rivers). Moreover, changing climate conditions, i.e. altered temperature and precipitation patterns, will add to the modification of groundwater dynamics and availability. Based on this circumstances it is a vital task for water supplier to secure (ground-)water resources to meet drinking water demand both in quantitative and qualitative manner. Managed aquifer recharge can play an important role in that respect.

The Graz AG, which includes the municipal water works, supplies about 50,000 m³ of drinking water per day to the second largest city in Austria (location shown in Figure 1). To sustain the groundwater extraction two managed infiltration sites are being operated: Friesach, about 20 km north of Graz and the Andritz site in Graz. About two thirds of the total water supply is provided by the two recharged aquifers, the remaining part is covered by a remote water line from the eastern alpine limestone region. About half of the pumped groundwater (16,350 m³/day) originates from managed infiltration into the aquifer. The difference is covered by lateral subsurface inflow, groundwater from the ambient aquifer and infiltrating surface water from the Mur river. The artificial recharge volume is taken from two local creeks that show almost no anthropogenic impact. Thus, no pre-treatment of the raw water before infiltration is needed. The average annual precipitation in the area is about 900 mm.

Both recharge sites show the same arrangement of infiltration structures. Next to the raw water inlet a settlement tank consisting of two chambers has been installed. If the turbidity level exceeds 5 FNU, the inflow from the tapped creeks is stopped by means of an automatic closing gate. Then the water flows into horizontal gravel filters with a granular size of 8/16 mm to remove suspended solids and reduce organic biomass. After an operation period of 2 to 3 years, the filter material has to be replaced. Following the horizontal flow filter system passage, the water is distributed by several valves and cascades into the infiltration basins. The cascades serve to increase the oxygen content of the water. For infiltration lawn and sand basins as well as infiltration trenches are in use.

The five lawn basins vary in size between 1,180 and 2,000 m^2 and show an infiltration capacity of about 1-2 m/d. The lower stratum, with a thickness of 20 cm consists of 8/16 mm gravel granules. The top layer consists of humus and was originally planted with grass. Due to the natural concurrence situation the grass has been displaced by local moss. The lawn basin operates intermittently for a period of 10 days and an intermission of further 20 days. The operational halt after 10 days is necessary to avoid the grass and moss dying. Additionally, four sand filter basins are used for recharging the groundwater (area between 930 and 3,000m^2). The uppermost 50 cm sand layer has a granular size between 0.06-2 mm. The average infiltration capacity is approximately the same than that of the lawn basins. Each of the sand basins is flooded for 4 days followed by a 3 days resting period. Especially during the warm summer months problems with the emergence of algae occur. The sand layer has therefore to be cleaned regularly.

Furthermore, 3 infiltration drains of 100 m each were installed which show a much higher infiltration rate than that of the sand and lawn basins. Regular maintenance work to ensure the infiltration capacity of the drains is necessary. Figure 1 shows an areal view of the Friesach site. The river is Mur (average discharge of 117 m^3/s) in the foreground. One lawn basin is in operation, the other two are in their resting period. The two sand basins are located between the river and the lawn basins. In the background one of the two horizontal pumping wells can be seen. The subsurface passage contributes to the high quality of the extracted water. The predominant groundwater flow direction in the Mur-valley aquifer is parallel to the Mur river with an average slope of about 0.5%. The groundwater level usually fluctuates between 4 – 6m below the ground surface. The general aquifer is built of quaternary sand and gravel fluvial sediments of locally strong varying conductivity and thickness. The groundwater level and quality is continuously monitored by means of a vast number of observation wells. To ensure the groundwater quality, several types of protection areas are in use.

In order to optimize the current recharge operation and to lay the basis for a possible extension of recharge structures a cooperation with the Graz AG was launched to gain in-depth understanding of (i) the elimination rate of different top layers of the recharge basins by means of a experimental study, (ii) the origin of the pumped water with the help of a numerical groundwater model and (iii) the costs of managed aquifer recharge.

METHODS

Infiltration experiments

To explore the trade-off between elimination and infiltration rate of the infiltrated water three concrete rings with a diameter of 2.5 m and a depth of 1.5 m each were built at the Andritz recharge site. The general filling structure of each ring is as follows (from bottom to top): 35 cm gravel 4/8 mm (drainage layer for discharge), 55 cm horizon of local gravel and 15 cm gravel underlaying (10 cm 8/32 mm, 5 cm 3/8) mm. The remaining 45 cm are packed with changing fillings depending on the experimental series. The layer thickness of the fillings relates to the experiences with pilot plants of Hütter et al (2006).

In the first ring the grain size distribution of the existing sand basin (grain sizes between 0.01 and 8mm) is used for reference and was not replaced during the experiments. In ring 2 a mixture between middle sand (0.2- 0.63 mm) and the local B-horizon is applied as the top layer. In later experimental series the grain size distribution is varied at the lower and upper end (0.4 – 8 mm) and also lawn tiles form the lawn basin in Andritz were utilized. The top 45 cm of ring 3 were filled with different mixtures of sandy gravel and humus soil and in the last experimental series planted with reed.

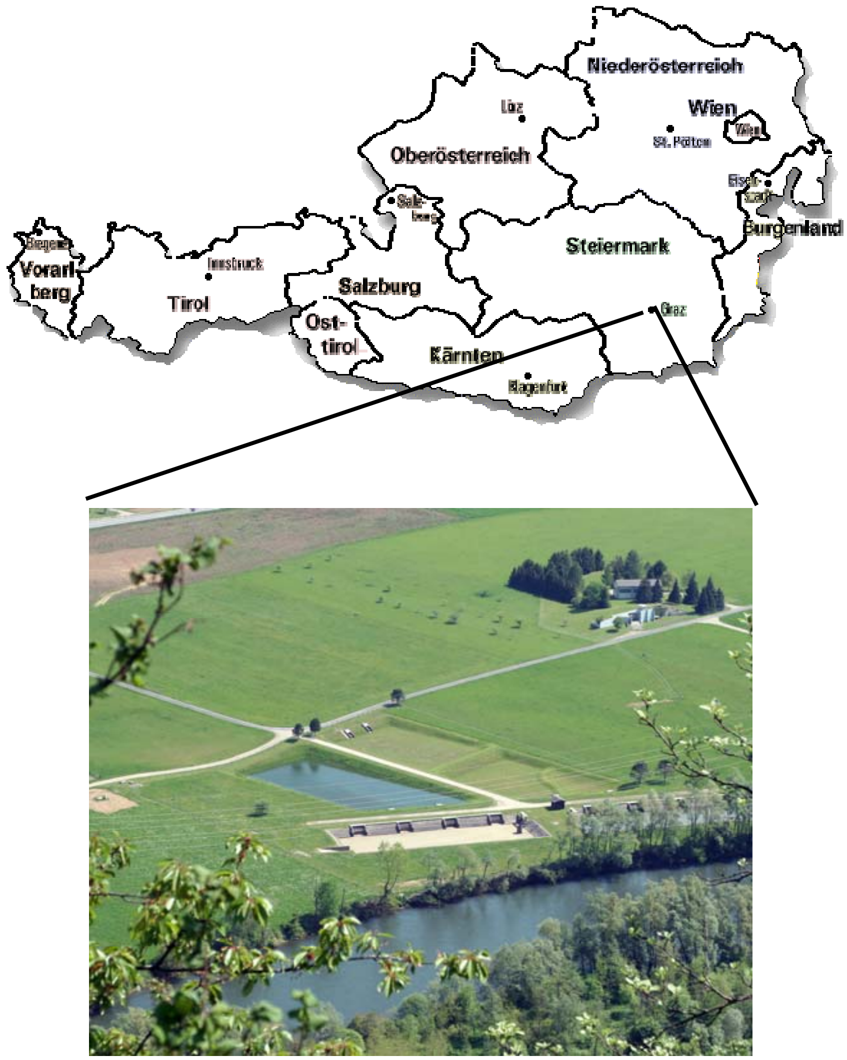

Figure 1: Location within Austria and areal view of the Friesach recharge site.

In order to consider the operational mode of the real sand basins, water is pumped to the 3 concrete rings for 4 days followed by a rest period of 3 days (only the series with reed in ring 3 was permanently supplied with water). The individual amount of water pumped to any ring is controlled by a ball valve so hat neither one falls dry or overflows. The entire unit is equipped with just one pump. Surface water is taken from the local creek as for the real recharge facility in Andritz. The inflow and outflow concentration of coliform germs, Escherichia coli and Enterococci as well as the water discharge at the three rings are measured. Standard ion and anion parameter were also observed but didn’t show any significant changes between inflow and outflow of the concrete rings. Chemical parameters like Ammonium, Nitrite or TOC are mainly filtered in the horizontal gravel filter system before reaching the recharge structures.

Numerical modelling

Managed aquifer recharge typically consists of the interplay between infiltrating allochtone water and subsequent withdrawal of groundwater. Thus it is important to understand the hydraulic impact of the added water onto the subsurface flow regime. The location of the infiltration structures and the resulting infiltration rate due to the operational mode (i.e. constant or intermittent, basin size, applied water volume) are the decisive variables in this respect. Based on the general hydrogeologic characteristics of the receiving aquifer the relative location (and the use) of the pumping wells determines the recovery rate of the infiltrated water.

A numerical groundwater model can be a valuable tool to optimize the operation of an artificial groundwater recharge site. In particular, the transient capture zones of pumping wells can be delineated which plays an important role in the protection of drinking water resources. Rock and Kupfersberger (2002) have implemented a particle tracking module that illustrates the significance of considering transient aquifer conditions such as artificial groundwater recharge. Within this method at regular intervals execution of the numerical groundwater flow routine is interrupted and particles, that are being started from a predefined starting grid and at predefined timesteps, are put forward using the current magnitude and direction of the groundwater flow velocity. Per particle, only the starting location, the actual position at the current time step and the total time elapsed since release of the particle needs to be stored. After the total simulation period is reached the relevant information about every particle is processed in relation to the respective starting location. The superposition of the starting locations yields the extent of the capture zone of the well.

For the artificial groundwater recharge site at Friesach a regional and transient horizontal groundwater flow model was set up and calibrated at about 30 monitoring wells (location shown in Figure 2) for the period between 2004 and 2006. Area-wide recharge was computed with a separate model for the unsaturated zone considering soil and land use information as well as daily meteorological data. The northern and southern model boundary was implemented as a Diriclet boundary condition based on groundwater level observations. The lateral inflow at the eastern model side was estimated derived from water balance calculations for the adjacent catchment areas. Interaction between groundwater and the river Mur was considered by the leakage concept. Individual daily time series for the 2 pumping wells, 2 sand basins, 3 lawn basins and 3 infiltration trenches were implemented corresponding to water meter readings. The average calibration error over the entire simulation period and all monitoring wells is about 16 cm, the resulting conductivity distribution varies between 1.3 and 9 $*10^{-3}$ m/s.

Based on this groundwater model transient capture zones of the 2 pumping wells are computed to find alternative operational modes of the recharge procedure. Furthermore, the associated recovery rate and the origin of the pumped water are determined.

Evaluation of infiltration costs

In assessing managed aquifer recharge as a valid option for sustainable water management the associate costs also need to be evaluated. Otherwise, social acceptance and economic implementation can not be achieved. For this purpose the maintenance and operation costs of the two Graz AG artificial recharge sites have been examined from 1996 on where the current electronic accounting system was installed. Maintenance costs of the following categories have been distinguished: infiltration basins, monitoring wells, pumping wells protection areas, intake structures at the local creeks and operational building. Additional significant cost positions include continuous supervision work and labour costs of the manual readings of the groundwater table and taking of water quality samples at the monitoring wells.

Furthermore, the construction costs of the main parts of the infiltration structures have been analyzed including the distribution pipe network, the horizontal gravel filter, the sand and lawn infiltration basins, the infiltration trenches and the water intakes at the creeks. In order to arrive at individual yearly depreciation costs specific life time periods between 30 and 50 years (lower bound relates to the sand basins at Friesach due to poor concrete quality and the upper bound to the water intake at creeks and the distribution pipe network) have been estimated. The actual construction costs have been adjusted for inflation with respect to 2008 as a reference.

Another approach to contemplate the costs of managed aquifer recharge is to quantify the (opportunity) costs that would be necessary to provide for the same amount (and quality) of water by different means.

These costs will highly depend on the local conditions, in particular on the sustainable availability of adequate water resources in a distance to consumers where conveyance costs are still reasonable. In regions of general low water availability managed aquifer recharge will create an added value if other sources of good quality water become available for drinking water supply that would have been used for different purposes (e.g. irrigation) otherwise. Moreover, a psychological component has to be taken into account if consumers are not used to drinking water sources other than no or only little pre-treated groundwater. In that case protest and refusal due to smell, taste or even cultural reasons may arise.

RESULTS AND DISCUSSION

Infiltration experiments

The results of the infiltration experiments (more details in Tischendorf et al., 2008) indicate that the existing sand basin (ring 1) and the sand mixture with the B-horizon (ring 2) have significant better retention rates than the sand mixture with humus (ring 3) for all three bacteriological characteristics. Retention rates are computed by relating the difference between inflow and outflow bacteria counts to the inflow value. However, the sand mixture with the B-horizon is much less permeable making it a less attractive top layer for a recharge facility (0.2 m/d compared to 1.3 m/d for ring 1). With exception of coliform germs ring 1 and ring 2 reveal similar retention capacities; however, the variation within the values of the reference sand mixture is considerable higher. In particular for Enterococci and Escherichia Coli the sand mixture with the B-horizon shows almost total retention. In contrast to the retention behaviour of rings 1 and 2 the values of the sand mixture with humus (ring 3) display strong erratic fluctuations for all three hygienic parameters. This may be due to the fact that the organic components of the added humus can only withhold fewer germs than expected and that also remobilization occurs.

The top filling in ring 1 remained unchanged within the second experimental series and even showed an increase in retention rates (all above 88%) for all bacteriological parameters. The variation of the grain size distribution in ring 2 and of the mixture with humus in ring 3 (less content of humus combined with smaller sand grains) in generally yielded improved results. The infiltration rate in ring 2 increased to the same level as in the reference ring; however, at the same time the retention rates of the bacteriological parameters decrease but are for Escherichia Coli and coliform germs still in the range of the reference ring. The modified mixture of humus and sand does not show the remobilization effect anymore, yet the retention rates for Escherichia Coli and Enterococci are still significantly lower compared to the existing sand mixture in ring 1.

In the third experimental series ring 1 confirmed the results of the previous sessions whereas for the use of lawn as top layer an increase of coliform germs at the outflow of ring 2 was observed. The retention rates for the other two bacteriological parameters were acceptable. Reed (in ring 3) showed promising results in terms of retention and infiltration rates, however, more long term and large scale experiences are needed to get to a final appraisal. Overall, the currently used sand (2.5% clay and silt, 61.5% sand, 36% gravel) represents a good compromise between further water purification and infiltration rate. Due to the raw water quality of the local creeks, which is not impaired by human activities, the initial incorporation of organic components in the filter layer doesn’t yield any advantages.

Numerical modelling

The left image in Figure 2 shows the resulting capture zones of the two pumping wells at the artificial groundwater recharge site Friesach in terms of travel times. The location and size of the infiltration basins is represented by the light blue shapes. It can be clearly seen how the capture zones develop around the two well locations (red areas with travel times less than 30 days). For longer travel times a common capture zone emerges that spreads the entire width of the model domain and further grows opposite to the main groundwater flow direction. The subsurface travel times between the different infiltration devices and the withdrawal wells range from 22 to 62 days.

Furthermore, conservative transport models were run to investigate the origin of the pumped groundwater and the recovery rate of the infiltrated water. Values for the longitudinal and transversal dispersion and a corresponding ratio were taken from the literature. The extracted water consists of 36% artificially recharged water, about 4% water from the river Mur (by bank filtration) and 60% ambient groundwater. The recovery rate of the infiltrated water is about 87%. Some of the loss is due to the close positioning of the sand basins alongside the river Mur which was originally intended as hydraulic barrier against the poor surface water quality (which has considerably improved in the meantime).

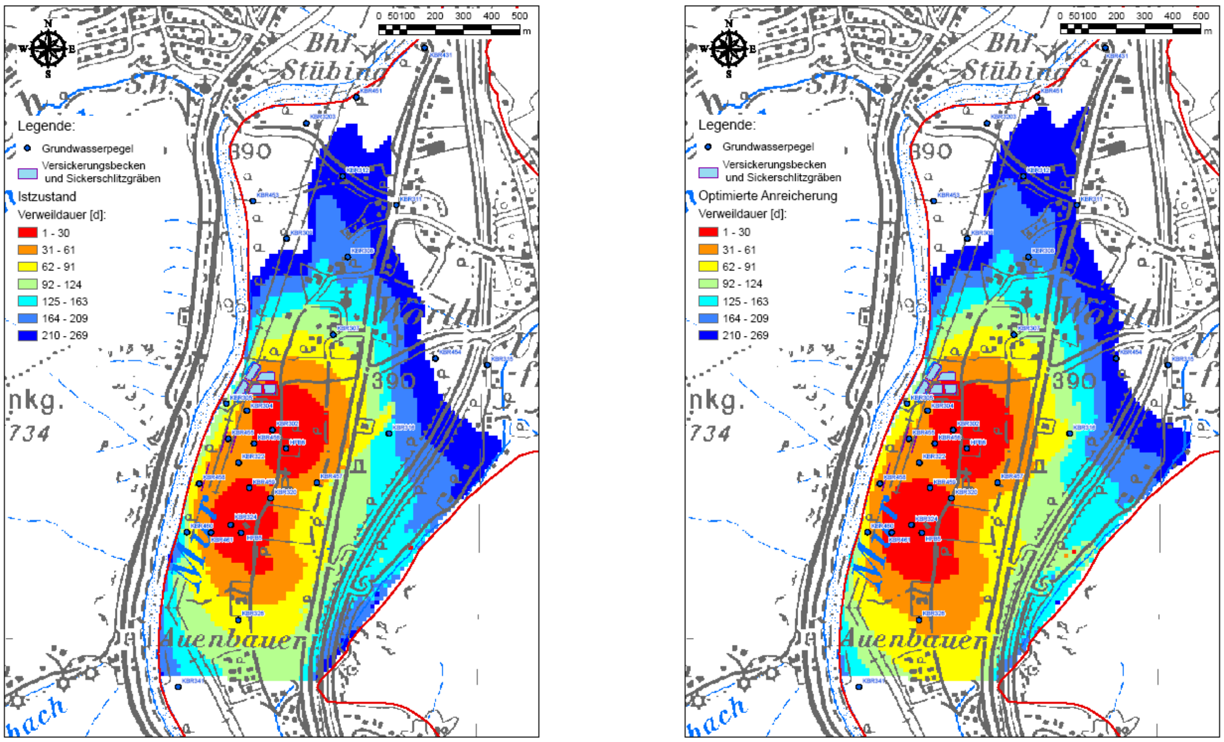

Figure 2: Extent of the capture zones of the two pumping wells for the actual artificial recharge operation between 2004 and 2006 (left) and for a hypothetical recharge regime considering the subsurface travel times to the pumping wells (right).

To improve the efficiency of the recharge efforts various scenarios were run with the groundwater flow model that consist of different operation modes of the infiltration, the extraction and combinations of both. Such scenarios include among others evaluating the effect of an additional infiltration basin (in volume equivalent to a lawn basin) at selected locations, finding a balanced partitioning of the extraction volume between the two pumping wells (currently the wells are operated alternating on a daily basis) or constantly feeding the sand basins during the winter period where the blooming of algae does not affect the infiltration (for more details see Kupfersberger, 2008).

The results of only one scenario should be discussed here. On the right hand side of Figure 2 the capture zones of the two pumping wells are displayed for a scenario where the time and amount of infiltration is adjusted according to the travel time between the individual infiltration structures and the pumping wells. The total infiltration volume is kept the same, however, the extraction volume is increased until the extension of the 60 days travel time zone reaches about the same location north of the infiltration basins as in the case of current operations (left image of Figure 2). By coordinating artificial recharge with withdrawal in that manner about 20% more groundwater can be extracted from the aquifer without noticeable extending the well capture zones.

Infiltration costs

Figure 3 shows the development of external costs (i.e. material costs and external contracts) in € related to maintenance and operation of the artificial recharge site in Andritz. The increase in 2002 is due to attributing a new cost category to the recharge operations which before that time was allocated to a different account. It can be seen that the main cost positions are the maintenance of the well protection areas (in average 59,000 €/year) and of the raw water intake (in average 21,500 €/year). The high amounts for maintaining the well protection areas between 1997 and 1999 are related to a storm event in 1997 and the following clean up efforts in the riparian forest region. The maintenance costs of the operational building is a cost position that is not completely linked to the artificial recharge operation since the building houses the central control unit of the withdrawal pumps and the drinking water reservoirs as well as some repair facilities. The maintenance of the infiltration basins adds up to 21,000 € per year. The overall average external costs during the analysis period is about 185,000 € per year. The second recharge site of the Graz AG in Friesach shows a very similar structure of external costs.

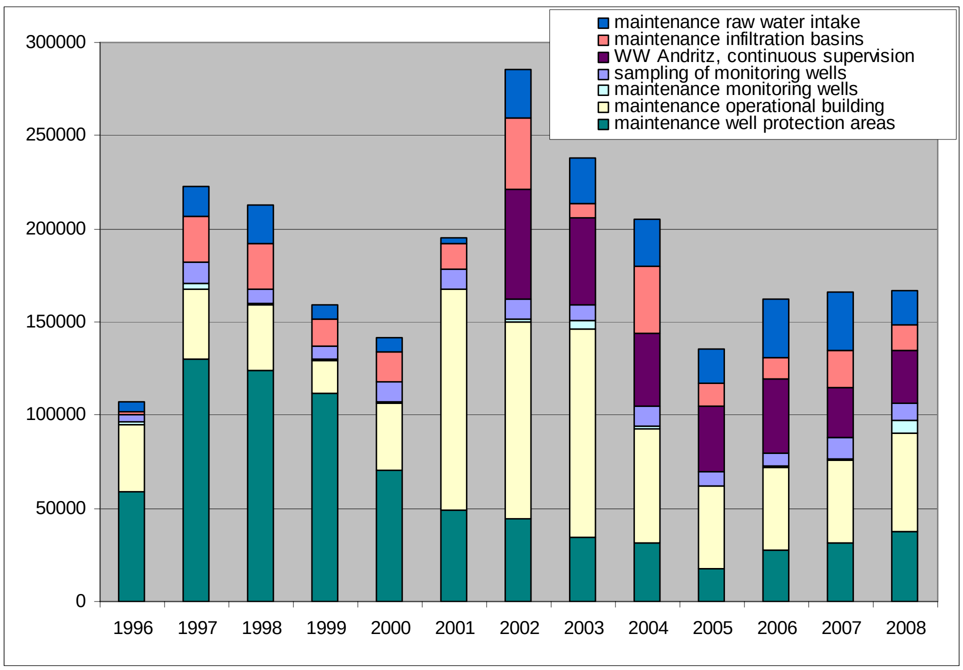

Figure 3: Composition of external costs (€) related to maintenance and operation tasks at the artificial recharge site Andritz between 1996 and 2008.

In the above costs the internal labour costs of Graz AG employees are not included. These costs total just under 500,000 €/year for the two recharge sites combined and are by far the most important cost position. Adding up all costs related to the artificial recharge operations yields an average of about 760,000 €/ year. However, since 2002 a continuous decline of the total costs is observed to the mark of about 600,000 €. If the total costs are correlated to the infiltration amount during the analysis period average costs of 13.7 Cent /m³ arise. Adjusting the yearly costs to inflation with reference to the year 2008 yields specific infiltration costs of 16.8 Cent/m³. However, it has to be clear that this value can only be an approximation since some of the cost positions will incur to the same extent even if significantly less water is infiltrated.

Parts of the infiltration facilities have been built as early as 1955 (water intake weir at the Andritzbach) with the majority being constructed in the early eighties. Given the construction cost index of 2008 the overall construction costs for the two recharge sites add up to about 9.5 million €. The raw water intakes and the pre-treatment structures are the largest elements of costs totalling 5.6 million € each. Given the same infiltration volume the associated costs vary from 1,739 € for the sand basin to 1,950 € for the lawn basin. In order to arrive at infiltration volume related construction costs individual life time periods of the different recharge facility components are considered. This procedure results in depreciation of construction costs of 3.6 Cent/m³ on average, accordingly adjusted for inflation, or about 216,000 €/year. Thus, from the 30 years experience of artificial groundwater operations in Graz consolidated infiltration costs of 20.4 Cent/m³ can be reported. Land acquisition costs are not included in this figure which will very much depend on the specific location.

Dillon et al. (2009) took a different economic approach examining the value instead of the costs of managed aquifer recharge. They have compared the utilization of managed aquifer recharge in 4 Australian cities and identified benefits by water storage, water conveyance and water quality improvement during the subsurface passage. All together, the authors estimated the value of managed aquifer recharge between AUS$ 1.5 and 6. Thus, compared to the costs computed for the recharge sites in Graz, the value outnumbers the costs by a large ratio.

CONCLUSIONS

Diverse aspects of the existing recharge facilities in Graz, Austria, have been investigated including the compromise between retention and infiltration rates of different sand mixtures or planted top layers of infiltration basins. The experiments yielded important knowledge for possible adaptation of infiltration basins or adding new basins since the water quality after the passage through infiltration basins is typically unknown. Furthermore, it has been shown that with the help of a transient groundwater flow model the operation of infiltration and withdrawal can be optimized by coordinating the two activities. Such a model is a valuable tool to predict the highly variable impacts of any intended or externally forced change of operation conditions and can also assist in establishing or modifying the design of a site specific monitoring network. It is expected that in the future more online measurement devices for observing groundwater level and quality will be put in operation. Thus, real time knowledge about groundwater conditions will become available which in combination with an adequate data base can be used for sound management decisions. The average operation and maintenance costs combined with the depreciation costs of the infiltration sites in Graz indicate that managed aquifer recharge is a promising component to sustain drinking water supply also in humid regions.

ACKNOWLEDGEMENTS

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Contribution of percolation tanks to total aquifer recharge: the example of Gajwel watershed, southern India

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Abstract

Hard rock aquifers located in semi-arid climatic conditions are especially prone to overexploitation because of limited storage and recharge. In Andhra Pradesh (southern India), such geological and climatic settings prevail and rapid increase in groundwater abstraction for irrigation has led to aquifer overexploitation in many districts. As a response to overexploitation, Central and State governments have launched watershed development programmes aiming at augmenting aquifer recharge using different man-made structures such as percolation tanks, check dams, defunct dug wells.
The objective of the present study is to determine the contribution of percolation tanks to the total aquifer recharge in a typical semi-arid hard rock watershed of southern India (Gajwel, 84 km^2) and develop a simulation tool to test the impact of various climatic scenarios/tank management strategies on tank percolation fluxes and overall groundwater balance. The modelling approach consists in a first component which simulates runoff from daily rainfall as well as volumetric storage in tanks. A second component computes deep percolation from tanks based on a tank water balance approach. The model is validated by field observations in part of the watershed (temporal variations of tank volumetric storage). Estimation of the recharge with the water table fluctuation method is about 21% of the monsoon rainfall whilst estimations of the additional artificial recharge, inferred from the tank water balance and the runoff model, range from 5 to 8%.

Keywords: tanks; hard rock aquifer; semi-arid climate; water management; model; India

Introduction

In most regions of southern India, groundwater resources are constituted by shallow hard-rock aquifers with modest yield and thickness limited to a few tens of meters (weathering layer). With the development of groundwater irrigated agriculture, these aquifers have become intensively pumped and water levels are on a decreasing trend in many areas which is indicative of alarming consequences on future water resources if no adequate management response is implemented.
A possible management solution, encouraged by Government agencies over the recent years, is to intervene on the supply side of the groundwater balance, i.e. by artificial recharge. For instance, the State of Andhra Pradesh set the objective to increase aquifer recharge from 9% of total rainfall under natural conditions to 15% by the year 2020 (Government of Andhra Pradesh, 2003).
Artificial recharge structures encompass percolation tanks, check dams, injection wells, defunct dug wells. In many places, a high density of tanks exists as they were the traditional water source for many centuries. The conversion of these traditional tanks into purely percolation tanks is increasing day by day (Sakthivadivel 2007). Several studies have looked at the hydraulic functioning of these tanks and their contribution to recharge augmentation (Muralidharan et al. 1995, Selvarajan et al. 1995, Sukhija et al. 1997, Gore et al. 1998, Sudarshan 2003, Machiwal et al. 2004, Sharda et al. 2006). The efficiency (i.e., volume percolated compared to total volume stored) of these artificial recharge structures ranges from 30% (Sukhija et al. 1997) to 57% (Mehta and Jain 1997).

The present study focuses on a representative hard-rock (Archean granite) semi-arid watershed where groundwater is intensively pumped for irrigated agriculture (Gajwel watershed, 84 km^2, Medak district, Andhra Pradesh, Figure 1). Over 30 tanks are distributed within the watershed and no surface runoff is observed at the outlet except in the case of exceptional monsoon rainfall (once every 5-10 years).
The objective is to compare the contribution of artificial recharge to the overall recharge computed at the watershed scale and discuss alternative management options.

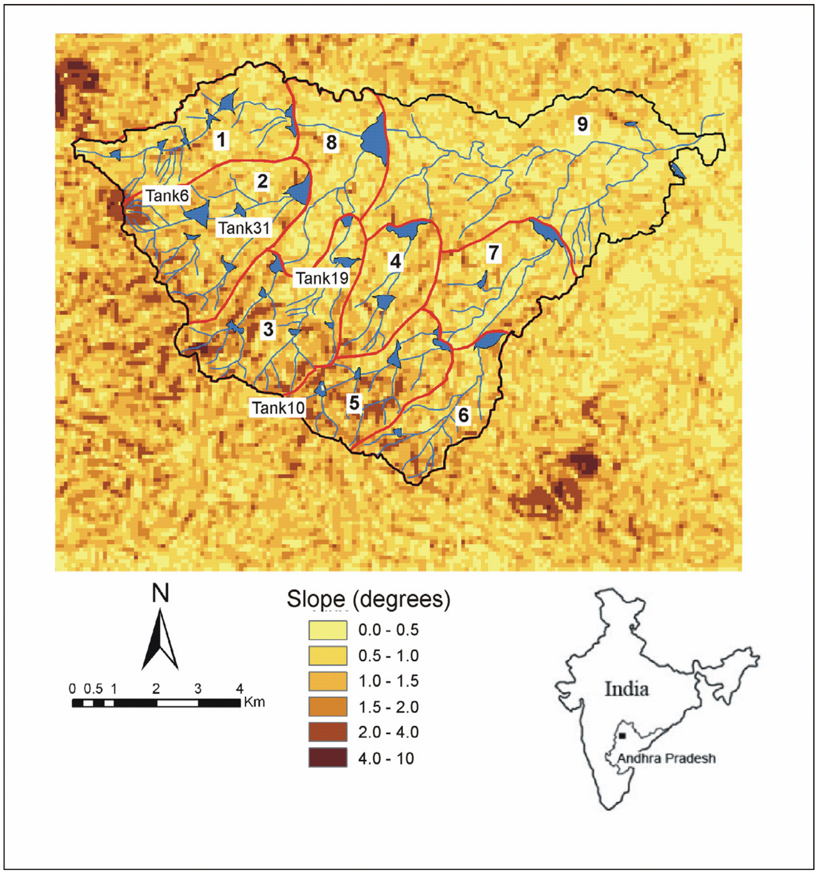

Figure 1: Location of Gajwel experimental watershed in southern India and slope map of the watershed with tanks in blue and drainage network. The watershed is split into 9 sub-watersheds numbered 1-9.

Computation of total aquifer recharge using groundwater balance

The groundwater balance is given by (e.g., Marechal et al. 2006):

$$R + RF + Q_{in} = E + PG + Q_{out} + Q_{bf} + \Delta S_{gw} \quad (1)$$

where R is groundwater recharge, RF the return flows, Q_{in} and Q_{out} the groundwater flows across the watershed boundaries, E the evaporation from the water table, PG the abstraction of groundwater by pumping, Q_{bf} the baseflow (groundwater discharge to streams) and ΔS_{gw} the change in groundwater storage:

$$\Delta S_{gw} = S_y \cdot \Delta h \quad (2)$$

With S_y the specific yield of the aquifer and Δh the water table fluctuation.

In Gajwel watershed (and other intensively exploited hard-rock watershed of southern India), Q_{in} balances Q_{out} across aquifer boundaries because in the upstream parts of the watershed, the piezometric surface describes a ridge-line (i.e., no flow boundary) which matches the surface watershed boundary (i.e., topographic ridge-line) and in the downstream part of the watershed intensive pumping is present on both

sides of the watershed limits thus strongly limiting lateral groundwater flows; Q_{bf} is non-existent as the water table is disconnected of the stream beds.
Therefore equation (1) can be simplified into:

$$R + RF = E + PG + S_y \cdot \Delta h \qquad (3)$$

Estimation of PG
A landuse interpretation from satellite imagery (ResourceSat MX-LISS IV, 5 m resolution) has been used to obtain the surface area for the different irrigated crops (rice, vegetables, mangoes orchards) for both the dry season (rabi) and the rainy season (kharif) (Perrin et al. 2008). Field surveys were carried out to estimate the average daily irrigation for the different crops. On this basis, it is possible to compute the seasonal groundwater abstraction per irrigated crop. To obtain the total groundwater abstraction, other uses such as poultry farms and domestic uses have to be accounted for (Table 1). Total annual groundwater abstraction is 248 mm (of which 243 mm is for irrigation).
As per 2008 statistics (Mandal Revenue Office Gajwel, comm. pers.), the number of irrigation wells within Gajwel watershed is 1134. Taking an average discharge of 12 m^3/hr (average from 37 wells measured in the watershed), a daily pumping of 6 hrs (Massuel et al. 2008), and a yearly pumping of 270 days (two paddy cultivations), annual abstraction is 262 mm (22.0 x 10^6 m^3), estimation which compares well with the abstraction computed from landuse.

Table 1: Computed groundwater abstraction, Gajwel watershed. Stars indicate inferred values.

	Irrigation_mm/day		Irrigated area_ha		Groundwater abstraction_mm	
	Rabi	Kharif	Rabi	Kharif	Rabi	Kharif
Rice	12	9	412	693	89	112
Vegetables	9	6*	113	113*	18	10
Orchards	9*	0*	93	93	14	0
Domestic					2.8	2.0
Poultry					0.2	0.2
Total					124	124.2

Estimation of RF
Irrigation return flows (IRF) have been calculating using a similar methodology than Dewandel et al. (2008) with a combination of water balance at the field scale and 1-D vertical variably saturated model so as to estimate average IRF at the seasonal scale. It is found that irrigation return flow coefficients C_f ($C_f = IRF/PG$) are comprised between 37-58% for rice and 36-55% for vegetables. For mangoes orchards, IRF is negligible as mostly drip irrigation is used. For domestic use and poultry, C_f is estimated to be 20% (Marechal et al. 2006).

Estimation of Δh and E
Piezometric campaigns were conducted after the monsoon has ended to obtain the highest water table (November) and at the end of the dry season to obtain the lowest water table (June). Between 40 and 55 piezometers (i.e., abandoned borewells) were measured for each campaign. The depth to water table is also used to estimate the evaporation flux from the water table using the relationship of Coudrain-Ribstein et al. (1998).
Equation (3) is solved twice a year, once for the dry season when recharge R is nil and therefore (3) is solved for S_y and once for the monsoon season for computing R (Marechal et al. 2006). On this basis, the groundwater budget at watershed scale has been computed for the years 2006-2009 (Table 2).

Table 2: Gajwel watershed groundwater budget, years 2006-2009. The star indicates that recharge had taken place in April 2008 due to cyclonic conditions during the dry season, thus invalidating the water fluctuation method for this period.

Year 1	Rainfall [mm]	ΔWT [m]	R[mm]	RF [mm]	E [mm]	PG [mm]	Balance [mm]
Jun2006-Oct2006	972	4,6	200	64	0,4	124	139
Nov2006-May2007	130	-2,1	0	59	0,4	124	-66
Annual data				122	0,8	248	73
Year 2							
Jun2007-Oct2007	493	0,8	101	47	0,3	124	23
Nov2007-May2008	235	-2,0	*	65	0,4	124	
Annual data				112	0,7	248	
Year 3							
Jun2008-Oct2008	730	3,0	147	59	0,4	124	82
Nov2008-May2009	100	-2,5	0	55	0,4	124	-70
Annual data				114	0,8	248	12

Estimation of tank storage capacity

For tanks 6 and 10 (Figure 1), the reservoir capacity has been computed based on GPS tracking around the tank shores at regular interval during their depletion so as to obtain the tank elevation contours. The final tank bottom topography was obtained by geostatistical interpolation (kriging).

Tank surface areas can be easily determined using satellite imagery and topographical maps. However, for water balance calculation, a relationship linking the observed area with the tank storage capacity (i.e., water volume stored) need to be established. In Gajwel watershed, tanks are located along drainage axes in a quite homogeneous landscape (no large change in topographic gradients, valley size, etc.). Therefore it is hypothesised that tank geometry is quite similar across the watershed and a single relationship may be used to compute tank storage volume from tank area (Figure 2). The relationship can reproduce storage volumes of tanks 6 and 10 within 20% of their actual volumes.

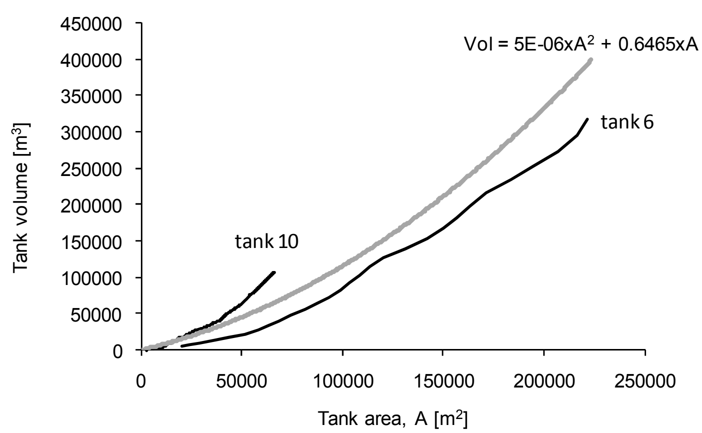

Figure 2: model used to compute tank storage volume from tank area.

Estimation of runoff

The detailed monitoring of rainfall (automatic tipping bucket rain gauge) and tank 10 filling (15 min interval pressure datalogger) during the 2007 and 2008 monsoons brings insight on the rainfall-runoff processes in Gajwel watershed. At the onset of the monsoon in June, the first 40-50 mm of rainfall replenish the soil moisture deficit (SMD) developed during the dry season. Then the first floods are observed, with surface runoff resulting from Hortonian overland flow, namely by infiltration excess but also saturation excess later in the monsoon season. On a daily basis, a simple linear relationship is proposed (Eq. 4) with runoff (Q) occurring when the total rainfall (R_d) exceeds the threshold (T). T decreases as the cumulative monsoon rainfall ($\Sigma Rain$) increases.

for $\Sigma Rain > 50mm$
and $T_{\min} \le \Sigma Rain < T_{\max}$

$$Q(t) = C_r \cdot R_d(t) \tag{4}$$

with C_r the runoff coefficient.

The calibrated model with C_r=0.22, T=30mm for $\Sigma Rain$ <200mm, T=15mm for $\Sigma Rain$ between 200mm and 400mm, T=10mm for $\Sigma Rain$ >400mm, reproduces quite well the progressive filling of tank 10 (Figure 3). Simulated versus observed water inflow in tanks 6 and 10 are plotted on figure 4 with a Nash coefficient of 0.994.

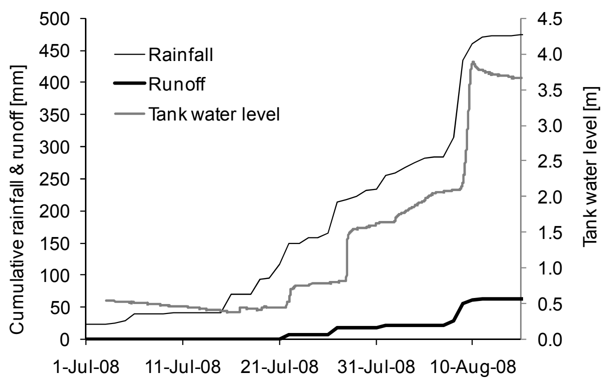

Figure 3: cumulative runoff from simulated daily runoff and observed tank water level, monsoon 2008.

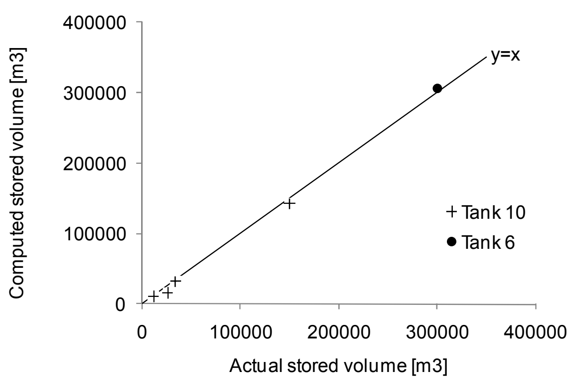

Figure 4: comparison of computed stored water volume in tanks 6 and 10 using the runoff model with observed stored volume (monsoon 2007, rainfall event spring 2008, monsoon 2008 including tank 6), Nash coefficient is 0.994.

Estimation of artificial recharge from tanks

Observed tanks depletion

Earlier studies (e.g., Sukhija et al. 1997, Mehta and Jain 1997, Perrin et al. 2009) have shown that the main fluxes out of tanks are evaporation (including transpiration by plants within the tank) and recharge; additional minor outflows are domestic and livestock uptakes and overflow.

Depletion rates of the four monitored tanks (Figure 1) are quite similar (Figure 5) and range between 16 and 20 mm/day. As evaporation rate is assumed to be more or less the same (tanks are within a few km), it means that recharge rates are in the same range. This observation indicates that observed depletion rates may be extended to all the tanks of the watershed.

Daily tank balance computation shows that percolation efficiency is between 40-65 % (56% tank 6 depletion 2008, 40-63% tank 10 2007-2009) of the tank capacity, similar to earlier studies (Sukhija et al. 1997, Mehta and Jain 1997, Perrin et al. 2009).

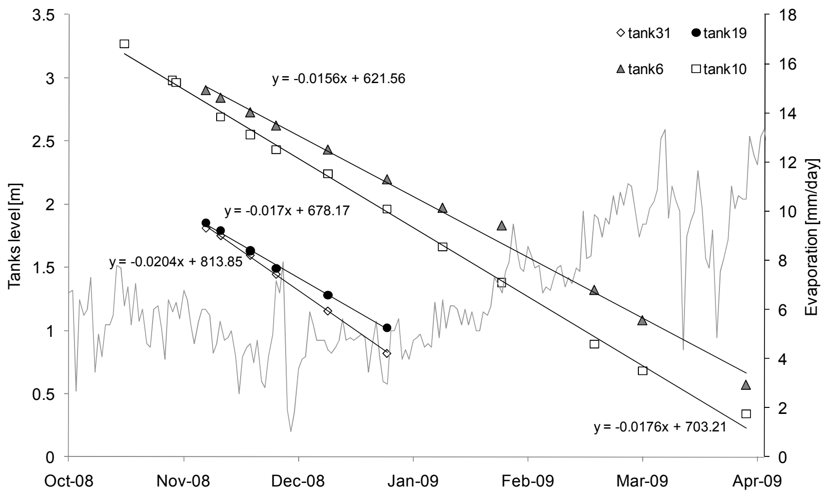

Figure 5: Tank depletion curves for the dry period October 2008 to April 2009, Gajwel watershed. Daily pan evaporation from ICRISAT meteorological station is also shown.

Computing artificial recharge from percolation tanks

For each sub-watershed (Figure 1), volumetric runoff is computed on a daily basis using the calibrated rainfall-runoff model and the respective sub-watershed surface areas. Then a daily tank water balance is computed taking into account loss by evaporation, recharge, overflow (after tank maximum storage capacity has been reached), uptakes and gain due to additional runoff. Water fluxes are computed for each sub-watershed for two values of percolation efficiency (0.4 and 0.6) which cover the range found in this study and other studies.

Sub-watersheds 1-6 are headwaters and will only overflow; sub-watersheds 7 and 8 have their own catchment and in addition will receive the overflow of respectively sub-watersheds 5, 6 and sub-watersheds 1, 2, 3. It is hypothesised that no runoff occurs in downstream sub-watershed 9 as slope is negligible and thick black soils with high storage capacity dominate.

Results (Table 3) show that most of the runoff is captured by existing tanks with outflows occurring only during the rainy years 2006 and 2008 (less than 10% of total runoff). The main fluxes are evaporation and percolation with evaporation higher than percolation for simulation with percolation efficiency of 40% and the reverse for simulation with percolation efficiency of 60%.

Table 3: Water budgeting of tanks at watershed scale for the lower (0.4) and higher (0.6) bounds of percolation efficiency. The year 2008-2009 budget is slightly truncated as model was run until end of April (May-June missing due to absence of meteorological data).

Perc.efficiency: 0.4					
	Runoff [mm]	Evap. [mm]	Recharge [mm]	Outflow [mm]	Uptake [mm]
year06-07	123.0	54.5	40.1	12.9	1.7
year07-08	54.6	30.1	20.7	0.0	1.7
year08-09	75.3	35.9	29.5	4.7	1.4
Perc.efficiency: 0.6					
year06-07	123.0	40.3	65.4	9.2	1.7
year07-08	54.6	20.5	29.9	0.0	1.7
year08-09	75.3	25.3	45.3	2.2	1.4

Discussion and conclusions

Recharge computed with the water table fluctuation method represents about 21% of monsoon rainfall and artificial recharge from tank computed using tank water balance and runoff model is comprised between 5 and 8% of monsoon rainfall (Figure 6). The remaining fraction of monsoon rainfall (71-74%) is mostly consumed by evapotranspiration and limited outflows in case of high monsoon. These results indicate that there is limited scope to build additional recharge structures as most runoff is already captured; however increased recharge may be achieved by improving the performance of existing percolation tanks. For instance if percolation efficiency is increased from 0.4 to 0.6, 10-20 mm more artificial recharge is generated (Table 3). This improvement may be achieved by increased hydraulic gradients near tanks and/or tank desilting. Another more efficient way to manage tanks would be to use directly tank water for irrigation so as to decrease irrigation pumping which represents a large share of the groundwater budget (Table 2) (Perrin et al. 2009).

The water table fluctuation methods used to estimate recharge assume that no recharge occurs during the dry season. This is not true in the vicinity of tanks as between 35-55% of artificial recharge occurs during dry season. For the method to remain valid, it is strongly advisable to use piezometers away from tanks so that they are not influenced by artificial recharge. If this condition is respected, the total recharge to the aquifer will be the sum of the recharge from the water table fluctuation and artificial recharge. In Gajwel, total recharge is therefore between 26-29% of monsoon rainfall (Figure 6).

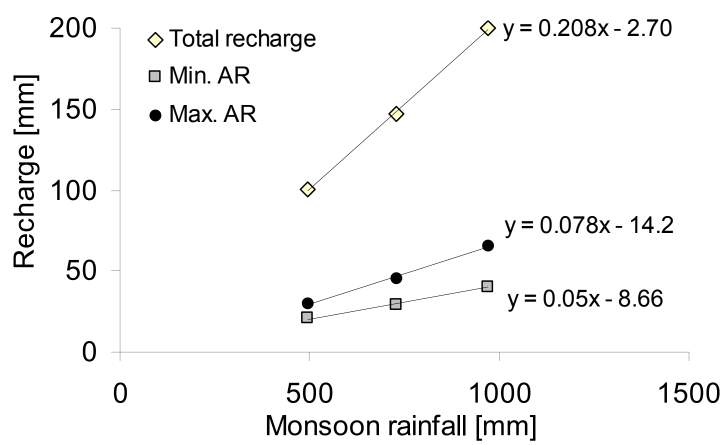

Figure 6: linear relationship between monsoon rainfall and recharge (total recharge from groundwater budget calculation, minimum artificial recharge (AR) with percolation efficiency at 0.4, maximum AR with percolation efficiency at 0.6).

The presented approach represents a preliminary estimate of the artificial recharge at the small basin scale but provide enough information to carry out further investigations. A larger panel of reservoirs has to be monitored within different soil conditions (i.e. red soil and black soil) for a proper rainfall-runoff model to be developed. Additional time series are required to improve the reliability of the results since, as shown by year 2008, the climate variability is very high.

In the framework of a new project, it is planned to apply a SWAT modelling approach to the watershed which will further consolidate the water budget estimates. Based on the modelling efforts, it will then be possible to test different water management strategies for planning sustainable scenarios.

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Integrating Aquifer Storage and Recovery (ASR) into a recycled water supply system for new large urban residential development in Geelong, Victoria, Australia.

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ABSTRACT

New growth areas in the city of Geelong (Victoria, Australia) have presented opportunities for large and therefore economically viable recycled water supply systems. With drought stressing water resources and climate change influencing longer term negative impacts, there are strong drivers for these alternative water supply schemes.

One development area, Armstrong Creek, will contain 22 000 homes, commercial and industrial zones, multiple schools, and public open space areas. The supply of recycled water to each property in this development will be used for toilet flushing, garden watering, and other low risk activities. It is expected that garden watering, will contribute to significant summer peaking factors (greater than 3 times average daily demand) which indicates the need for large winter and/or balancing storage to balance treatment plant production. Once the development is complete, over 4 000 ML of recycled water will be supplied annually.

Cost comparisons show a clear benefit of ASR over traditional surface storage in terms of construction cost, land acquisition and many other factors. As an added system component, the increased storage resulting from ASR will allow deferral of expensive treatment plant upgrades and overall cost. The large storage accessible also makes it the best tool to achieve maximum recycling from water reclamation plants.

Although the suitability of ASR at Armstrong Creek and other areas is yet to be fully evaluated, the potential benefits have been sufficient to warrant significant investment into field investigations of the local aquifer system which occurs at depths of up to 250 m.

This paper presents the business strategy supporting the integration of ASR into centralised water recycling systems, and focuses on the economic justification for pursuing the detailed and costly investigations required.

Key Words: Aquifer Storage and Recovery, Cost Estimates, Recycled Water, Victoria, Australia.

Nomenclature and units used: 1 megalitre (ML) = 1000 cubic metres (m^3) = 1000 kilolitres (kL), AUD = Australian Dollars, AHD = Australian Height Datum.

INTRODUCTION

The city of Geelong is located 75 km south west of Melbourne, in Victoria, Australia. To the south of Geelong lies the main growth area for residential development, known as Armstrong Creek. Over the next 20 to 30 years this area will become home to up to 60 000 people living in 22 000 new homes. The area is being planned with firm sustainability principles in mind, aimed at creating a liveable community with more environmental and social sustainability.

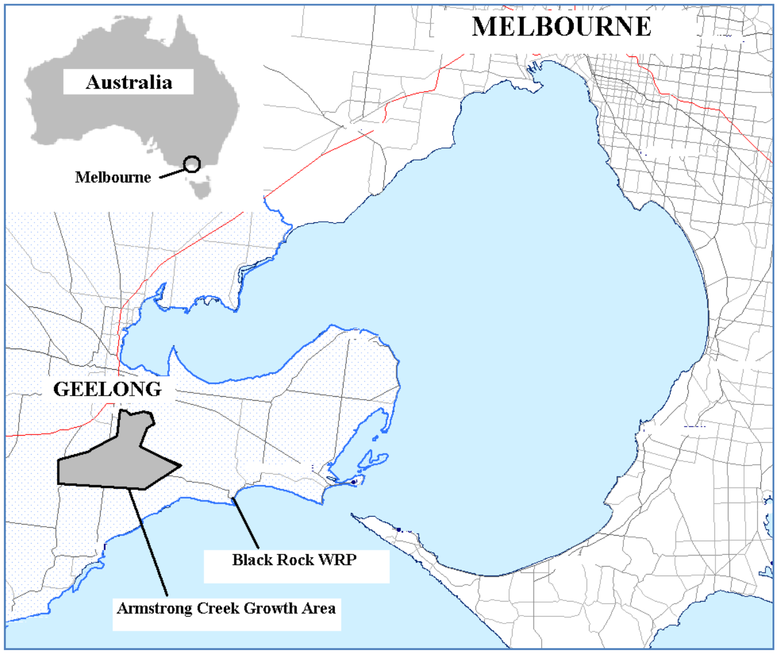

Figure 1: Location of the Armstrong Creek Development

This approach to sustainability has lead to a strong desire for recycled water to be utilised in the development to offset potable (drinking) water demand. The most obvious and cost effective alternative water source for this area is the Black Rock Water Reclamation Plant (BRWRP), which is conveniently located just a few kilometres south west. BRWRP is Geelong's main water reclamation plant, currently treating on average 45 ML/d.

As the result of good cooperation between government, developers and the local utility (Barwon Water), a new alternative supply system will be built to service this area alongside the standard potable supply (referred to collectively as "dual pipe supply"). This will comprise a new Reverse Osmosis-Ultra Filtration (RO-UF) treatment plant to be constructed at Black Rock which will improve the currently treated wastewater to an adequate standard (designated by the Victorian government environmental and health regulators as "Class A"). The original proposal for the system included a traditional storage, transfer and reticulation system to deliver this recycled water to the area.

ASR Opportunity

This scheme presents a somewhat rare opportunity in the Australian context for both recycled water and ASR feasibility. Here we see that this proposed system contains the key essential ingredients discussed by Dillon et al (2009) for a successful Managed Aquifer Recharge (MAR) or ASR scheme.

In particular for a dual pipe supply or an ASR scheme, this area offers:

- A guaranteed group of customers that will expand quickly over 20 years,
- Users that are located close to existing infrastructure/source (i.e. BRWRP), and
- A potentially suitable aquifer (cost dependent) for storage and recovery.

This case for recycled water supply can then be potentially improved further by incorporating an ASR system. The economic, social and environmental benefits that ASR can offer are well known, and include more localised storage opportunity, and a reduction in the level of evaporation, algae and mosquito populations (Dillon et al, 2010), Furthermore it also has the potential for consideration as a water quality protection barrier (Page et al, 2010). Importantly In this case it can also better satisfy the need for:

- Cost effective storage that can be staged with flexibility (i.e. combating uncertainty in demand projections), and
- A very large seasonal storage in the future, once the bulk demand increases, that may not be possible otherwise.

METHOD

The methodology outlined here is consistent with the approach presented in the Australian Managed Aquifer Recharge Guidelines (NRMMC-EPHC-NHMRC 2009). Given the expense estimated (up to $2.5 Million AUD) to fully investigate and prove this as a truly feasible and safe opportunity consistent with these guidelines, a careful approach has been taken to staging the investigation program, and therefore limiting the risk to the organisation.

By having multiple stages of preliminary investigation and review the likelihood of unnecessary cost being invested in such an exercise is reduced. This is illustrated in Figure 2, however there are multiple hold points and sub stages even within this broad description.

Figure 2 also outlines the concept of project risk compared with a more traditional approach. This concept is important to ensure each project stakeholder (management, regulators, community etc) understands and is aware that the organisation may choose to tolerate increased project risk to allow great benefits later, if the scheme is proven.

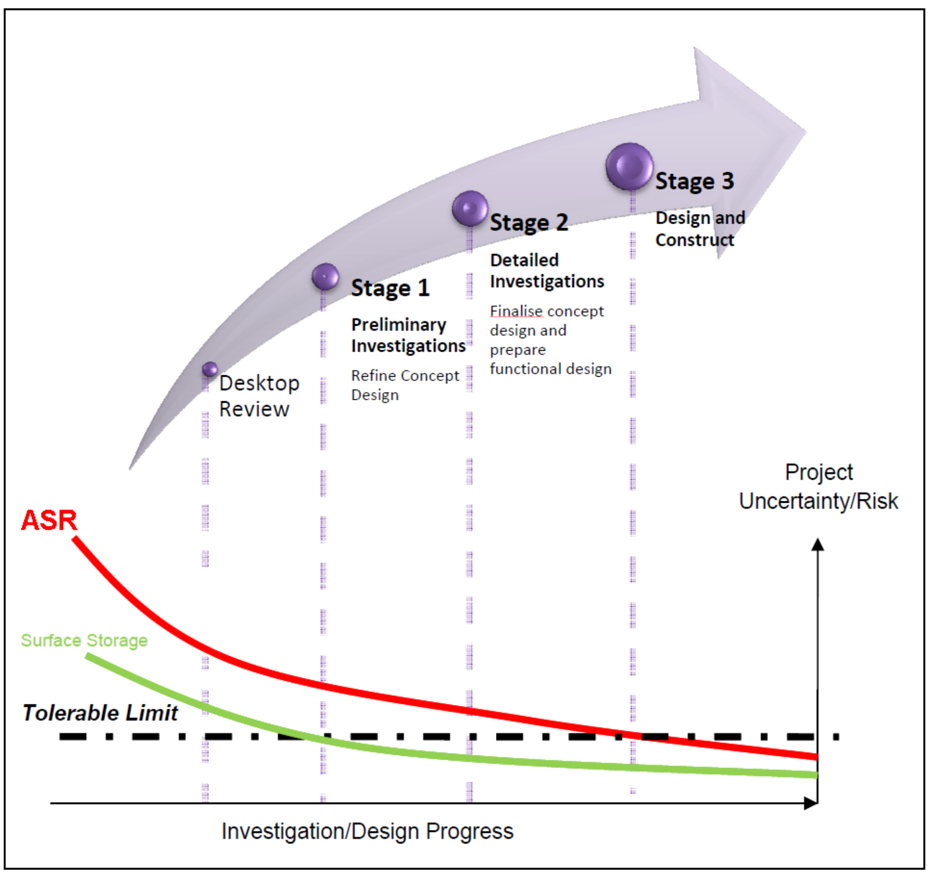

Figure 2: ASR Research and Development Program, based on the approach presented in the Australian MAR guidelines (NRMMC-EPHC-NHMRC 2009)

With this initial assessment the approach has more focus on the core business economics first.
A preliminary risk assessment has generally satisfied the social and environmental questions for now, until further research is undertaken which will enable more tangible assessment of potential impacts. Particular knowledge gaps that will be satisfied in the next stages include:

- Comparative Greenhouse Gas Emissions,
- Regulatory management gaps (licencing etc), and
- Risk to any high value Groundwater Dependent Ecosystems (GDEs).

Hydrogeological Review

A Preliminary review of field data indicates that a number of potential target aquifers are present in this Armstrong Creek/Black Rock area.

However, a formation known as Torquay Group (TGp) occuring in this area, nominally between 0 m AHD and -250 m AHD, offers the best opportunity for maximum storage volume and transmissivity. The TGp formation is generally comprised of interbedded limestone with marls (clay seams).

The native groundwater in this aquifer is understood to be relatively saline, with bore water sampling indicating a range between 1000 and 7000 mg/L. This is important for the planned recovery efficiency of the system, where more injectant water may mix with the native groundwater and loose value. However, the existing salinity levels have also lead to a limited number of stock and domestic bores being installed in the area.

Estimating transmissivity and storage coefficient, using limited borehole and pump test data, was important to develop an appropriate concept design. For this study, a transmissivity value of 50 m^2/day, and a a storage co-efficient of 7.5 x 10^{-4} have been used to complete the required estimates. These values along with the concept designs, will be further refined as more extensive field work is undertaken.

Concept Design

Using the review of hydrogeological information and the currently planned treatment and transfer infrastructure, a generalised concept design for an ASR system was developed that could be staged to fit the proposed sequence of development for this recycled water scheme. As the existing transfer main alignment has been proposed it is logical that any ASR system bores would be installed close to this pipeline to minimise additional cost.

An important step in sizing the infrastructure for this conceptual scheme involved simple water balance modelling. Predicted demand patterns and growth from the Armstrong Creek development were used against the proposed recycled water treatment plant staging to assess both the excess water able to be injected, but also the deficit over the summer period when recovery would be required.

Using this approach, the ASR system was found not only to aid seasonal peak requirements, but also to greatly defer the originally considered treatment plant upgrades.

Originally a treatment plant upgrade would be required in 2015/16, but if the aquifer can in fact allow the “banking” of water in the initial 7 years, whilst the demand from Armstrong Creek is low, then the upgrade will not be required until 2025/26 or later. This is illustrated in Figure 3.

As the demand stabilises once the current growth zone is complete, there will potentially be water already available in the aquifer for new markets.

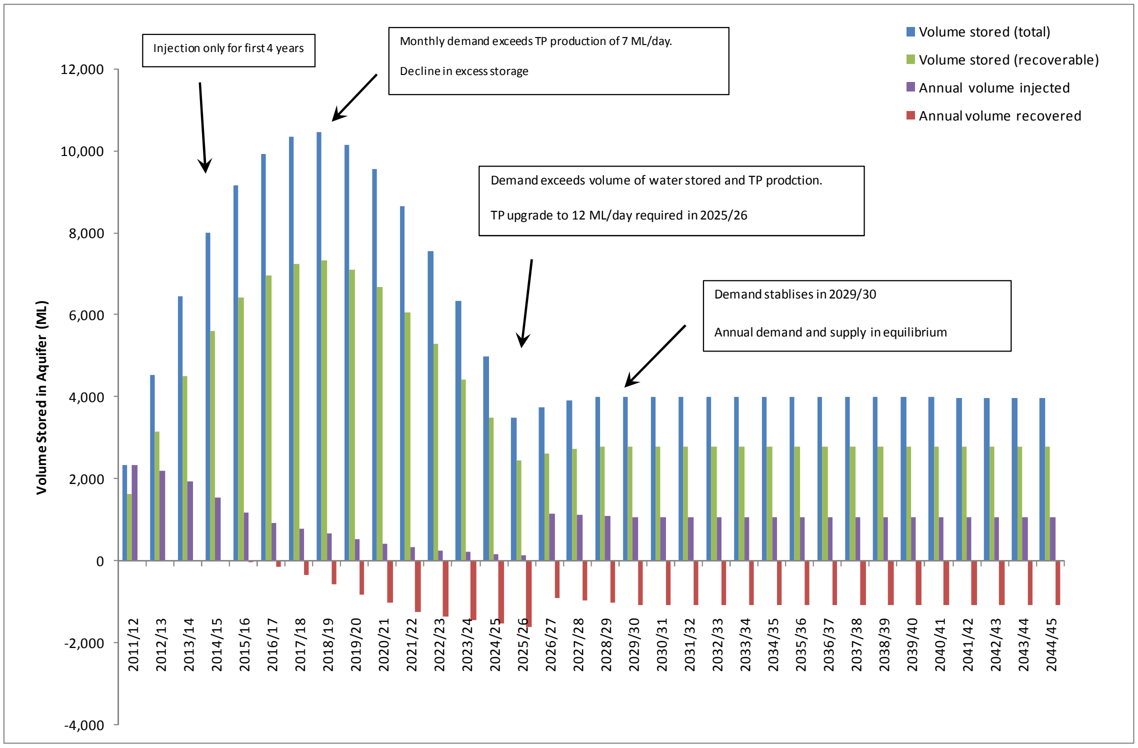

Figure 3: Graph showing injected recycled water accumulating to defer future upgrades.

By creating a concept design based on the principle of maximising this deferral of costly capital treatment plant upgrades, savings can be expected despite conservative operational cost estimates applied to the ASR component.

By comparing conceptual ASR alternatives with an indicative base case an argument is created to pursue ASR based on proving three key ideas:

1. That an ***ASR scheme equivalent*** to a small surface storage can be cost competitive and offer significant triple bottom line advantages,
2. That an ASR scheme can ***defer treatment plant upgrades*** through banking water for multiple years in the right hydrogeological environment, and
3. That an ASR scheme involved in a much larger seasonal function in the future can more easily achieve a ***maximum water recycling scenario***.

Using these ideas cost estimates were constructed and then inputted into the NPV pricing model. The cost estimates are based on information from local capital works and operational activities at Barwon Water.

An important component in building this cost comparison has been how much operational cost to apply to each of the ASR concept schemes. For this, the costs incurred in managing an existing groundwater bore field nearby (Barwon Downs) was used. Although the use of this source is highly variable, due to its use in drought response, on average this 4 to 6 bore (500 to 600 m deep) bore field is considered to have a more constant operating cost of approximately $ 150 / ML.

Generally the estimates were generated with a conservative approach in mind. For example considerations included:

- Recovery efficiencies of 70 % of the total stored water per year being recoverable,

- Little or no technological improvements over the lifetime of the scheme, and
- High peaking factors (up to 3 times average daily demand) applied to Infrastructure design (adding cost to infrastructure sizing).

A Net Present Value (NPV) pricing model was used, incorporating a full cost recovery "Building Blocks" approach to produce a cost per ML value for the base case (no ASR component) and each of the 3 key ideas as progressive options for comparison.

RESULTS

The cost estimates are presented in the following table, which describes the main infrastructure differences considered and the resulting cost based results.

Criteria	Base Case Comparisons			*Preliminary Assessment Only* Ultimate Recycling Goal[11]	
	Base Case	***ASR Equivalent***	***ASR – Max Treatment Plant Deferral***	***With Surface Storage***	***With ASR Storage***
Total Scheme CAPEX	$ 71.5Million	$ 64.5 Million	$ 53.6 Million	*~$ 321 Million*	*~$ 243 Million*
Peak Scheme OPEX	$ 580 / ML	$ 609 / ML	$ 730 / ML	*$ 550 / ML*	*$ 730 / ML*
Class A Plant[10]:					
- **Built Plant 2013**	7 ML/d ($ 6.5 Mill)	7 ML/d ($ 6.5 Mill)	7 ML/d ($ 6.5 Mill)	*7 ML/d ($ 6.5 Mill)*	*7 ML/d ($ 6.5 Mill)*
- **Upgrade 2015**	+ 7 ML/d ($13 Mill)	+ 7 ML/d ($13 Mill)	*Not Required*	*+ 7 ML/d ($13 Mill)*	*+ 7 ML/d ($13 Mill)*
- **Upgrade 2019**	+ 6 ML/d ($15 Mill)	+ 6 ML/d ($15 Mill)	*Not Required*	*+ 6 ML/d ($15 Mill)*	*+ 6 ML/d ($15 Mill)*
- **Upgrade 2025**	-	-	-	↓	↓
- **Max Upgrade**	-	-	-	*+40 ML/d (~$ 70 Mill)*	*+40 ML/d (~$ 70 Mill)*
Storage :	***Basin(s)***	***Aquifer***	***Aquifer***	***Basin(s)***	***Aquifer***
Maximum Volume Used:	400 ML ($ 10 Mill)	400 ML ($ 3 Mill)	7 000 ML ($ 19 Mill)	*6 000 ML (~$ 120 Mill)[6]*	*6 000+ ML [7] (~$ 40 Mill[1])*
Class A Water Cost:					
Full Cost Recovery Mode (NPV)I[9]	$ 1605 /ML	$ 1475 /ML	$ 1420 /ML	*$ 1369 /ML*	*$ 1289/ ML*

Table Footnotes:

1. Assuming 20 relatively large bores and pump systems.
2. Sewage inflow assumed to be ~60 ML/d in 2030 for the purposes of this exercise to set a theoretical ultimate goal.
3. Actual storage lifetime may be longer despite cover and lining renewal
4. Includes assumed system losses and indicates % of available water provided to end users. Indicates 100% of available water recycled.
5. Surface storage and larger borefield cost estimates include no land aquisition costs. Additional land not required for smaller storage, but for very large storage this would be required adding considerable cost.
6. Effective storage is large enough but access limited by bore field capacity, also requires treatment upgrade to 60 ML/d.
7. Assuming $ 20 000 per ML
8. Total storage available in this ultimate scheme is far beyond 6000 ML, if used to bank water beyond the annual cycle.
9. Assuming required Class C demand and required infrastructure increase also.
10. More recent water pricing work costs these figures slightly higher, but here we consider the original business case, and use the model template produced in early 2009. Figures are best viewed as relative for comparison.
11. Class A Plant component of the total Recycled water plant for Black Rock. I.e. some water produced will go to Class C Mix but not considered here. Timing of upgrades is dependent on demand from users still to be confirmed.
12. Cost Estimtes generated for this comparison are very preliminary and whilst produce a plausable comparison, should be used with caution.
13. All costs are in australian dollars (AUD)
14. Cost estimates presetned here are considered to be ± 30 %.

Table 1: Cost Comparison Table for all Opportunities Considered

It is important to note the absence of any additional land acquisition costs or major environmental works that may be required for the larger schemes. The actual location of a potential large surface storage is very uncertain and therefore costs are difficult to apply to this comparison, however this aspect may be better catered for in a more comprehensive social and environmental assessment.

The idea of a surface storage of this size is difficult for most to comprehend, and as such the information considered here highlights the idea that such an option, and in turn that large scale scheme, would not be a viable pursuit.

CONCLUSIONS

These options and comparison show that the integration of an ASR system can contribute cost savings, despite the relative increase in operational effort. Logically the cost effectiveness improves with the scale of the scheme, and at a much greater rate than the surface storage scenario.

The range of costs sits within that presented by Dillon et al (2010), when discussing a variety of ASR schemes in operation in Australia. However, the results presented here consider a "whole of scheme" business case built around supply to a dual pipe development may not be directly comparable.

It is useful to compare these costs with potable water supply pricing in this area. Current prices are now set at approximately \$ 1.8 / kL (\$ 1 800 AUD / ML), with pricing projections indicating a rise to and/or exceed \$ 3.0 / kL (\$ 3 000 AUD / ML) within the next 10 years. Undoubtedly this will help make a recycling/ASR project of this type even more attractive.

Whilst this paper focuses on rationalising the economic side of pursuing this ASR component, it is important to acknowledge the significant social and environmental benefits that will result in comparison to a large surface storage. So, even with marginal cost-benefit, the scheme will be preferred.

With this approach justified, the initial stages of an ASR scheme will set a path towards a maximum recycling goal in the longer term that is much more realistic.

ACKNOWLEDGEMENTS

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Managed Aquifer Recharge through a penetrated river bed in Shandong, China

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Abstract: The Huangshui River, located in the Northeast of Shandong Province, China, shows an outstanding example for water conflicts arising from piecemeal action as well as rapid growth of population, industry, and agriculture. To tackle these problems, a joint Chinese-German project was launched in 2008. An important part of the project is the optimization of the existing MAR facilities along this river. Here, eight dams reduce the outflow into the Bohai Sea. The water which is stored in front of these dams infiltrates into the underlying aquifers. This process is accelerated by the construction of 2500 infiltration wells. The main objective of these facilities is to reduce the sea water intrusion caused by a massive overexploitation of groundwater in this region. Furthermore, it is intended to use the infiltrated water for agricultural purposes. The area around the MAR facilities serves therefore as a groundwater reservoir which stores water during the three wettest months of the year. To prevent groundwater outflow into the sea and increase the storage capacity of the reservoir, additionally an underground dam of about six km length has been constructed. Within the scope of the project a detailed coupled surface and groundwater model has been developed which describes the reservoir and all relevant physical processes as a part of a complete and interactive system. It focuses on the interaction of runoff, recharge, surface and groundwater, both from the quality and the quantity point of view. The model includes the complete basin of the Huangshui River in order to be able to analyze the selected optimization strategy of the reservoir as well as the effects of other measures on the water balance of the region. The model is integrated in a GIS supported DSS which takes into account different stakeholders as well as future social and economical developments. At the conference it will be focused on the results of the models and the way these results can attribute to find a profound solution to the problems in the region.

Keywords: IWRM, China, intrusion, DSS, groundwater

Introduction

The Shandong Province, especially the Huangshui River Basin and Longkou County (Figure 1), is an out-standing example for water conflicts arising from piece meal action as well as fast growing population, industry and agriculture. In the coastal catchments of the Shandong province the water scarcity is even increased due to salt water intrusion, reducing the usability of available water resources. Furthermore, social status and income of farmers in the Shandong province is significantly below a level that would allow them to keep up with the technology development in irrigation and farming. The socio-economic problems can only be tackled by truly integrated water management approaches. In Figure 2 the salt water intrusion development in Longkou City is shown. In the 90's already many measures against the saltwater intrusion have been implemented. In 1995 for example, an underground dam was finished in the downstream part of the Huangshui River, 1.2 km from the seaside, with a total length of approximately 6 km (Liu et al., 2003). The average depth of the dam is 26.7 m. On the one hand, the dam prevents the ground water from flowing into the sea. On the other hand, it can stop the sea water intrusion. Even after this project was finished, the area affected by salt water intrusion continued to increase. It is part of the project presented here to investigate the influence of this dam and to analyze whether its effectiveness can be increased.

The general objective of this project however is to bring together German expertise in water management and newer developments in context with the European Water Framework Directive with the research efforts in the coastal area of Shandong Province to relieve the desperate water scarcity situation. The total water consumption within the project area (ca. 1560 km²) amounts in average (years 2000 until 2007) to about 207 million m³/a. It is composed by approximately: agriculture (irrigation) 73%, domestic 10%, industry 16% and environment 1%. Although water saving techniques in irrigation after many years of research and international cooperation are introduced, agricultural water demand still increases. With a usable runoff of about 190 million m^3, the water demand of 207 million m³/a clearly exceeds the water resources, in average by about 10%.

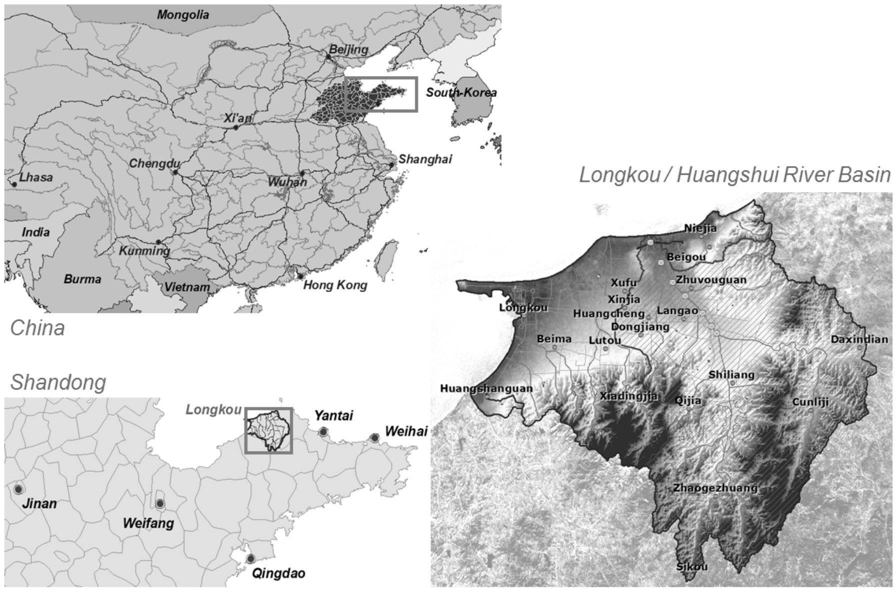

Figure 1: Study area Huangshui River Basin / Longkou

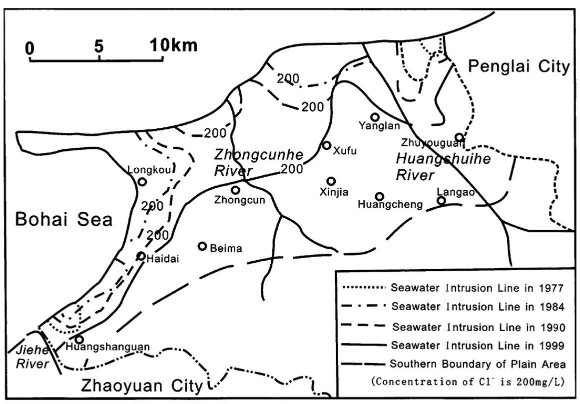

Figure 2: Development of seawater intrusion in Longkou City (Wu et al., 2008)

This problem is even more severe considering the monthly and annual distribution of water resources and water demand (Kutzner et al., 2006). Also the spatial distribution of the extractions have a great impact. After further assessment of the present situation and the many abatement measures tried with, especially to stop salt water intrusion, it was found that there is a tremendous potential to improve the situation by appropriate integrated water resources management (IWRM). If a solution for this extremely complicated problem can be developed and implemented the methods and technologies used may be generalized and applied to the whole Shandong Province or even to other parts in the world. The project presented in this paper is funded by the German Ministry of Education and Science BMBF and the Chinese Ministry of Science and Technology MOST. On the German side seven project partners - both private and public companies or institutes - and on the Chinese side five partners - only public institutes - are involved.

IWRM method

The steps and components applied in the IWRM project presented here basically can be listed as follows:

- Setup of a so-called **measures catalogue**. From the analyses of the present situation, all potential measures for the project area and the considered water-related issues, especially in the field of water saving, ground water recharge, water recycling, structural measures against salt water intrusion and institutional measures, are collected, sorted and combined in a logical and functional measures catalogue. Every measure will be appraised according to a number of criteria, relevant for determining a sustainable integrated solution. Sustainability is considered in their economic, social and environmental aspects.

- A Decision Support System (**DSS1**) to find the most promising combinations (**measures scenarios**) of these potential measures, only partially taking into account their geographical location. For instance, for a very specific irrigation technique criteria like construction and operation costs, durability, potential water saving ability, required space, potential risks, employment opportunities, necessary training, effects on water quality and groundwater enrichment, etc., will be estimated by a team of German and Chinese experts. This will result in a fuzzy estimate of performance of this measure for each relevant criterion. Using algorithms based on the *Fuzzy - Analytic Hierarchy Process* (F-AHP) and Dynamic Programming, a finite amount of optimal combinations of measures will be generated based on the aforementioned fuzzy expert opinions. A complex dataset is thus decomposed into smaller constituent elements between which pair-wise comparison is elicited, enabling well-founded expert-assessments of performance for each measure according to each criterion (Monninkhoff et al., 2009).
- A GIS-based information system, which incorporates:
  - Information to all selected measures scenarios and the possibility to specify each measure according its location, intensity or size.
  - A number of hydrological models, which together describe all relevant hydrological processes and which are coupled on the fly or by automatic data exchange. These models will simulate runoff, infiltration, evaporation, overland flow, groundwater recharge, surface water flow, groundwater flow.
  - Tools which automatically implement the measures within the geographical setup of each model (for example weirs, reservoirs or subsurface dams). These tools will also import the relevant model results into the GIS database.
  - Criteria and multi-criteria evaluation routines to analyze the simulated measures scenario (**DSS2**). The basic criteria will be similar to the ones of DSS1, but they will be separated in a number of components, which each can be evaluated by automatic routines using the results of the model together with a number of relation tables implemented within the database.
  - Tools which visualize (GIS maps, tables and time series) the results of these evaluations and enable the user to compare these with analyses of previous analyzed measure scenarios.
  - Also, a multi-criteria visualization based decision and negotiation support technique, named the *Reasonable Goal Method / Interactive Decision Map (RGM/IDM)* technique (Lotov et al., 2004), will be implemented within the GIS environment. This tool provides excellent options to explore new ways to balance competing interests (Monninkhoff et al, 2009).
  - The GIS system cannot only be used to analyze the influence of several measures scenarios for a given period using observed meteorological conditions, it will also support analyses to the influence of different **development scenarios** (for example climate change) on a single measures scenario.
  - Documentation routines which summarize the measures which were analyzed and give a clear overview of the effectiveness of these measures.

In the Figure 3 the basic concept of the GIS-based and model-supported system (DSS2) is displayed. The models which are mentioned in this figure will be explained in the next section.

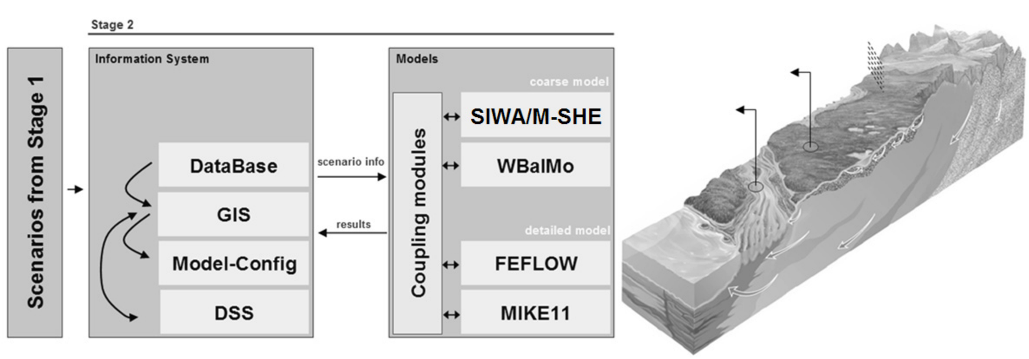

Figure 3: Concept of the GIS-based and model supported DSS system (Winter et al., 1998)

Hydrological models

The basic model concept includes two different model approaches; a relatively coarse and a detailed model. The **coarse model** is used to verify the general feasibility of the water usage proposed in the selected measures scenario. For this, the groundwater flow processes are of secondary importance. This model consists of the simulation packages SIWA (Wirsing et al., 2010) and WBalMo (Kaltofen et al., 2008). Furthermore, a MIKESHE model (Sahoo et al., 2006) has been set up to verify the results of SIWA and to support mass transport modelling also for the unsaturated zone. SIWA is a 1-D empirical soil water model, based on landuse, soil, slope and groundwater depth data. The model provides a fast and robust way to calculate spatially distributed monthly groundwater recharge and direct runoff rates. MIKESHE dynamically couples all major hydrologic flow processes, including 2-D overland flow, 1-D channel flow, 3-D saturated zone flow, 1-D (Richard's based) unsaturated zone flow, snowmelt and evapotranspiration. Overland flow is calculated by a diffusion wave approximation using a finite difference scheme. In contrast to SIWA, MIKESHE has the advantage that also mass transport processes can be modeled in the soil water. On the other hand, if no relevant pollution is introduced in the model area by way of the root zone, SIWA provides a more convenient and faster method to simulate the land surface processes. As the mayor problem within this project is related to salt water intrusion, it was decided to renounce the modeling of mass transport in the soil water zone and to use SIWA instead of MIKE-SHE. WBalMo (Water Balance Model) is an interactive simulation system for river basin management. WBalMo has been used to identify management guidelines for river basins, design reservoir systems and their operating policies, and to perform environmental-impact studies for development projects. By recording of relevant system characteristics during the simulation, probability estimates can be provided for water deficits, maintaining minimum runoff or reservoir levels. If the resulting deficits for all water users of a coupled WBalMo and SIWA simulation are within the deficit limits set in the GIS environment, the suggested water usage from the evaluated measures scenario can be accepted and relevant results can automatically be transferred to the detailed model (groundwater recharge, extraction rates and positions as well as direct runoff to the streams by overland flow).

The **detailed model** also consists of two software packages; FEFLOW (Diersch and Kolditz, 2002) and MIKE11 (Szylkarski, 2002). FEFLOW provides an advanced 3D graphically based modeling environment for performing complex groundwater flow, contaminant transport, and heat transport modeling. Both saturated and unsaturated flow regimes can be modeled. It uses a Galerkin-based finite element numerical analysis approach with a selection of different numerical solvers and tools for controlling and optimizing the solution process. Most important though for this project is the possibility to take into account density dependent flow processes. MIKE11 is a powerful numerical surface water model developed by Danish Hydraulic Institute (DHI) to simulate 1D flow problems. The software offers to simulate unsteady flow in river networks as well as looped networks using an implicit finite difference scheme. MIKE11 and FEFLOW can be coupled using the module IfmMIKE11 (Monninkhoff and Schätzl, 2008). The module has been extended for this project in order to be able to take into account also mass exchange processes between the ground- and surface water bodies. The detailed model will give information about the impact of the proposed measures on the groundwater levels, the salinity of the rivers and the groundwater, as well as the effectiveness of the artificial recharge along the rivers.

GIS environment

The GIS system will be based on ESRI ArcGIS® and for the pilot application, its database will be limited to a personal geodatabase. The setup implements a four level program architecture. First of all, different roles have been introduced (*role level*); administrator, analyzer and development scenario handler. An administrator is allowed to set overall settings to the system, for example the locations of the model components, the base databases and the base project. The administrator can also define a updated present state within the system. A development scenario handler can define new **development scenarios** for, among others, climate, demographic or economic changes. The main role however is the analyzer role, by which the proposed measures can be specified and its effects can be analyzed.

A project represents the second level in the program architecture (*project level*). A analyzer needs to open or create a project before he or she can start to analyze. Within each project a measures scenario as well as a development scenario (or present state) has to be selected.

A project also includes the definition of the type of simulation which should be analyzed (*simulation type level*). It is intended to support two options; (1) to simulate the same period which was used to calibrate the model (2000-2007), including all observed time series (climate, extractions, etc.) and (2)

to start a prognosis simulation from the year 2007, using generalized time series. In case the second option has been selected, a number of additional options will be available in which the generalization of the time series can be specified.
After creating or opening a project, the fourth level of the program architecture becomes visible. This *task level* is referring to a number of tasks which the user has to complete or is allowed to perform (see Figure 4). These tasks include; Project management, Measures, Simulation, Model Results Visualization, Evaluation and Comparison. Every task has its own dialog and the dialog for the task Measures is dependent on the measures available in the selected measures scenario. For example, in case the measure "apply micro spray irrigation for fruit trees" is available in the selected measures scenario, then the dialog will support the user to select the areas in which fruit trees are available and a micro spray irrigation is likely to be applied. A proposal for the implemented dialog which supports this is also shown in Figure 4.

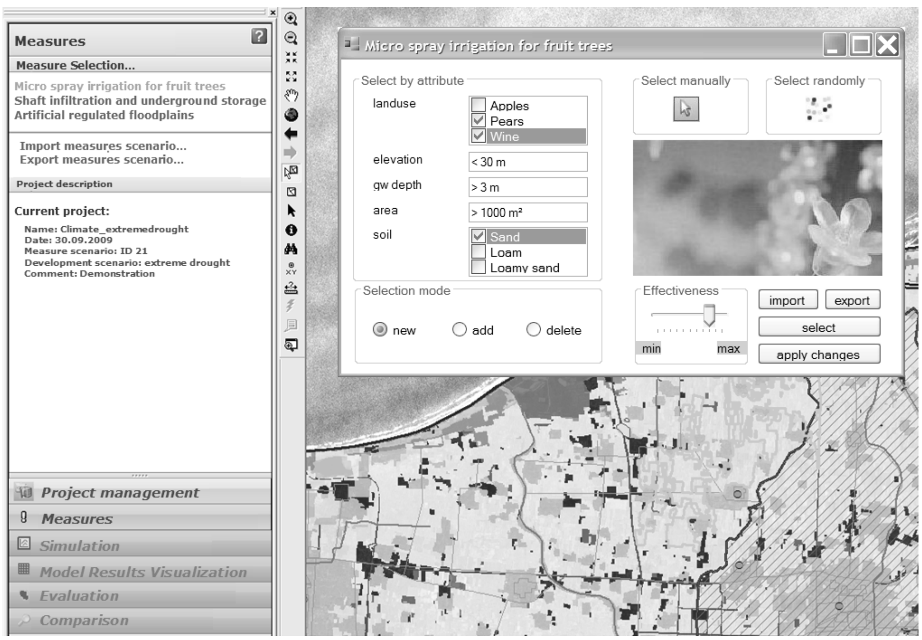

Figure 4: Design for the dialog of the task Measures.

In the task Simulation the models (coarse or detailed) can be started and their results can be imported. The integration of the defined measures within the models setup will be done automatically. Within this task also the deficits generated by WBalMo can be accepted or rejected. The task Model Results Visualization will give the user the opportunity to visualize, among others, groundwater levels, salinity values or direct runoff at a certain time, to plot time series at a single point (extractions, demand, deficits, etc.), before these results will be evaluated by the DSS routines (Evaluation task). In particular the visualization of the deficits should be implemented in a way, that it contributes to a reliable decision, whether the proposed extractions can be accepted or not. Special database tables will be implemented to support this. These deficit tables and additional spatial distributions should be used to evaluate the results of the coarse model and give the user an idea of the possible outcome of the detailed model and the DSS2 evaluation. The intended workflow of the system therefore involves a loop between the tasks Measures, Simulation (coarse model) and Model Results Visualization until the results of the coarse model are accepted by the user and the detailed model can be started. In the task Comparison the evaluation can be compared with other previously evaluated results. Here, the leveled architecture can be used to support the user in selecting similar projects to compare the present project with.

Model state and results

The setup of the five models mentioned above has been completed. The MAR facilities within the downstream part of Huangshui River are represented within the coupled FEFLOW/MIKE11 model. The FEFLOW model is discretisized by a finite element mesh consisting of about 34,500 elements (per layer). The integration of all main rivers has been taken into account in the mesh design. Additionally, a mesh refinement around the most important lakes and reservoirs has been carried out, allowing a accurate simulation of the interaction between these water bodies and the surrounding groundwater, especially in the area of the MAR facilities. In this area 2500 pieces of 2 m diameter recharge wells penetrate the semi-permeable river bed to enlarge the infiltration rates. Furthermore, eight dams at the downstream end of eight separate infiltration sections guarantee that there will be enough water within the river to be infiltrated. These dams are integrated in the MIKE11 model. The recharge wells are represented by enlarged leakage coefficients within FEFLOW at the boundary nodes at those locations. In this way it would also be possible to represent a gradual clocking of the wells by applying a time varying leakage coefficient. This, however, has not been implemented yet. The top of the groundwater model was defined by the DTM; it ranges from sea level close to the coastline to about 500 masl in the South. The vertical extent of the model corresponds to quaternary deposits. The thickness of the groundwater model alternates between 90 m in the North-West and 5 m in the southern and eastern parts of the model. The latter area represents the alluvial deposits along the upstream parts of the rivers. The number of model layers and their spatial extensions were derived from the 1:50,000 geological map, two geological cross sections as well as twelve detailed boreholes. In total twelve layers have been implemented to represent the complex geological structures. The top layer represents among others the semi-permeable deposits of loam (partly sandy), silt or clay soils on the surface. An overview of the model and the complex geological structure in the area is given is Figure 5.

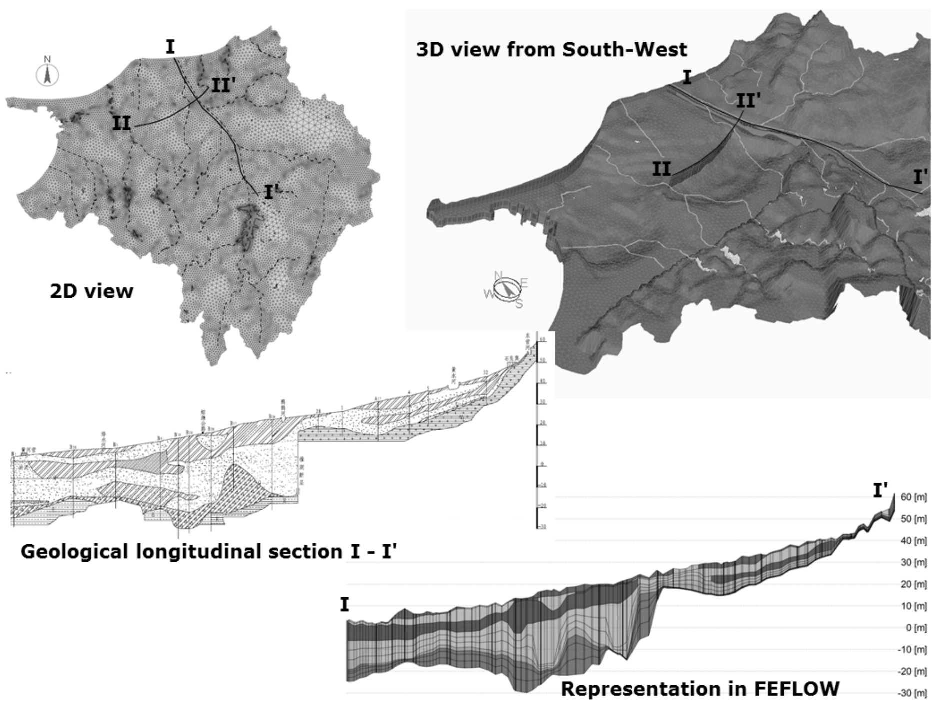

Figure 5: FEFLOW mesh and the representation of the geological layers

The monthly groundwater recharge is provided by the SIWA model. For the years 2000 – 2007 an area weighted average of 53 mm/a has been calculated. The total average runoff amounts 71 mm/a. An overlay of the SIWA recharge values upon the FEFLOW elements is performed automatically. For the northwestern boundary, which proceeds along the Bohai Sea coastline, first kind boundary conditions (Dirichlet) with a fixed head of 0 m (average sea level) are used. The main rivers are represented by third kind boundary conditions (Cauchy). For these rivers a coupling to MIKE11 has been implemented, so that the water levels in the rivers do not have to be specified within FEFLOW. The coupling uses new features, by which the exchange rates between the surface and groundwater depend on the actual wetted perimeter within the river. From the MIKE11 results, the wetted perimeter is updated for every time step automatically. The integration of the runoff values calculated by SIWA within the MIKE11 model has not been completed yet. The initial groundwater levels could be interpolated from about thirty observation wells. These observed levels also build the main verification parameter of the model. Pursuant to the available observed groundwater levels the calibration period will cover the years 2000 until 2007. Spatially distributed extraction rates have been calculated from the data delivered by the Chinese partners. From these data a district dependent relationship to the available landuse classes could be derived, which can be used to adapt the extractions according the measures being specified. An example of a calibrated groundwater observation gauge is shown in the Figure 6.

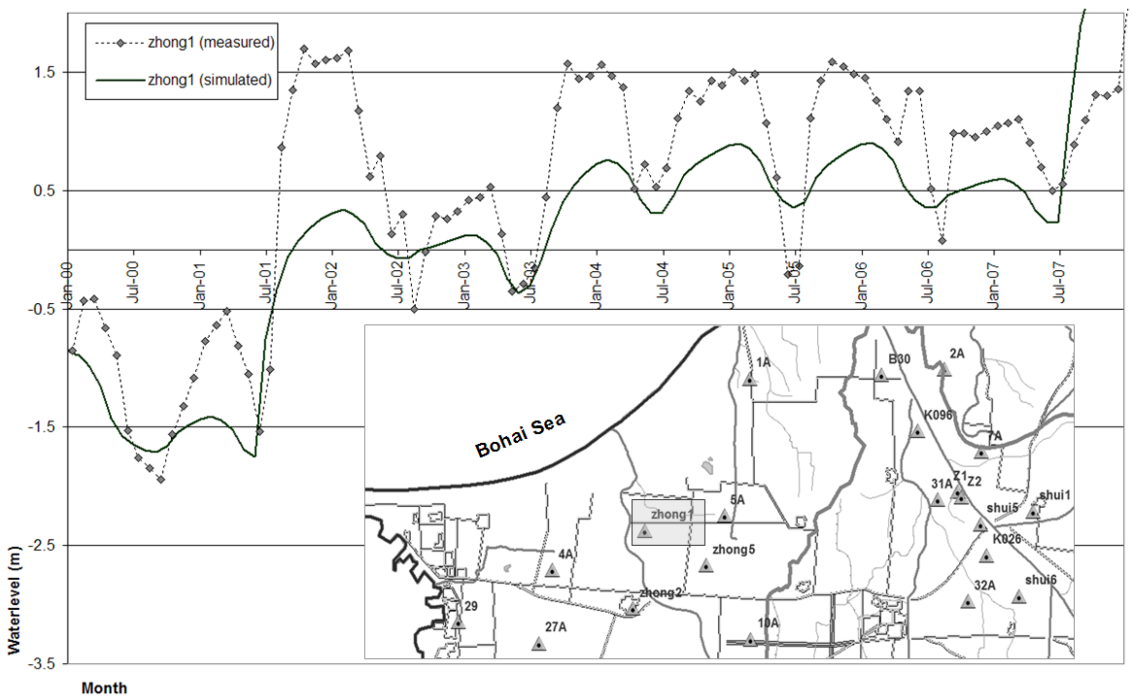

Figure 6: Observed and calculated groundwater levels at groundwater gauge Zhong1

Outlook

The final calibration of the hydrological models, in particular a detail analyses of the MAR facilities in FEFLOW, will be finished until the end of 2010. The GIS concept presented in this paper has already been partly implemented. It is planned also to finish it until the end of 2010. It was shown that the concept is based on a relatively flexible architecture. This leveled architecture already has been successfully tested in other projects (Dietrich et al., 2009). It guarantees that different measure scenarios and criteria can easily be integrated, that additional tasks can be implemented or tasks can be exchanged and that hydrological models can be altered or exchanged. In this way, also for other parts of the world, having similar or related problems, the system could easily be adapted and support politicians as well as engineers in making well-founded decisions.

Acknowledgements

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Artificial Recharge In Damascus Basin, Almazraa Drinking Water Field

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ABSTRACT:

Damascus the capital of Syria suffers during the summer period from an enormous lack of water. The water supply during this time is managed from well fields which are localized in the city and surrounding suburbs. The water is pumped from Quartanary porous unconfined aquifer which forms the youngest sediments of the Damascus basin located in the south west of Syria. During the wet winter period the drinking water is drawn from two big karst springs Barada and Al Figeh. In extreme years in flooding time the springs discharge 50-60 million m^3 of water without getting any benefits from this huge amount. The water authority of Damascus (DAWSSA) tries to manage the supply with simple MAR techniques. Since 2000 they use little part of the flood water (52500 m^3 to 1817195 m^3) for artificial recharge of the aquifer. The quality of the water is the same as for the drinking water supply as it is drawn directly from the springs (300µS/cm). The hydrographs of the observation wells in the basin demonstrate no additional rise of the water table during artificial recharge. However, artificial recharge is the only alternative at the moment bigger collector for the flood water will open the chance to stimulate the demand during the dry period.

KEYWORDS: Artificial recharge AR, Damascus city, Groundwater, Hydrograph.

INTRODUCTION

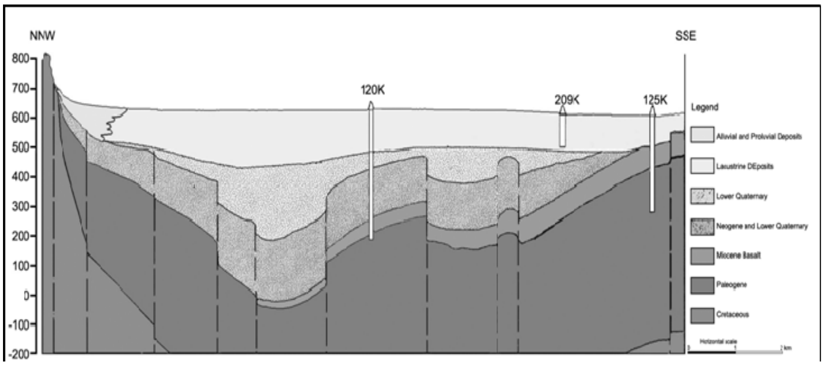
Damascus city faces now one of the most difficult challenges to supply 3,5 million inhabitants living in Damascus and Damascus country side with the necessary amount of water as well as agriculture and industry activities which are mainly centralized in the capital. The problem increases with the rapid population growth in the next decades. The main water resources for Damascus city are Al-Figeh, one of the largest karst springs in the world, and Barada springs. Well fields, exclusively used for drinking water abstraction supply in addition during the dry period from May to November the city.
The dirnking water demand for Damascus is estimated in 2010 with 435000m^3/day (150L/day/capita) and in summer when the water authority follows the rationing policy which differs from year to year according to annual rainfall rates, water shortage increases to reach its maximum value in August and September. Under these conditions other alternatives should be developed to satisfy the demand of the city taking into account the very good quality of the water which flows in Barada River during the flood period.

GENERAL BACKGROUND:

There are two rivers flowing in Damascus plain Barada and Alawaj. Barada river which is the main surface water resource originates in Anti-Lebanon mountains(1500m Asl)in Al-Sheir Mansour fed by Barada spring(average discharge 3.12m^3/sec)and terminates in Oteibeh lake. The lake has dried out in the last decades and the river reached the area only a few times. Al-Awaj River originates in the eastern part of Haramoun mountains (2814m Asl) in Al-shekh mountain. Many small springs are feeding the river which flows into Damascus plain and terminates in Hijaneh lake which has also dried out.

GEOLOGY AND TECTONICS:

Damascus city is located in the north western part of Damascus plain which is one of the 11 sub basins which form Damascus basin. It is a sedimentary depression of Quaternary and Neogene age, surrounded by basalt outcrops at the southern and east southern margins. It has an asymmetric elliptical form with axis oriented in north east -south west direction. The sediments differ in Damascus depression from poorly sorted gravels and conglomerates with clayey, sandy cement near Qassioun foothills, to fluvial-proluvial deposits which are common in west and central part of the plain. The eastern and southern parts and in Otaiba and Alhijana lakes are characterized with lacustrine deposits which are mainly of marls and clay with sandy lenses and freshwater limestone. (Hobler,M.& Rajab,R.2002). Some faults strike Damascus depression. Their directions are north east-south west, and some have a north west-south east direction. Damascus thrust fault separates the mountain range from the plain. Faults play an important role in changing the thicknesses of Quaternary and Neogene deposits fig (1).The total thickness of Quaternary reaches in Damascus plain about 400m (Alammareen 2000).



Fig(1): Geological section in Damascus plain (Hobler M. & Rajab R.,2002)

HYDROGEOLOGY:

Upper and recent Quaternary represented by alluvial-proluvial deposits a-ap(Q_{III-IV}) and lacustrine deposits l-la(Q_{III-IV}) form the upper aquifer followed by Neogene basalt (mostly at the southern and east southern margins of the plain) and underneath Cretaceous consisting of limestone and gravels in the mountain areas. There might be an important Jurassic aquifer in greater depth which is not explored jet. Hydraulic connection occurs between these aquifers probably through faults and fractures.

The aquifer of alluvial-proluvial a-ap(Q_{III-IV}) from upper and recent Quaternary is the most important one in the case study because the area of the drinking water fields is located within this aquifer. Its upper part consists of sand and gravel, silt and clay and the thickness of this formation differs widely and can reach 100m. The aquifer of lacustrine deposits l-la(Q_{III-IV}) from upper and recent Quaternary consists of lacustrine clayey marly deposits with low sorted gravels and sands (Hobler,M. & Rajab,R. 2002) and this aquifer prevails in the area of Oteibeh and Hijeneh lakes. The total thickness is about 200m and limnic limestone is very common in these deposits. Due to little rainfall groundwater recharge in Damascus plain takes place in the west of Quasioum mountain and some surface water, irrigation, and rainfall infiltrate in aereas which are not sealed. More important is the recharge through Damascus fault system from deeper aquifers. This system is not investigated sufficiently and no real details are existing about this recharge. Groundwater depth in Almazraa was in the time from 1995-1998 between 8-10m and groundwater level took a minor ascending trend where the minimum depth was in 1995 (9.32m), in 1996 (8.25m) and in 1997 (6.95m). The period from 1999-2001 was very dry with precipitation between 67.8mm in 1998-1999 and 119mm in 2000-2001. This can be clearly seen on the hydrograph because of the lowest values of groundwater level which are noticed between1998-2001.In 2002 the rainfall was higher (143.3mm) than the years before and the end of this dry period. Groundwater depth decreased to about 14m in May. From 2003 to 2005 the depth was shallow between 5-10m and this corresponds with the good rate of precipitation in these years. From 2006-2008 water depth increased again steeply to reach about 20m in April 2008.

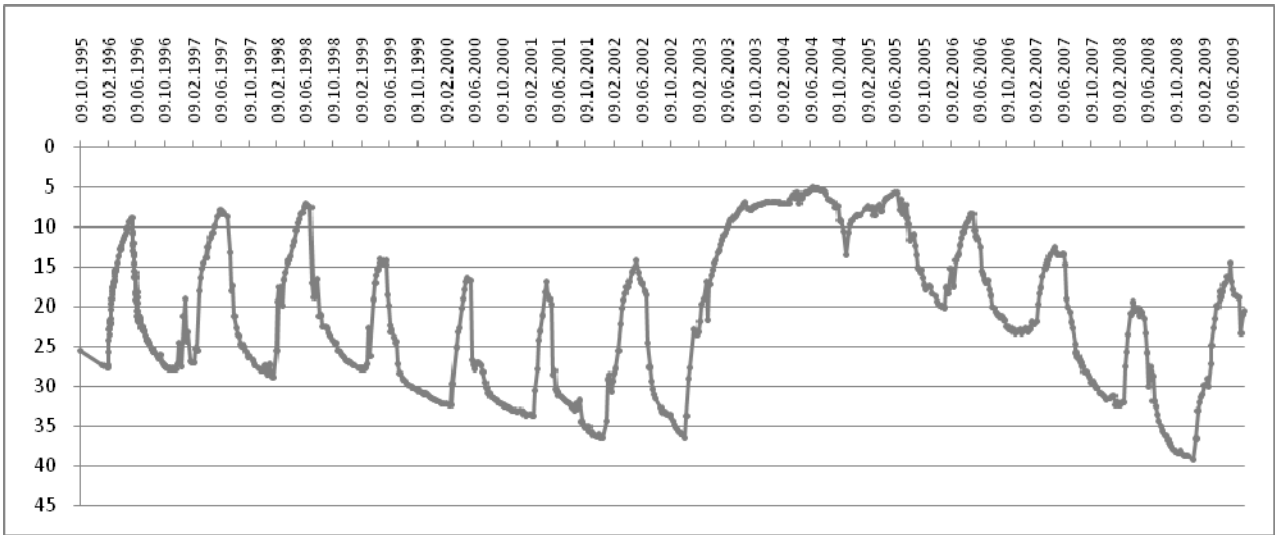


Fig (2): Changes of groundwater depth in Alamazraa drinking water field

Groundwater direction was in the nineties west-east reflecting the topography and flow direction of Barada river. In 2008 the direction changed to northwest-southeast (Fig. 3).

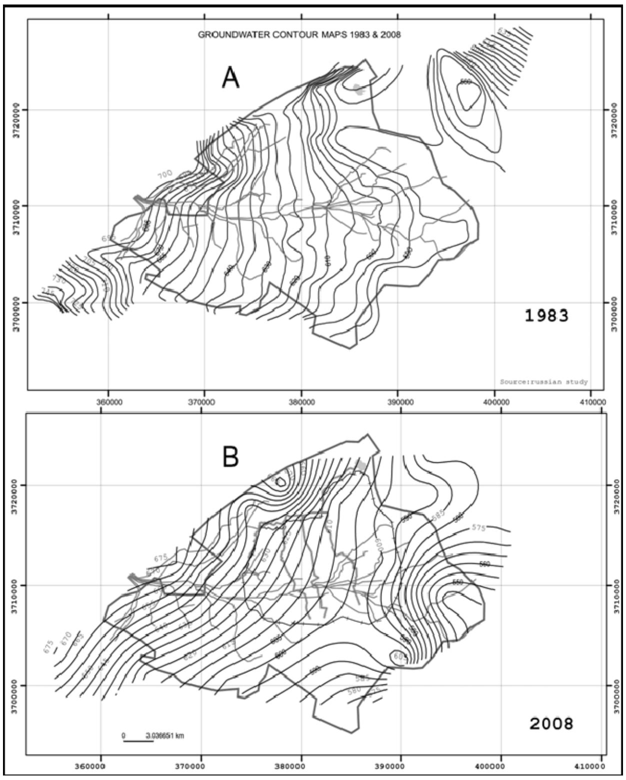


Fig (3): Change of groundwater direction in Damascus plain

This can be explained through the properties of the aquifer. The a-ap(Q_{III}-Q_{IV}) aquifer is not built up homogeneous with changing of fine and coarse sediments but more important is the change of the irrigation area with effluent waters. The treatment water plant in Adra (north east of Damascus) distributes treated water to large new irrigated fields in the eastern and north-eastern part of Damascus plain. Due to the recharge in this areas a higher groundwater level disturbs the normal flow system can be registered. Over exploitation in the plain especially in the eastern and south eastern parts where most of agricultural activities take place leads to lower groundwater level. Thus the natural groundwater flow direction is disturbed and a slowly change took place.

DAMASCUS CITY WATER SUPPLY:

The main sources of water supply in Damascus city are Al-Figeh karst spring and Barada karst spring. Both have a very high α and the run off turns fast down. The two springs are located in North West of Damascus. The discharge of both springs does not meet the water demand of the city during the dry season. Therefore Damascus water authority abstracts in addition water from the drinking water well fields to satisfy the needs of inhabitants during the dry period. The well fileds are located directly within the city. From about 14 wells fields and some other reserve wells fig (4) the drinking water is produced. The concentration will be on Almazraa water well field where the first trial of artificial recharge was executed. It is the biggest one of these fields and contains about 23 wells whose depths range between 80-125m.

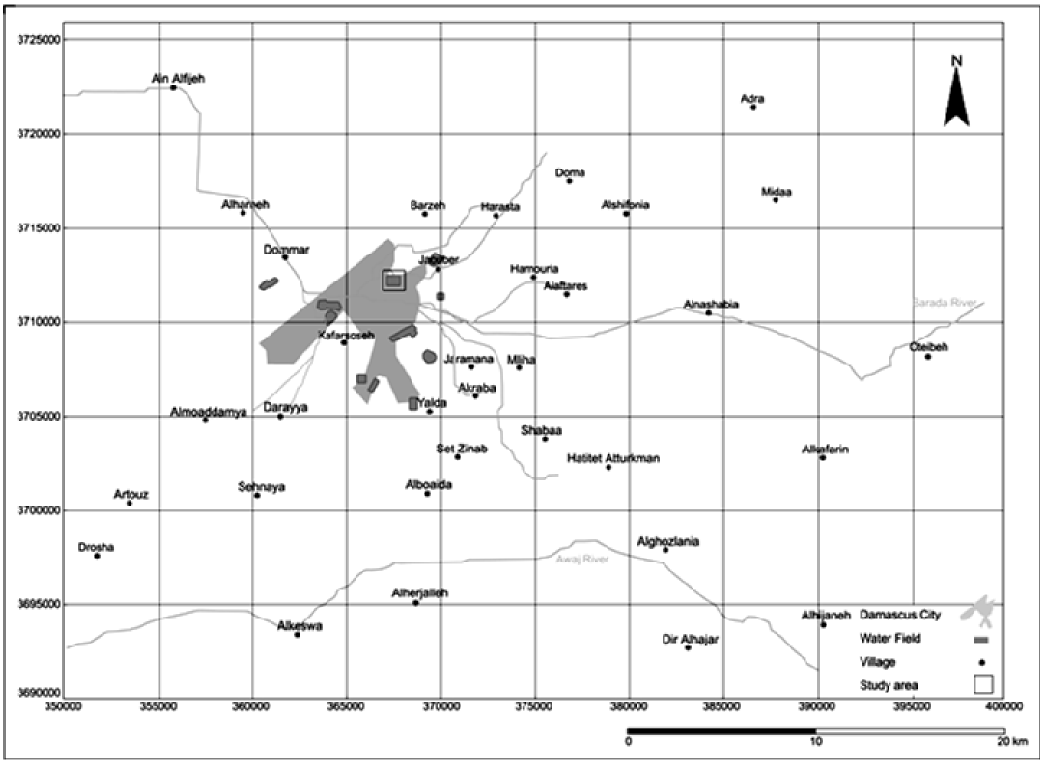


Fig (4): Drinking water fields in Damascus city (Al-Ammareen 2000)

ARTFCIAL RECHARGE IN ALMAZRAA AND CURRENT CONDITIONS:

DAWSSA started from 1997 during the raining season with high run off an artificial recharge (AR) trial in Almazraa to allocate a portion of Al-Figeh water to supply Damascus city in dry seasons. The quality of the spring water is very good. The electric conductivity (EC) is 300-350 µS/cm and no bacteria can be detected. Two wells (I, II in fig. 5) have been drilled for this purpose and water has been drawn from the fresh water reservoir through one of the drinking water supply pipes directly to the injection wells. The calibration for the behavior of the recharge was calculated depending on this trial in 1997. Figure 5 shows a locations map of observation and injection wells.

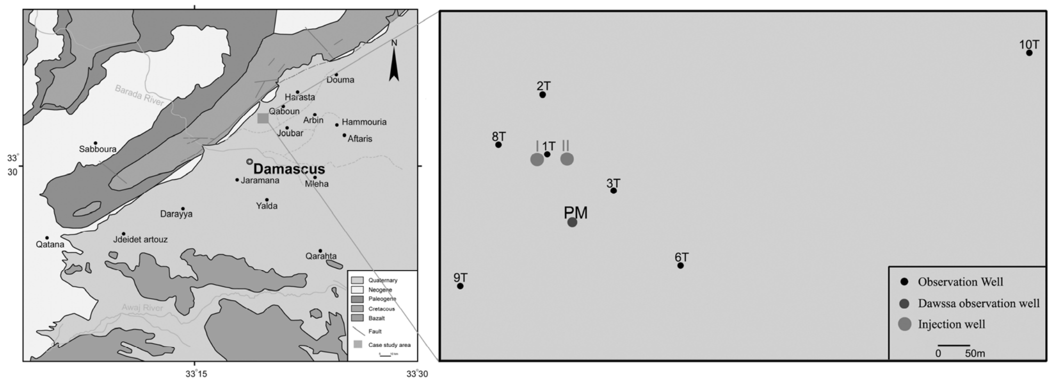


Fig (5): Observation wells network in the injection site

The annual precipitation in 1997 was about 154mm. 7 wells were used to detect the impact of water injection on the groundwater level. 258000m^3 water was injected and well PM is used to register groundwater depth continual. The experiment has been started April 5th 1997 and lasted 66 days. The two injection wells have a diameter of 3m and a depth of 30m. In the first 42 days in well I 4000m^3/day were recharged and in the next 21 days 2000 m^3/day were recharged in both, well I, II. For the last 3 days 2000 m^3/day were injected in well II. Groundwater level changes were observed in the well field. The results and the changes of groundwater level are shown in table (1).

Table (1): observation wells and groundwater level changes in the AR 1997

Well ID	Distance from injection well (m)	Water level t=0 (m Asl)	Water level 6.4.1997 (m Asl)	Water level 30.4.1997 (m Asl)	Water level 30.5.1997 (m Asl)
2T	118	684.7	685.25	687.7	687.7
8T	75	684.4	n.m	687.3	687.3
1T	22.5	684.2	683.52	688.3	688.3
I	0	683.8	n.m	692.5	692.5
II	50	683.6	684.45	687	687.1
3T	144	683	683.15	685.8	685.8
9T	265	683	n.m	685.4	685.4
6T	315	680.7	n.m	683	683
10T	885	681.7	n.m	683.9	684

The following parameters R, k, v, T were calculated depending on the results of this trial:

$$KD = \frac{Q}{2\pi(s1 - s2)} \ln\frac{r2}{r1}$$

$$KD = \frac{4000}{2\pi(2.9 - 2.2)} \ln\frac{885}{75} = 2244.62995 \ \ m^2/day$$

$$T = KD \ \Rightarrow K = \frac{T}{D} = \frac{2244.62995}{125} = 17.957 \frac{m}{day} = 2.079 * 10^{-4} \, m/s$$

$$V = KI = 17.957 * 0.0192 = 0.345 \ m/day$$

Where: K is hydraulic conductivity coefficient (m/day).

D is the aquifer thickness (m).

Q - discharge m^3/day.
s1 - groundwater drawdown in well 1 (m).
s2 - groundwater drawdown in well 2 (m).
r1 - distance between piezometer 1 and the injection well (m).
r2 - distance between piezometer 2 and the injection well (m).
Depending on Sichardt equation the effective radius R of AR can be calculated:

$$R = 3000 s \sqrt{K_f}$$

$$R = 3000 s \sqrt{K_f} = 3000 * 8.7 * \sqrt{2.079 * 10^{-4}} = 376.271 m$$

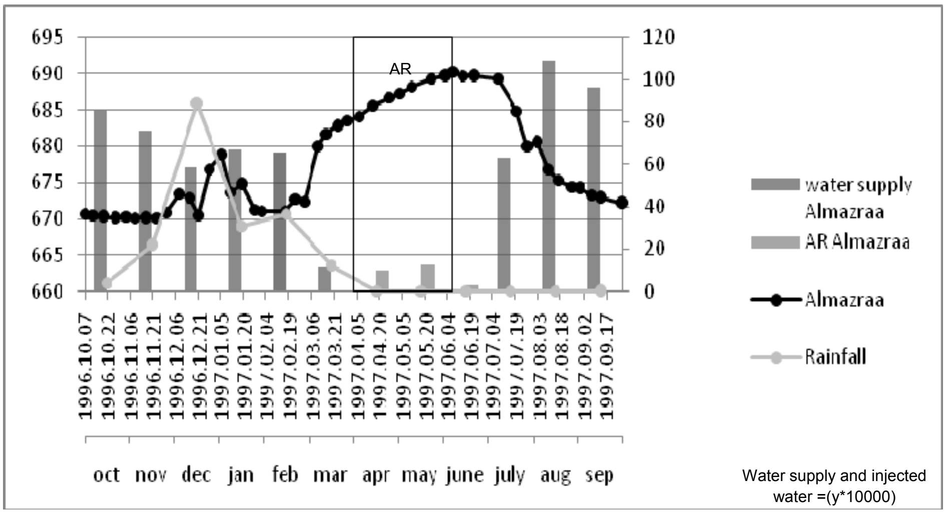
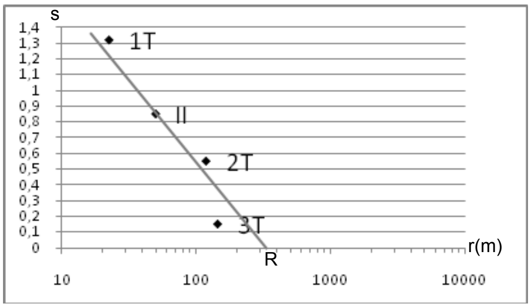


Fig (6): Hydrograph of well PM in 1996-1997

Figure 8 shows the change of the groundwater table in relation to the recharged water. The calculated R matches with the measured data of S in the distance r from the injection well (Fig. 7). The graphs show the direct relation of the artificial recharge and can be used for calculation of the upconing of the water table in the injection well as well as in the surrounding well field.



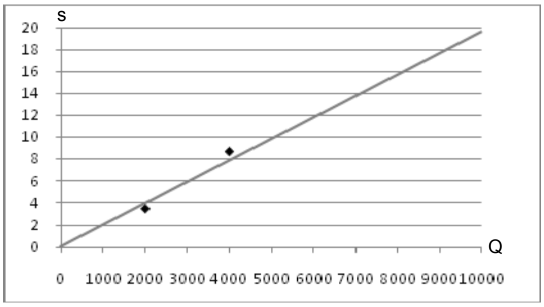


Fig (7): Calculation of R

Fig (8): Calculation of maximum S

The second trial was executed in 1998 in the same two wells. Artificial recharge was not possible in 1998-1999 because there was no surplus from the springs. Figure 9 shows the hydrograph of the observation well in 1998-1999. Precipitation in 1998-1999 was about 67mm. This year is chosen because of the contrast to others with low precipitation. Groundwater has been abstracted from the well field all over the year because of the very low discharge of Barada and Al-Figeh springs figure11. Groundwater level raised up about (13m) in the flood period and the maximum level has been recorded in May.

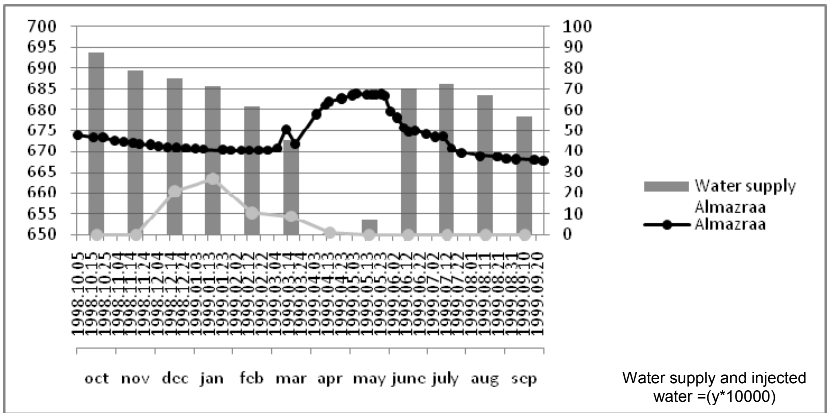


Fig (9): PM well hydrograph 1998-1999

Due to the high precipitation rate in 2002-2003 (290mm) and consequently the high discharge of Al-Figeh spring, DAWSSA carried out again AR in Almazraa. Figure 10 shows the impact of AR on groundwater level in the region.

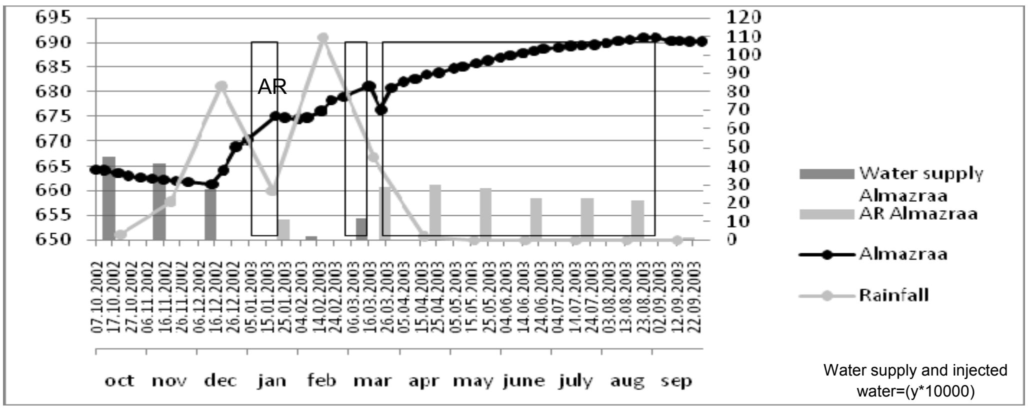


Fig (10): PM well hydrograph 2002-2003

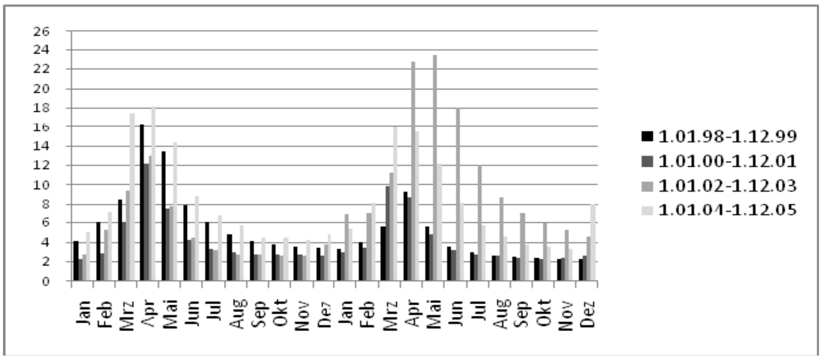


Fig (11): Discharges of Al-Figeh spring

The hydrograph shows a different trend from the one of 1998-1999 and the uprising trend is a reflection of natural and artificial recharge. The amount of injected water was 1817195m^3 (about 7 times of water quantity which was injected in 1997) and the maximum value of groundwater level was in August with 691m while in (1998-1999) was 683m and in 96-97 was about 690m . Water was not injected contentiously but there were interruptions. The recharge started January 6th to 21st. From January 22nd to February28th and from Mars17th - 22nd the recharge was interrupted. The hydrograph shows a small fluctuation during this period. The recharge was interrupted again from September 3rd and at September 20th the recorded level of Groundwater was 690.3m.

RESULTS AND DISCUSSION:

Artificial recharge process under current conditions in Damascus plain is one of the optimal solutions which can be added to other factors (new water resources exploration and water treatment) to avoid water shortage. The results have shown the possibility of successful AR. Nevertheless the impact is not significant. The water of Barada River must be recharged directly to prevent evaporation. The amounts which can pass through the pipe net are little and will not sustain the demand of the city. The need of a new design to recharge the surplus water directly in the aquifer is very urgent. The run off of Al Figeh spring varies in a wide range with a maximum up to 120 million m^3 in one outstanding year and this makes it difficult to build a system in the optimal dimension. Only the river bed can be used to transport any amount to localities where AR must take place. Beside technological aspects, social aspects should be taken into account. Barada River and the reservoirs have to be protected from pollution and a protection zone should be established. There is a necessity of education to prevent the dump of garbage during the dry season when the river bed is dry. The water supply of Damascus could be secured for another 60 to 120 days with a good managing of aquifer recharge.

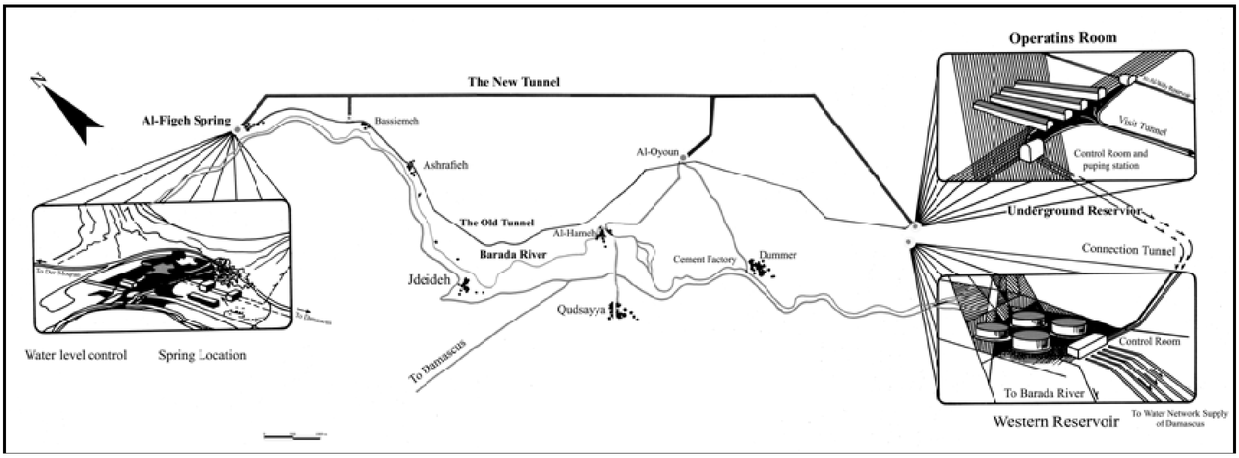


Fig (12): Al-Figeh spring and water reservoirs of Damascus city water supply

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Capturing the Lessons of ASR Failure from Trials in Unconsolidated Aquifers

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Abstract

In this paper the challenges in developing sustainable ASR operations in unconsolidated, low permeability aquifers, where operational problems such as well clogging tend to be more acute, are explored from the viewpoint of two case studies; one from Australia, the other from Thailand. Both studies were undertaken independently over different time-frames and brought together for comparative purposes. The Australian case study involved injection of wetland-treated urban stormwater into a low transmissivity, fine-grained siliceous aquifer; whereas the Thai case study involved injection of advanced-treated canal water into a colluvial/alluvial aquifer with high fines content. Although the design and execution pathways for the two trials were substantially different, ultimately the same endpoint (trial abandonment) was reached. This examination of the causative factors of failure, which were mainly related to poor recharge water quality for the Australian case, and poor hydraulic performance brought about by poor site selection for the Thai case study, lead to a number of key lessons being derived that could prove helpful to those considering ASR in similar environments.

Keywords: ASR, well clogging, redevelopment, unconsolidated aquifers

INTRODUCTION

Innovation and development in science and technology is often the product of trial and error, which is, by definition, dependent upon some degree of failure. As this becomes more fully understood, so too, failure is becoming less stigmatized (eg. note the growing number of on-line forums on the topic). An environment where failure is openly expressed and discussed can offer a useful channel for towards successful endeavours. In the field of aquifer storage and recovery (ASR) such channels have yet to completely open-up, as indicated by the literature having not adequately considered the negative aspects of ASR – the more recent work by Bouwer *et al,* (2008) is a notable exception. Evidence for this claim arises from an international review of ASR, where documented cases of ASR 'failures' were less than 10% of case studies identified (Pavelic and Dillon, 1997).

Clogging is an anathema to ASR researchers and practitioners (Pérez-Paricio and Carrera, 1999; Dillon, 2005; Pyne, 2005). It can be the first and sometimes the last thing to go wrong in the process of commissioning an ASR facility. Unconsolidated aquifers are globally-widespread and significant storage zones that present particular challenges to maintaining operational viability, particularly for low or moderate permeability formations. Opportunities to enhance groundwater resources through ASR have been diminished due to a lack of knowledge on the design and commissioning requirements for unconsolidated aquifers.

The object of this paper is to provide a concise summary of the underlying causes and main lessons learnt from two cases of ASR failure; one from Australia, the other from Thailand. The two cases were conducted independently and are brought together here for comparative purposes. More detailed analysis of the Australian site is given by Pavelic *et al,* (2008).

SITE DESCRIPTIONS

The Australian study was carried out at the Urrbrae wetlands site in Adelaide, South Australia (Fig. 1a) to test the viability of injecting wetland-treated urban stormwater into an unconsolidated siliceous aquifer to recover for summer irrigation. A ready source of water from a large (350×10^3 m^3 capacity) wetland at the site, a demand for water to irrigate the school playing field and the interest of a group of partners to develop an operational ASR scheme for the first time in this type of hydrogeological setting in Australia made the study viable and attractive for research. The target aquifers, known locally as the Carisbrooke Sand and the Port Willunga Formation, consists of medium- to fine-grained calcareous sand with some ferruginous and inter-bedded silt layers in the upper formation, and coarse sands and gravels with varying lignitic content in the lower formation. The ASR well was drilled by the rotary mud drilling method to a depth of 84.7 m and completed with 6-inch wire-wrapped stainless steel screen assembly over the three most productive layers identified from geophysical logging and drill-hole cuttings. The airlift yield of the well was 3 L/s and the bulk transmissivity was estimated to be around 6 m^2/day. Source water is taken from the wetland and pretreated by rapid sand filtration prior to recharge.

The second study was carried out at Nong Taphan, in Rayong District, Thailand (Fig. 1b). This site was favoured over a number of others as a result of a multi-criteria ranking procedure that took into account land availability, aquifer characteristics, water quality and data availability. The site is located next to a small irrigation canal within a broad valley, infilled by up to 300 m of weathered granite deposits. The target aquifer is comprised of alluvial and colluvial material including sands, silt and gravel interbedded with clay. The target unit extends over depths ranging from 40 to 70 m bgs, is semi-confined, and highly anisotropic and heterogeneous in nature. Transmissivities range from 4-55 m^2/day. The depth of the ambient potentiometric surface is around 1.5-3 m bgs. A dual-purpose ASR well and three observation wells were drilled by reverse circulation drilling at the site (a second ASR well targeting the shallow formation was also constructed but not discussed here). The ASR well was drilled at 16-inch diameter and 12-inch casing installed with wire-wrapped, stainless steel, screens and a non-calcareous gravel pack. A treatment plant was constructed to improve the quality of the raw canal (source) water by chemical dosing, flocculation and sedimentation then filtration, followed by de-oxygenation at the ASR well-head. This treatment train was constructed to minimize clogging risks associated with suspended particles and oxidation of reduced iron and manganese known to be present in the source water and at highly variable levels in the ambient groundwater which was devoid of oxygen.

Figure 1. Selected images from the Urrbrae site (left pane) and Nong Taphan site (right pane). *Photos for Urrbrae show an aerial view of the wetland (a); storage tank and ASR well head in background (b); rubber-lined pond and delivery pipe (c). Photos for Nong Taphan show canal (a); treatment plant (b); ASR well (c); monitoring well (d).*

RESULTS

The injection phase of the Urrbrae trial was operated over a 6-week period whereby about 4 $\times 10^3$ m^3 of stormwater was stored. Over this time, injection rates were reduced by more than 80% from an initial value of around 3 L/s to a final value of 0.5 L/s. Injection was thus halted due to the unacceptably low flow rate. The severity of clogging is indicated by pumping test data that showed that the specific capacity had declined from 13 m^3/d/m to <5 m^3/d/m. The potential causes of clogging included suspended solids or hydrocarbons entering the well (oil residues were observed in the wetland); biofilm production on the well screens and surrounding natural gravel pack; iron precipitation; gas entrainment and remobilisation of residual drilling muds or fines from the aquifer. Upon realization of the extent of the problem, a detailed program was conducted to restore the clogged ASR well involving repetitive backwashing; followed by injection of chlorine disinfectant; and then bailing/surging to recover sand that had in-filled the lowest screened interval. None of these techniques proved successful, and at the end of the redevelopment program the specific capacity had only returned to 6 m^3/d/m, or less than half of its initial value.

At Nong Taphan 6 weeks of injection was also performed. Initial injection rates commenced at 2.8-3.6 L/s but rapidly declined with an average overall of 0.9 L/s and the total injected volume of 3.2$\times 10^3$ m^3. Much of the recharged water was recovered and the turbidity initially evident in the recovered water quickly dissipated. No net deterioration in the specific capacity of the well was observed over the brief duration of the trial, but the short duration made long term performance forecasts difficult to establish.

DISCUSSION

The quality of the water for the Urrbrae site was far poorer than Nong Taphan, even though the transmissivity of the aquifer was five- to ten- fold lower (Table 1). Higher levels of pre-treatment was clearly needed for the Urrbrae site to avoid excessive well clogging problems. What is the quality of source water needed for sustained operations can be drawn from the experience of the Netherlands where the aquifers targeted are of a similar nature (Olsthoorn, 1982). Even though the quality of water required to avoid clogging will be very aquifer-specific, the generally accepted guideline requires high quality water, characterized by upper limits for values of clogging indicators such as the membrane filtration index (MFI) of <(3-5) s/L^2 and assimilable organic carbon (AOC) of <10 µg/L (Hijnen and van der Kooij, 1992). Table 1 shows that the quality of source water at Urrbrae was significantly poorer than these criteria, whilst the quality for Nong Taphan, although not demonstrably within those limits (since MFI and AOC were not measured), was certainly closer than Urrbrae, and suggests that a high quality water alone is not sufficient to ensure low clogging rates.

The reason why the performance of the Urrbrae ASR well was not restored significantly is interesting considering that these techniques have consistently been successful in a variety of other ASR studies. Particle filtration and biofilm growth were clearly partially responsible for reduced well efficiency. Mechanical clogging from residual drilling mud that may also have invaded the formation or remobilization and deposition of aquifer fines probably had a compounding impact, although this is difficult to verify in practice. Further, a perforation in the well screen joint caused by an act of vandalism (as seen from downhole camera footage) had caused infilling of the screens with sand and reduced the effectiveness of procedures to unclog the ASR well.

The Nong Taphan trial was conducted to test the viability of ASR; it did not set about to generate new water supplies that could be economically justified. At the same time there were minimum performance expectations that were strongly felt. The long term yield of the well of 4.2 L/s was unconservatively estimated to translate to an injection rate of 3.8 L/s, which lead to dissatisfaction in the performance and the swift termination of the trial. Considerable investment in drilling and geophysically logging of pilot boreholes; conducting surface geophysical surveys, and monitoring flow rate and waterlevel data on a continuous basis linked to a remote data acquisition system, could not overcome the fact that the aquifer had little additional storage capacity and low-moderate transmissivity. These characteristics were given insufficient weighting in place of easily redeeming features such as source water control and land availability. The host of factors that influence site selection were aggregated to determine a ranking value.

The semi-arbitrary way in which this is applied to each sub-criteria makes the process suboptimal, given that fatal flaws can be eclipsed by the other factors.

Table 1. The quality of recharge water at the Urrbrae and Nong Taphan sites

Parameter	Urrbrae	Nong Taphan
Target aquifer type	*marine sands*	*alluvium/colluvium*
Transmissivity (m^2/day)	*6*	*30-55*
Pretreatment	*wetland & rapid sand filtration*	*advanced, including de-oxygenation*
MFI (s/L^2)	90-390	-
Turbidity (NTU)	0.8-55	<2
TOC (mg/L)	4-13	<10
BRP (µg/L)	39-331	-
DO	present	<detection

CONCLUSIONS AND RECOMMENDATIONS

The results from two contrasting ASR case studies have been presented here. The hydrogeological conditions at the two sites have similarities, but there were vast differences in the level of investment given to establish and operate the trials, and differing emphases on the importance of follow-up investigations once problems were observed.

For the Urrbrae study, the fundamental problem was that the level of pre-treatment given to the source water was inadequate for the low transmissivity aquifer targeted, irrespective of the lack of success in restoring the injectivity of the ASR well. This was exacerbated by the absence of nearby observation wells and monitoring data during the injection phase of the trial to provide early-warning. For the Nong Taphan study the cause of clogging is difficult to ascertain precisely but likely to be due to mobilization of fines, possibly compounded by precipitation of iron and manganese brought about by ineffective operation of the de-oxygenation unit. The main issue in this case was the low performance relative to expectation, compounded by the selection of a site which could never achieve such expectations.

This cross-comparative work serves to reaffirm that failure is a relative rather than absolute concept, and highly site- and context- specific. A number of lessons have emerged that can be summarized as follows:

- where possible, conduct lab studies in advance of field trials to avoid investment losses later on

- source water quality guidelines to assess the likely clogging impact that have emerged (eg. Dillon *et al,* 2010) need to be demonstrated or adapted to local conditions

- monitoring of clogging would have provided valuable information to enable its management and been cheaper and more expedient than trying to infer causes and rectify clogging without the necessary data

- diagnosis of the cause of problems is made difficult if detailed investigations are conducted too late and in the absence of nearby observation wells and monitoring data

• site selection based upon multiple criteria, whilst a useful tool, should not be relied upon exclusively as it can be highly subjective

Further, firm criteria for success should be developed and agreed upon well in advance of constructing on-the-ground facilities. This would include clear definition of project goals and adherence to well-documented protocols for assessment of technical feasibility and conceptual design (Pyne 2005; Bouwer *et al,* 2008; Dillon *et al,* 2010).

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Aquifer Storage and Recovery Feasibility Study, Australian Capital Territory, Australia

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ABSTRACT

A feasibility study of the ASR potential of the Australian Capital Territory (ACT) was undertaken as part of a larger study of alternative water supplies for urban usage. The ACT is Australia's Capital and contains a population of about 350,000 people. Studies have been commissioned to replace up to 3000 ML/yr of potable water being used for non-potable end uses. Strategies that are being considered include stormwater harvesting, wastewater reuse and sewer mining.

ASR is being considered in conjunction with stormwater harvesting ponds, with some detailed desktop studies of harvesting costs being undertaken.

The ACT lies in southeastern Australia and experiences a temperate climate; receiving about 620 mm/yr rainfall opposed to about 1650 mm/yr evaporation. Rainfall has been below average since the year 2000 and temperatures have also increased since that date.

The hydrogeology of the ACT is comprised predominantly of fractured Paleozoic rocks with some minor alluvium along the main surface water drainage lines. Bore yields are highly variable, but rarely exceed 10 L/sec; storativity is low (less than 5%). Hydraulic gradients are variable, but steep, with the watertable a subdued reflection of the topography. Most groundwater discharge is as base flow to the areas streams.

Sites of enhanced bore yield have been identified as areas where small ASR schemes may yield between 50 and 200 ML/yr. A number of different ASR scheme designs have been proposed and these have been incorporated into a least cost approach to ranking stormwater harvesting options by CSIRO.

Consideration has been made of the existing regulatory system. The urban area has been sub-divided into several catchments, with each having a sustainable yield estimate. These estimates take account of ecosystem support via the linkage between the groundwater and surface water systems. They are explicitly defined to support baseflows in the area's rivers. In most cases, the full sustainable yield has been allocated to users. This means that ASR will be operated in a fully allocated system. As well, water quality objectives have been set for each catchment area and these are a primary constraint on the water quality able to be injected.

The study concluded that a specific style of ASR scheme is feasible for the groundwater systems of the ACT. Specifically, modelling external to the feasibility study showed that small ASR schemes of between 100 and 250 ML/yr capacity, coupled with a stormwater harvesting structure, could improve the economics of the system by allowing lower capital expenditure on a smaller design requirement for the harvesting structure.

A number of potential ASR scheme designs were considered.

KEYWORDS: Australia, Groundwater; Fractured Rock; Aquifer Storage and Recovery

INTRODUCTION

The recent drought across South-eastern Australia has highlighted the need to examine alternative sources of water for supply to the urban environment in the Australian Capital Territory – Australia's national seat of Government. In particular, water is being sought that would enable non-potable sources to be substituted in situations where the ongoing use of potable water is unnecessary and inefficient.

Territory and Municipal Services (TAMS) of the ACT Government retained the services of CSIRO Land and Water to undertake a study to assess the feasibility of achieving a 3 GL/yr water saving target by 2015, by utilising stormwater as a potable water substitute, within a Triple Bottom Line (TBL) decision-making framework. This study is called the Canberra Integrated Urban Waterways Project and is co-funded by the ACT Government and the National Water Commission.

The detailed objectives of the CSIRO study are to:

- Quantify the amount of stormwater that can be harvested by considering various harvesting options, e.g. new ponds, existing ponds and lakes and managed aquifer recharge (MAR) and possible mixing of stormwater with locally treated wastewater obtained through sewer mining
- Identify potential end uses of stormwater
- Generate a shortlist of supply/demand options using an appropriate screening process, by considering all possible combinations of stormwater capturing options, mixing stormwater with wastewater and potential end uses
- Evaluate performance of the shortlist of options and ranking them using a TBL (Triple Bottom Line) decision-making framework by considering hydrological, functional, social, economic and environmental aspects of supply/demand options in total water cycle context,

The study was undertaken in two stages. Stage 1 included achieving of objectives 1 to 3 above and Stage 2 included achieving of objective 4 above.

Aquifer storage and recovery (ASR) was identified as a key part of the process to obtain this integrated water solution. In recognition of this, TAMS engaged Salient Solutions Australia, Pty Ltd to provide expert advice to CSIRO Land and Water on aspects of ASR, so that ASR schemes could be fully integrated into the broader water supply options. The final report from CSIRO reflects that advice, and this report provides a compilation of the background material that was provided to CSIRO. The full ASR picture will be gained by reference to both the CSIRO final report and this report.

The work reported here is provided by permission of the ACT Government.

The objectives of the work to be carried out by Salient Solutions Australia where to undertake investigations to determine the potential for aquifers across Canberra to pump and store captured stormwater and recycled water, for recovery and use for irrigation.

The feasibility studies will involve consideration of the technical aspects of:

- storage capacity of the aquifer locally to the proposed stormwater harvesting areas and nearby areas of potential irrigation demand;
- groundwater hydraulic gradients at these sites and the resultant efficiency of such schemes at the local level;
- ability to get water into and out of the aquifer and how this might produce impediments to scheme operation;
- relevant water quality issues; and
- conclusions and recommendations as they are relevant to each site.

CONSTRAINTS TO IMPLEMENTATION

There are a number of constraints to the implementation of MAR schemes. The following lists these and discusses each within the context of the conditions in the ACT and specifically where there is mention of MAR in the ACT water management legislation.

Under the Water Resources legislation currently operating in the ACT, a recharge licence must be granted before any recharge of groundwater systems can be undertaken. In granting a recharge licence the legislation requires that consideration be given to, amongst other issues;

- the risk of the rising level of ground water damaging soil, rock or structures; and
- the risk of damaging ecosystems that depend on the area in question; and
- the risk of affecting the natural drainage of surface water of the area in question; and
- anything else the authority considers relevant.

The Environment Protection Act for the ACT seeks to maintain the quality of all waterways, including groundwater, across the ACT. The standard adopted is compatible with the environmental values of the waterway as mentioned in the Territory Plan, Water Use and Catchment Policies. The Territory Plan seeks to maintain the environmental values of all surface water in the ACT and does not specifically mention controls on groundwater. It is interpreted that the control on injecting water into aquifers is as it pertains to the impact that this injection would have on the water quality of associated surface water bodies.

Section 50 of the Environmental Protection Act refers to the discharge of stormwater to groundwater and makes it an offence to discharge stormwater that has greater than 60 mg/L suspended solids. No other water quality guidelines for receiving groundwaters are provided. This local guideline was an arbitrary value and was developed prior to the Managed Aquifer Recharge Guidelines (NHMRC 2009) were accepted by all Australian jurisdictions. These newer guidelines have since been implemented. The local water quality guidelines for injected waters are assumed to be those specified under regulations to the EP Act, for each Water Use and Catchment Code (from the Territory Plan). The overall guidelines would be the minimum requirement across all the specified Uses for a specific catchment.

Interpreting the above constraints is difficult and explicit discussion should be had with the relevant authorities to assess what controls will be enforced on any MAR scheme.

Generically, however, there are several constraints that can be noted.

- Groundwater levels should not be allowed to come within a certain distance of the groundsurface during the operation of an MAR scheme. This distance is debateable at this time, but should be about 5 metres. This control is directly aimed at avoiding damage to infrastructure from high watertables.
- The water quality of injected water should not cause current groundwater users in the vicinity of a proposed scheme to cease using groundwater because of changes to water quality as a result of the injection.
- The water quality of the receiving groundwater should not be such that it causes the water quality of the recovered water to exceed the water quality requirement of the intended user.
- The water quality of the injected water should not cause the environmental values of any receiving surface water body to be diminished.
- The extraction of groundwater from an MAR scheme should not infringe on the rights of existing groundwater users in the vicinity.

An additional constraint that applies (and was implemented after the study was completed) was that according to the national MAR Guidelines, extraction of groundwater from an ASR scheme should not deplete base flow. This is not a major issue in the ACT as the Sustainable Yield of the aquifer is determined so as to safeguard the base flow of the rivers and streams. As well, ASR schemes will not be allowed to extract more water than has been injected, so the long term water balance should be maintained. The residual issue then is whether the recovery phase during the summer use period will locally impact on base flows seasonally. This will need to be managed locally.

REGIONAL HYDROGEOLOGY

The following section describes the hydrogeology of the ACT urban area and is based on previous work by Evans (1987) with some modification. Additional drillhole data has been added to the previously available data set and bore yields have been updated.

The geological history of the ACT region has been previously reported by Owen and Wyborn (1979), Richardson (1979) and Abell (1992), and is summarised in Evans (1987).

Evans had previously subdivided the ACT and environs into eight hydrogeological units or provinces:

- Ordovician and Early Silurian sedimentary rock sequences;
- Older Middle Silurian volcanics;
- Younger Middle Silurian volcanics;
- Canberra Formation;
- Volcanics of the Captains Flat – Lake George area;
- Intrusive granitoid rocks;
- Devonian volcanics to the north of the ACT; and
- Alluvium and colluvium.

Descriptions of these rocks are provided in Evans (1987).

Of these provinces, two fall outside of the ACT and will not be considered further; these being the volcanics of the Captains Flat – Lake George area and the volcanics to the north of the ACT. In addition, this study has highlighted part of the Older Middle Silurian volcanics, the Painter Volcanics, as a separate unit. For the purposes of this study, there are seven hydrogeological provinces that will be analysed further:

- Ordovician and Early Silurian sedimentary rock sequences;
- Older Middle Silurian volcanics (except Mt Painter Volcanics);
- Mt Painter Volcanics
- Younger Middle Silurian volcanics;
- Canberra Formation;
- Intrusive granitoid rocks; and
- Alluvium and colluvium.

The following section describes the hydrogeological characteristics of each of the seven provinces identified above. The provinces can be grouped into fractured rock aquifers and aquifers in alluvium/colluvium.

Fractured Rock Aquifers

The distribution of the hydrogeological provinces is shown in Figure 1 and each is described in the following sections.

In general, the fractured rock aquifers are composed of a zone where the fracturing in the rock is open to depth, and this allows water to infiltrate and move through the rock mass. Groundwater flow is via gravity and usually flows according to topographic gradients. Nearly all recharge to the aquifer ends up as discharge to neighbouring streams. It is rare for groundwater flow to by pass a topographic catchment. Thus, most streams act as discharge points for the aquifer.

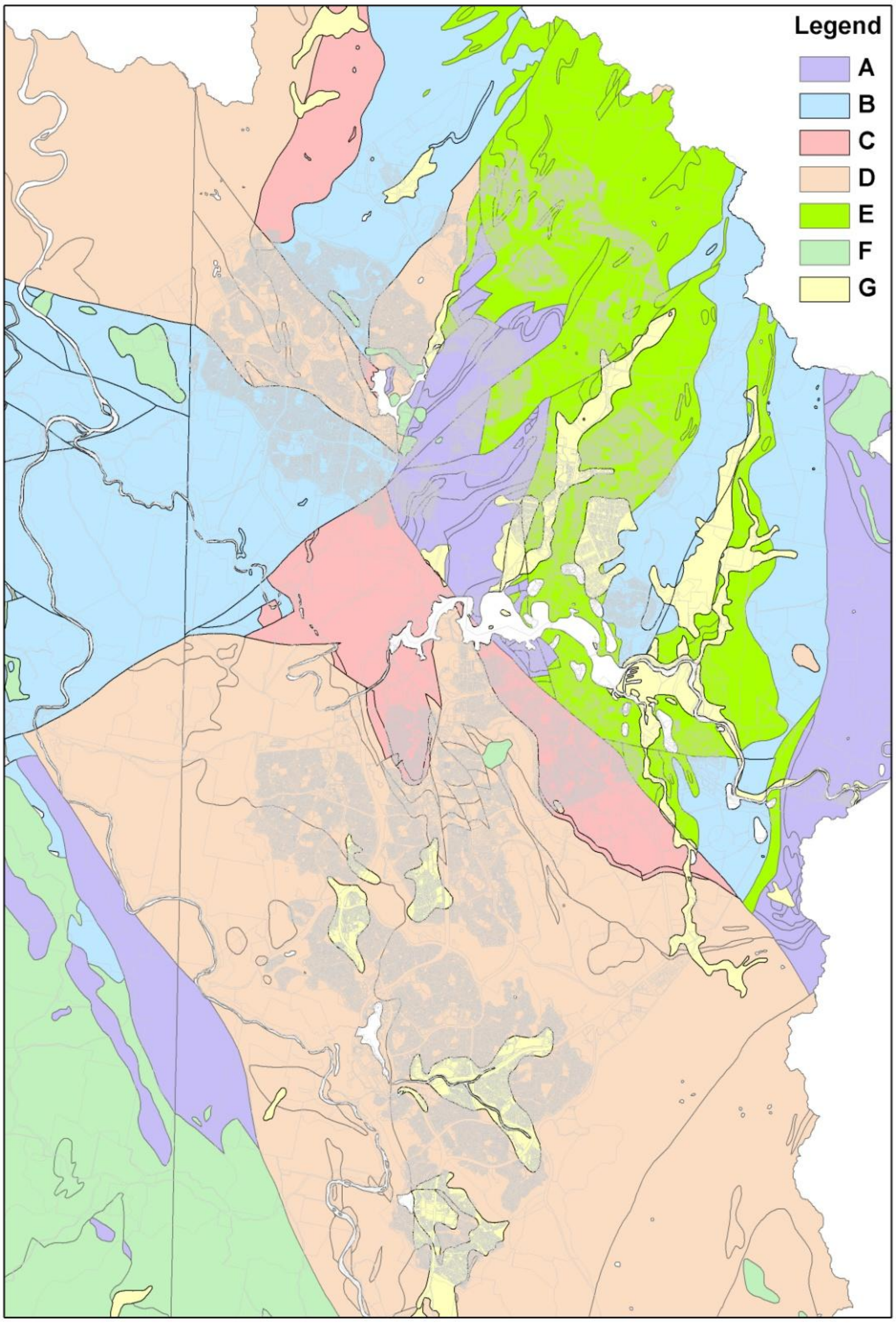

Figure 1: Distribution of hydrogeological provinces across the ACT urban area

Fractures are usually open to depths of 100 metres or so. Groundwater may be interested below this depth, but it tends not to contribute significantly to bore yield. Most bore yield is obtained in the top 40 to 60 metres.

Ordovician and Early Silurian sedimentary rocks (A)

The Ordovician and Early Silurian sediments are found in one distinct zone in the urban area – stretching from just south of Lake Burley Griffin in the Parliamentary Triangle north-westwards to Black Mountain and the O'Connor Ridge.

The province is also found in the Kowen area north of Queanbeyan and in the hills immediately west of the Murrumbidgee River in the Tidbinbilla area.

The major rock type is interbedded sandstone, siltstone and mudstone that has been highly folded and faulted. The rock has usually been weathered to some degree.

Older Middle Silurian Volcanics (B)

This rock province comprises a range of volcanic rocks that were formed during the earliest phase of volcanism in the region and is found in three major areas:

The Mt Majura – Mt Ainslie ridge line and to the eats of the Woolshed Creek valley;

In a northeasterly trending region centred on Hall; and

Immediately southwest and west of Belconnen stretching further west across the Murrumbidgee River.

Rock type varies from massive ashflow tuffs to thinner interbedded sedimentary units. The rocks are distinguished by the presence of pyrite and other sulphide mineralisation. This has a major impact on groundwater salinity.

The unit has been folded and faulted.

Mt Painter Volcanics (C)

The Mt Painter Volcanics are part of the older middle Silurian volcanic rocks but have been partitioned from them based on the higher bore yields obtained across the ACT urban area.

The rock unit is found in the Jerrabomberra – Narrabundah – Red Hill – Forrest area stretching northwestwardly across Scrivener Dam. There is also an area of Mt Painter Volcanics lying west of Hall, but outside the ACT.

The rocks are comprised of similar ashflow tuffs and interbedded sediments. The unit also includes the Yass sub-group which is a thin layer at the very top of the sequence. This sub-group is comprised of sediments including limestone.

The unit has been folded and faulted. This unit does not appear to have the same level of sulphidic material associated with it, and it supports a large number of groundwater users.

Younger Middle Silurian Volcanics (D)

The younger Middle Silurian volcanic unit occupies the large tract of land underlying Lanyon, Tuggeranong, Woden and Weston Creek. It is also found under parts of Belconnen and further to the northwest.

The province includes thicker areas of sediments such as the Yarralumla Formation found along the Red Hill ridge, and in the valley at the northern end of Yarralumla Creek

It has similar rock types to the older volcanic province, but has no sulphidic material associated with it. The Yarralumla Formation is comprised of sediments including calcareous varieties. Except for the Yarralumla Formation, the rocks of this province tend to be more massive than the older volcanics. It is also faulted and folded.

Canberra Formation (E)

The Canberra Formation is found in a broad U shape running down the centre of the Woolshed Creek valley and then northward from Lake Burley Griffin underneath the Sullivans Creek catchment and further north through Gungahlin.

The province's rock types are more varied than the other provinces, comprising sandstones and mudstones, tuffaceous sediments, ashflow tuffs and the occasional limestone layer. The rocks have been folded and faulted.

Granitic Intrusive rocks (F)

Granitic intrusive rocks do not occupy any of the urban area of the ACT except for a small intrusion under the Federal Golf Course. The main body of this hydrogeological province is found in the Brindabella Mountains to the west of the Murrumbidgee River. The province is comprised of the various Siluro-Devonian intrusive rocks – granite, granodiorite, adamellite, etc.

These intrusive rocks are fractured and faulted.

Colluvial/Alluvial Aquifers

The lower slopes and valley floors of the ACT region are covered by a shallow layer of alluvial and colluvial material of recent geological age. These materials are generally clayey in nature, but coarser layers are present throughout. These coarser layers act as shallow aquifers. Several areas of well developed aquifers are already known, but large tracts of sediments are unexplored and the full extent of the aquifers is unknown. The well developed alluvial/colluvial aquifers are found in the Dairy Flat area associated with the Molonglo River, along Woolshed Creek, and in the Lanyon and Wanniassa areas. Smaller occurrences are known along parts of Sullivans Creek and in the Horse Park area of Gungahlin. Other occurrences are expected to exist.

Groundwater flow in these shallow aquifers is nearly always associated with their associated surface water features. Recharge is via rainfall and streamflow, and discharge is always to the surface water feature. Upwards leakage from underlying fractured rock aquifers also occurs in the lower parts of each catchment.

ASR POTENTIAL

A measure of the potential feasibility of any MAR scheme is the rate at which groundwater can be injected and extracted from a particular site, the ambient groundwater quality and the storage capacity of the aquifer. The following sections outline qualitatively, how these three aspects vary across the hydrogeological provinces.

Bore Yields

Bore yields within the hydrogeological provinces vary according to the degree of fracturing intersected in any borehole (in the case of the fractured rock aquifers) or whether coarse material is encountered in any alluvial/colluvial sequence.

Figure 2 provides the cumulative percentile bore yields for each hydrogeological province. Overall the data shows that bore yield across the ACT varies from 0 up to 24 L/sec. However, the median bore yield varies between 0.5 and 1.1 L/sec across the provinces. This means that 50% of bores drilled had a yield greater than this value. More interestingly, the 80th percentile value varies between 1.5 and 2.5 L/sec, again meaning that only 20% of bores drilled have a higher bore yield.

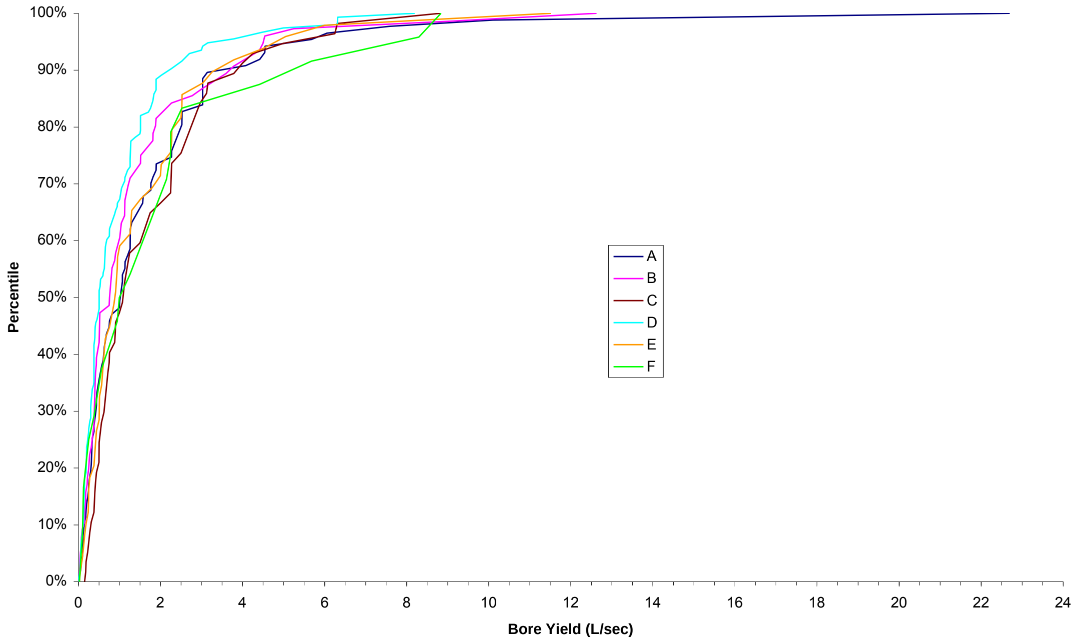

Figure 2: Percentile distributions for bore yields for each hydrogeological province. Note that the number of bores for each Province are: 93 for Province A; 34 for B; 30 for C; 165 for D; 42 for E; and 21 for F

The provinces can be ranked on their bore yield distribution. In order of highest to lowest bore yields they are C and A, E and F, B and then D.

Groundwater Quality

The quality of groundwater does not vary much between hydrogeological provinces. Rather, there is more variability within each province between the fresher water in the main recharge areas, compared to the more saline water in the discharge areas. Overall, however, groundwater salinity is fresh when compared to other areas of south eastern Australia. The freshest aquifer is province F; all other provinces are essentially the same groundwater quality, except for province B which has the highest groundwater salinity.

Groundwater salinity is relatively fresh, with the highest salinity being about 3,500 mg/L (province B). In most areas, salinity is less than 1,500 mg/L. The range of salinity (other than for province B) does not vary markedly between provinces. Rather, there is marked variability according to geomorphic and landscape position (Evans, 1987). Areas higher in the landscape have lower salinity generally, when compared to areas lower in the landscape. As will be discussed in the next section, it is the higher landscape areas (and hence fresher salinities) where it is more likely that ASR schemes will be feasible.

Iron and manganese are important chemical constituents that can confound ASR schemes by clogging of screens and aquifers. The groundwater chemistry data were not comprehensive enough to provide a large number of values for these constituents. Anecdotal evidence suggests that some hydrogoelogical provinces are linked to higher levels of both elements, but a more thorough understanding of their distribution is required prior to development of an ASR capacity.

Depth to Watertable

Depth to watertable will control the ability to inject water into an aquifer without raising the watertable (or groundwater level) to levels that could be assessed under the ACT Water Management Act as being capable of damaging soil, rock or structures. The height to which groundwater levels rise when water is injected into aquifers is controlled by the specific yield of the aquifer (a parameter related to aquifer storage). The fractured rock aquifers all have similar specific yield values (between 0.005 and 0.02), while the alluvial/colluvial aquifers have specific yield values of between 0.05 and 0.2.

When water is injected into an aquifer the level will rise based on the specific yield. That is, if one metre of water is added to an aquifer over its area, water levels will rise by 5 m if the aquifer has a specific yield of 0.2 or by 50 m if the specific yield is 0.02.

Given that most of the fractured rock aquifers discharge to the drainage lines, and therefore have water levels close to the surface in those drainage lines, it is obvious that injection sites need to be set back from the drainage lines by some distance to satisfy the depth to watertable test. Likewise, the alluvial/colluvial aquifers are primarily situated along drainage lines and will therefore be subject to the same constraints.

As an example, consider the requirement to store 1 ML of water in an aquifer over an area of 1 ha at a specific yield of 0.01. This would require a *head space* in the aquifer of 10 metres. To store 100 ML of water and restrict the water level rise to 10 metres, one would need an area of 100 ha of aquifer.

One might use this example to set a criterion that stipulated that sites must have a depth to water of at least 15 metres prior to injection to meet the current legislation pertaining to impacts of injection. However, this issue needs to be explored further as there is no current definition of how this requirement can be quantified without accurate aquifer characterisation and modelling.

The depth to watertable is the same across all provinces, in that it varies according to topographic position. That is, it is deepest under the ridges and hills, and shallowest along the drainage lines.

OVERALL POTENTIAL

The forgoing discussion can be compiled into an overall potential rating for each hydrogeological province for MAR schemes in the ACT. Figure 3 shows the potential rating.

The rating of potential has been decided based heavily on the available bore yield information, as groundwater salinity and depth to watertable either vary little between provinces or have more variability based on topographic position. However, as shown in Figure 2, there is little discrimination in yield between the various hydrogeological provinces. The approach was to consider the difference in province bore yield at the 80th percentile ranking, and to consider the economics of ASR schemes at three different yields. The difference in yield at the 80th percentile level is from a low of 1.75 L/sec to a high of 3 L/sec, representing a difference by a factor of about two. This can be further developed as the chance of obtaining a supply at that level of yield is about one in five.

CSIRO undertook economic modelling of stormwater harvesting sites. Part of the modelling exercise was to consider whether a site was economic as a purely surface water harvesting site or whether the inclusion of an ASR component reduced the overall price of produced water. The modelling was conducted on a number of generic ASR schemes as set out in the section on ASR Conceptual Models (below). The bore yield of these generic models were also varied to include three yield scenarios, each ASR scheme with yields of 1, 2 and 4 L/sec. The modelling showed that schemes had the lowest water cost for the highest yield, and that schemes where yields were 1 L/sec were not economic.

This economic modelling work provided the basis to rank the various hydrogeological provinces in terms of their overall feasibility.

The province ranking from highest potential to lowest is C, A, G and E, B and F and D. Figure 3 shows the distribution of the potential for ASR schemes and is based on the conclusions discussed above. The ranking represents areas where it is more likely to have a successful ASR scheme form areas where it is less likely. The ranking relies on the generic modelling results that show that a scheme with bore yields less than 2 L/sec are not economic.

Given that the bore yields used in this analysis were based on the 80th percentile ranked yield, even in the highest potential category the chances of developing such a scheme are still only about 20%. The utility of the potential ranking is to establish that it is economic to include ASR schemes in association with stormwater harvesting in certain areas, and it is not economic in other areas. Local hydrogeological conditions will then dictate whether such a configuration will actually be possible.

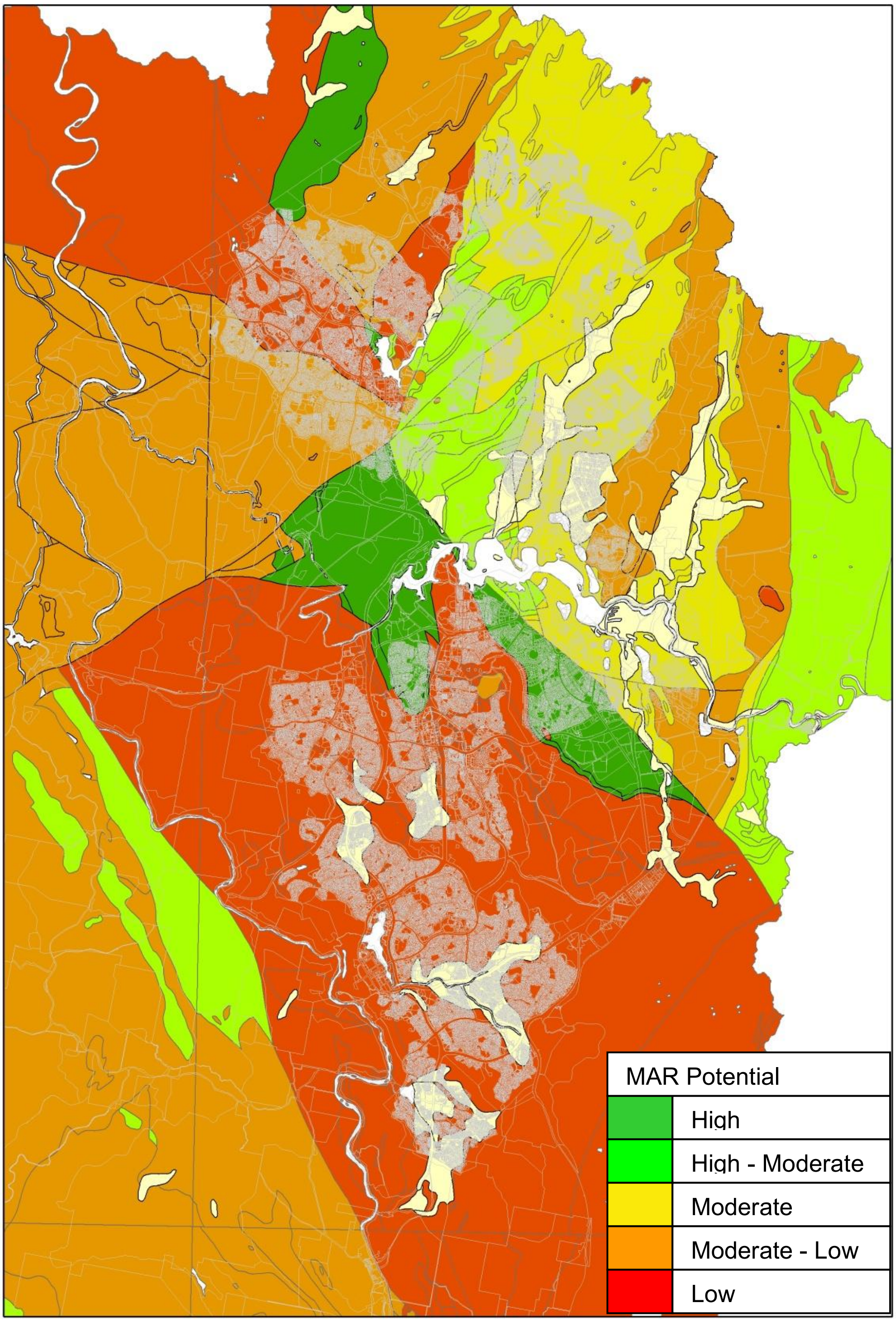

Figure 3: MAR Potential for the ACT

ASR CONCEPTUAL MODEL DESCRIPTION

The following are generic examples of different types of ASR schemes that are considered to be appropriate to the ACT Urban region. The examples set out the basic conceptual model of the receiving aquifer as well as an initial design scope of what infrastructure might be required. These generic models were used by CSIRO to model the economics of stormwater harvesting with and without an ASR component.

In all cases the demand is assumed to be 5 ML/ha/year (5 ML/ha over 1 ha for one year). Further, the demand is assumed to be over a 7 month irrigation period (October to April). As mentioned above, bore yield varied across three values, 1, 2 and 4 L/sec.

The generic ASR conceptual models are not necessarily located within any one hydrogeological province. In some instances, however, it is obvious that some models are specific to a province; for instance, models ASR 2 and ASR 3 are both only applicable to colluvial/alluvial aquifers.

ASR 1: Fractured Rock

This example describes the conditions likely over most of the ACT, with an ASR scheme of single injection/recovery bores.

Assumptions:

- Average bore yield is 2 L/sec (range is 1 to 4 L/sec);
- Injection rate and bore yield are equal;
- Irrigation demand is 5 ML/ha/year;
- Recovery efficiency for single bore ASR is 50%;
- Fractured aquifer is 70 metres thick;
- The delivery reliability of the water extracted from an ASR scheme is 100%. That is, water will be supplied at the stated rate for all times;
- Injected water will be required to be free of all particulate matter;
- Any water 'lost' from an ASR scheme will eventually return to streamflow as baseflow. Therefore, the timing and ecosystem costs and benefits of decreased and increased baseflow needs to be factored into the costs and benefits calculations when considering alternative schemes;
- Areas where the thickness of the unsaturated zone above the watertable is at least 15 m, are situated at least 30 m above the valley floor;
- Drawdown in the bore during recovery mode is 30 m.

The following is based on a per unit irrigated area basis.

An injection rate of 2 L/sec is equal to 0.173 ML/day. This also equals the demand at the water source at 100% reliability for 5 months. There is no requirement for water supply during the injection season.

The injection rate of 0.173 ML/day over a 5 month period (153 days) is about 25 ML. Assuming 50% efficiency, this would provide a water supply of 12.5 ML over a 7 month period, enabling irrigation for 2.5 ha.

Each bore would require a drillhole at a minimum 200 mm diameter to 70 metres at a cost of $200/metre cased. Headworks would cost $25,000 per bore, and telemetry/control gear would cost an additional $5000 per bore. This equates to a total capital cost of $50,000 per bore.

Ongoing operational costs would involve a 30 metre lift from the water source (pond) to the bore and another 30 metre lift from the bore during the pump season.

ASR 2: Shallow Alluvium

This example details conditions associated with alluvial aquifers in the ACT. These types of aquifers are restricted to the major drainage lines and as such, are found low in the landscape. Examples include floodplains and low terraces associated with Dairy Flat, Woolshed Creek, Sullivan’s Creek, Ginninderra Creek, etc.

The most efficient Managed Aquifer Recharge scheme suited to these environments would be via ‘spreading’ techniques where surface water containment structures were designed to leak into the shallow alluvium with recovery works further down the valley. Such recovery works might include cut-off trenches or spearpoint systems.

Some challenges exist for construction of these types of MAR options.

They are generally in the bottoms of the major drainage lines and as such are directly connected to the associated surface water systems. This connection will result in inefficient schemes, with a larger proportion of water lost from the scheme to base flow. In other studies, a set back of at least 200 metres has been suggested as a nominal distance to decrease the losses to streams from ASR schemes. A setback distance of this magnitude would eliminate most alluvial deposits in the ACT from consideration.

The aquifer properties of the alluvium are highly variable and the investigation costs of proving individual scheme feasibility will be high.

The thickness and distribution of all alluvial deposits is unknown.

Watertables are very close to the surface in the bottoms of the valleys. This is a consequence of these sites being the discharge zones of the broader aquifer flow system. Any additional recharge superimposed on these shallow water level depths would result in water levels being either at or above the ground surface. This has the potential to result in waterlogging and resultant infrastructure damage.

The site of potential water demand will need to be close to the valley floor. Any demand site away from the valley floor will require both supply mains and will incur pumping costs.

Design of the recharge works will require excavation into the alluvial aquifer so that a high rate of leakage can be obtained. This excavation will be required after the water has been cleaned of particulate matter. A passive recharge basin or similar structure will be subject to clogging, so higher than usual levels of maintenance will also be required.

Even though this type of scheme is suited to the ACT environment it will not be analysed further due to poor quantification of the costs.

ASR 3: Sportsgrounds

A special case of Case ASR 2 is that associated with sportsgrounds.

In some instances, the substrate below sportsgrounds has been able to store excess irrigation water. There is potential for this water to be used subsequently to meet irrigation demand in the future. There is also potential for the water held in storage to be augmented from harvested stormwater or from sewer mining sources.

The highest potential for viable substrates associated with sportsgrounds will occur where there is:

Sufficient storage capacity in the sediments immediately below the sportsground; and

A confining layer that precludes deeper movement of any irrigation leakage to other aquifers.

The optimum ASR scheme would appear to be a combination of local water capture (either via a small weir or similar structure), a wetland filtration system to remove particulate matter, an injection design that allowed gravity feed to a coarse substrate and a collection system (or sump) where water under the sportsground could be collected for re-use during periods of irrigation demand.

Assuming that individual sportsgrounds are about 1 ha in size, this would entail a scheme designed to deliver about 5 ML/year.

Impediments to the viability of such schemes are the lack of data on the subsurface characteristics of sportsgrounds in the ACT and the 'discovery cost' to obtain such data.

ASR 4: Aquifer Storage, Transfer and recovery

This case involves designing schemes around paired injection and recovery wells. There are two subsets of this type of scheme. The first involves using the aquifer as the distribution system (and therefore reducing the costs of piping) and injecting water close to the source, with recovery undertaken at some distance from the source. This subset is referred to as ASTR Regional. The second subset is where paired injection/recovery bores are used at the demand site. This is a variant of case ASR 1 and is referred to as ASTR Local.

The benefits of an ASTR approach would be:

- Increased efficiency due to the separate recovery bore sited down gradient from the injection bore. It is assumed that recovery efficiency would rise to 75%;
- Ability to inject over a 12 month period, thus reducing the demand at the source and increasing the annual efficiency of storing water (that is, no long periods when water would not be harvested at the source). This would also lead to lower required rates of injection and the possibility of finding more suitable sites for such schemes. Water availability may be an issue, but surface water flow modelling showed that on average, surface water would be available for harvesting in all years.

Disbenefits of an ASTR approach would be:

- Increased infrastructure costs associated with paired bore injection/recovery;
- The need for increased maintenance to avoid clogging in the injection bore.

ASR 4a: ASTR Regional

As discussed above, this case involves injecting water into the aquifer very close to the water source and allowing the natural hydraulic gradient of the groundwater to transport the water to the demand site.

The main impediment to the implementation of this type of scheme is the requirement that the regional flow pattern is such that groundwater can be transported to the site where it is required. This can only happen where the groundwater flowlines are essentially parallel, as opposed to convergent on a discharge point. A good example of this type of groundwater flow system is the broad slopes of Forrest and Red Hill (though there is no water source near the top of the groundwater flow line in this example).

ASR 4b: ASTR Local

As discussed previously, this is similar to case ASR 1 but has paired injection and recovery bores.

ASR 5: Allocation Increase (Transferred Rights)

The final example of an ASR scheme that could potentially be used in the ACT is where water is injected into an aquifer with the intention of increasing the Sustainable Yield of the aquifer and thus allowing further granting of entitlement within the water resource management sub-catchment, but not necessarily trying to capture the injected water. This could occur where there were potential groundwater users who were unable to access groundwater in a sub-catchment because the current sustainable yield volume was already fully allocated.

The scheme would rely on the fact that any injected water would eventually exit the sub-catchment as baseflow in a stream. As long as extraction of additional entitlement didn't exceed the amount of water being injected, the policy goal of protecting baseflow would be met. The advantage of such a scheme would be to avoid the cost of piping and annual recovery costs, as there is no need to pipe water to the user.

Essentially, the result of this type of scheme would be to convert peak surface water flow into baseflow (that is, altering the flow duration curve without diminishing the overall total volume of flow).

CONCLUSIONS

The study concluded that a specific style of ASR scheme is feasible for the groundwater systems of the ACT. Specifically, modelling external to the feasibility study showed that small ASR schemes of between 100 and 250 ML/yr capacity, coupled with a stormwater harvesting structure, could improve the economics of the system by allowing lower capital expenditure on a smaller design requirement for the harvesting structure.

Further, it was shown that bore yields less than 2 L/sec would not support an economic ASR scheme. This provided a cut-off in terms of which hydrogeological provinces were more likely to be favourable for consideration of coupling an ASR scheme to stormwater harvesting.

The approach was one of establishing the theoretical feasibility of ASR schemes in this fractured rock, low bore yield environment. It showed that whilst it is theoretically feasible to develop such a scheme, the chances of finding the right set of conditions were low (about 20% at best in the most favourable setting). This indicated that the largest risk in terms of implementing such an approach was the discovery costs of finding a suitable ASR site close enough to a stormwater harvesting location to be economic.

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SELECTION OF SUITABLE SITES FOR ARTIFICIAL RECHARGE IN KUWAIT USING GIS TECHNOLOGY

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Abstract

Advantage of a hydrogeological database for Kuwait, developed recently, was taken for selecting suitable areas for artificial groundwater recharge in Kuwait using Geographical Information System (GIS) technology. Hydrogeological criteria for the suitability of an aquifer for artificial recharge in Kuwait were established based on the field experience in Kuwait and elsewhere and numerical modeling study. These criteria were used in delineating areas where the two target aquifers in Kuwait, namely, the Kuwait Group and the Dammam Formation, have suitable combination of aquifer properties for artificial recharge. The nearness to infrastructure like roads, power and water supply, the main demand centers, etc. were taken in to account in the ultimate selection of the areas that could be considered for recharge. The areas were further graded on a suitability scale of 1 to 4 (least suitable to most suitable) depending on the degree to which the areas have the preferred combination of the hydrogeologic parameters. Three areas were found to be suitable for recharge in the Kuwait Group aquifer, whereas only one area was found to be suitable for recharge in the Dammam Formation aquifer.

Keywords: Dammam Formation; Kuwait Group; Aquifer; Hydrogeological Database

Introduction

Arid Kuwait depends completely on its desalination facilities for its requirement of potable water and has to have a substantial water storage capacity to cater for the basic survival needs of its population during an emergency when the desalination plants may be out of commission. Expansion of the conventional surface-storage facilities to meet demands for longer periods is very costly. There is, however, a large volume of storage available in the aquifers. There is also a significant augmentation of this storage capacity due to the exploitation of these aquifers at increasing rates since the 1960s. The utilization of this natural storage for creating a reasonably large reserve of useable water is technically feasible and less expensive than surface storage facilities.

A series of studies carried out by the Ministry of Electricity and Water (MEW) (Senay, 1977; Al-Sumait and Senay, 1999) and Kuwait Institute for Scientific Research (KISR) (Mukhopadhyay, 1992; Al-Awadi *et al*., 1994; Mukhopadhyayay *et al*., 1994; Mukhopadhyay *et al.*, 1997; Viswanathan and Al-Senafy, 1998; Mukhopadhyay *et al*., 1998; Al-Otaibi and Mukhopadhyay, 1999; Mukhopadhyay *et al*., 2001; Mukhopadhyay and Al-Otaibi, 2002; Mukhopadhyay *et al*., 2004) over the past several decades have indicated that Aquifer Storage Recovery (ASR) is a viable option for Kuwait, both technically and economically. The selection of suitable sites for the creation of a strategic reserve of useable water for emergency use, taking into considerations both hydrogeological and operational optimization is reported here. The generalized hydrostratigraphy of Kuwait is presented in Fig. 1.

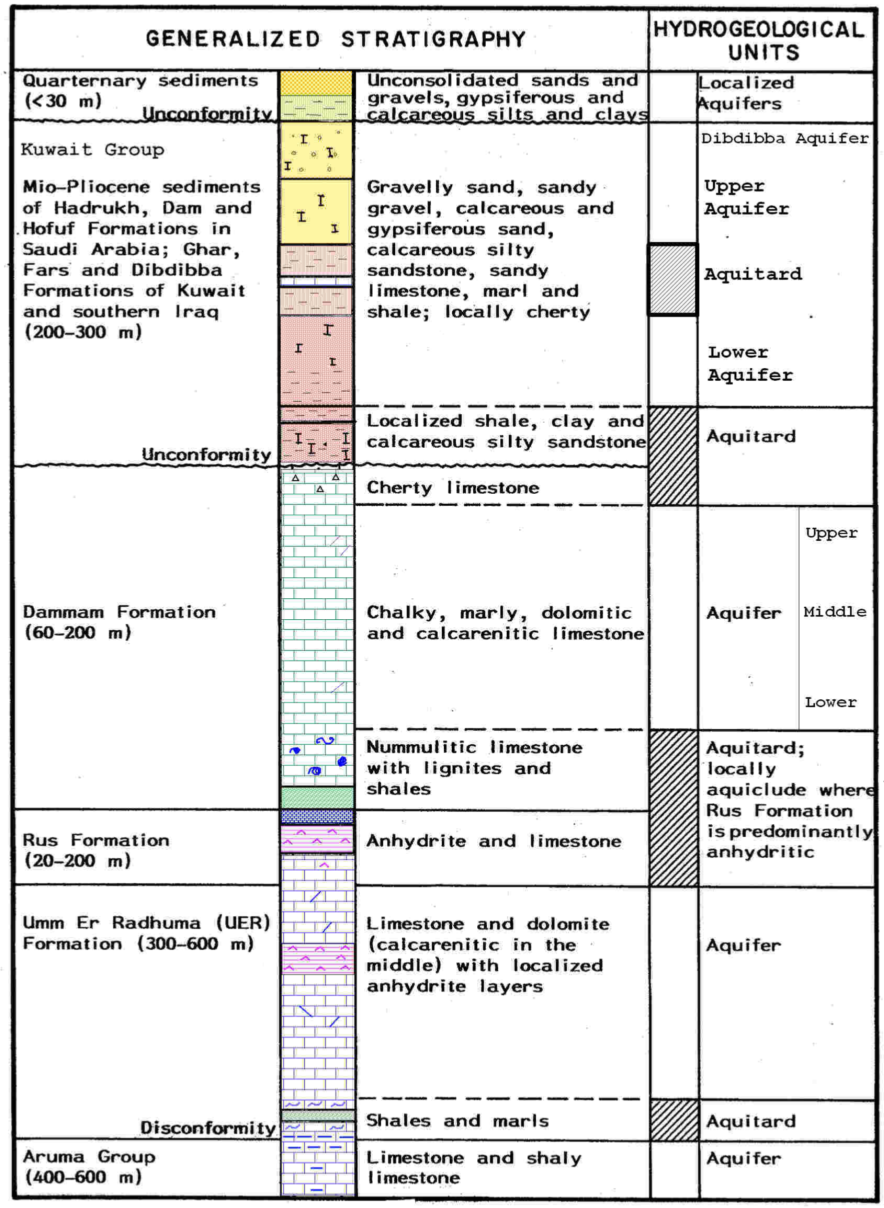

Fig. 1. Generalized stratigraphy of the Tertiary sediments in the Arabian Peninsula (Source: Mukhopadhyay *et al.*, 1996)

Method Used for Site Selection

Field experiments (Mukhopadhyay et al., 1994), laboratory investigations (Mukhopadhyay *et al.*, 1998; Mukhopadhyay *et al.*, 2004) and sensitivity analysis through numerical modeling (Mukhopadhyay and AlOtaibi, 2002) have suggested that in order to achieve optimal recovery and operational efficiencies, the aquifer characteristics at the selected recharge site should meet certain criteria. Besides, operational and economic considerations also determine the suitability of a site for recharge. The criteria for suitability can thus be categorized as a) Hydrogeological considerations; and b) Operational and economic considerations. For conditions prevailing in Kuwait, these criteria are presented below.

Hydrogeological Considerations

Transmissivity. The transmissivity should be moderate. Too high transmissivity that often means very high hydraulic conductivity where the aquifer thickness does not change drastically within short distance, although allows high injection rates, causes large scale mixing with the native groundwater, causing a reduction in recovery efficiency. If high transmissivity is the result of large aquifer thickness, the injected volume of water creates only a narrow cylindrical zone around the well with large scale mixing with native groundwater. Too low a transmissivity, on the other hand, causes rapid rise in water head within the injection well and limits injection rate. Clogging of the injection face also becomes a more serious problem in low transmissivity aquifers, especially if the aquifer is clastic in nature. Due to the karstic nature of the Dammam Formation, the problem of clogging was found to be less in this aquifer and a reasonable range of transmissivity for success of artificial recharge was assessed to be 150 - 500 m^2/d.

Field experiments and laboratory investigations, combined with the results from the numerical modeling of the Dammam Formation aquifer provided guidelines for setting the suitable transmissivity range for this aquifer. Since the clogging problem is more acute in the clastic Kuwait Group aquifer, a higher range of transmissivity (250 - 1000 m^2/d) is assigned for the viability of artificial recharge in this aquifer that is also supported by the results of field experiments.

TDS. To avoid lowering of the recovery efficiency due to mixing and effects of buoyancy (Merritt, 1985), the TDS content of the native groundwater should not be too high. In Kuwait, hydrogen sulfide (H_2S) is often associated with high TDS groundwater (TDS > 10,000 mg/l). The high TDS areas are to be avoided for this count also. Numerical modeling of recharge in the Dammam Formation aquifer (Mukhopadhyay and Al-Otaibi, 2002), has indicated that other conditions remaining the same, the recovery efficiency decreases almost four folds (from 41% to 10%) when TDS of the native groundwater increases from 5000 mg/l to 10000 mg/l. Thus 5000 mg/l appears to be the upper limit for TDS content of native groundwater to obtain reasonable (40% and above) recovery efficiency from artificial recharge.

Depth to Water. In the areas where the water table in the target unconfined aquifer is too near the surface (depth ≤ 50 m), there will be very little room for accommodating the rise in water head in the injection well. This is both due to normal mounding from recharge and from clogging of the injection face by suspended solids and dissolved gases in the injection water and the rise may reach 50 m or more, as indicated during field experiments and numerical modeling. The flooding and instability of the area surrounding the injection well can also occur under such circumstances. In the case of a confined aquifer, if the potentiometric head is very high reaching near the surface, this will mean injection under pressure that is operationally difficult.

Lateral Hydraulic Gradient. If the original lateral hydraulic gradient is steep, the body of the injected water tends to flow down the hydraulic gradient away from the injection point. As a result, the ASR well may not be able to capture all of the injected water during recovery, thus reducing recovery efficiency. Downgradient movement of the injected water body will also encourage more mixing with the native water that is mostly poorer in quality and will further reduce the recovery efficiency. The recharge site should, therefore, have a lateral hydraulic gradient as low as possible. In the present case, it has been empirically decided to limit the lateral hydraulic gradient at 0.2% (*i.e.*, a maximum drop of 2m in head at a distance of 1 km) at the selected recharge site.

Vertical Hydraulic Gradient. Vertical hydraulic gradient across different recharge zones in an injection well causes loss of recovery efficiency. This is due to imbalance in the intake rate and the production rate in these zones, created by the hydraulic gradient between zones. There are, however, only a few data on the vertical hydraulic gradient prevailing at different sites in Kuwait and these are mainly concentrated in the central and southwestern parts of Kuwait. The problem caused by the vertical hydraulic gradient can be avoided, however, by confining the recharge within one aquifer unit with more or less uniform potentiometric head. It is expected that before completing the ASR wells, data concerning vertical hydraulic gradient between different aquifer units will be collected and evaluated and the injection zone will be selected judiciously so that high vertical hydraulic gradient across the zone will be eliminated.

Operational and Economic Considerations

Apart from the hydrogeological factors, available facilities and utilities at the selected recharge site are also to be considered for operational and economic reasons. The recharge site should be easily accessible and should be either served by electricity and water distribution network or else, be near enough to these utilities so that cost of laying pipelines for water conveyance and electrical network for providing energy for pumping water to and from the site is minimized. The existence of a major road network (dual carriage highways) has been used as a combined indicator for the existence of such facilities in the present case, as generally, water and electrical network follow the major road network. It has been assumed that an area within 20 km of a major road on its either side will be suitable for locating an artificial recharge site. The water table or potentiometric head at the recharge site should not also be too deep either (like ≥ 150 m below the surface), as that will increase the cost of recovering injected water during the time of need.

Selection of the Recharge Sites

General

Based on the criteria enumerated in the previous section, the selection of sites suitable for artificial recharge in the Kuwait Group and the Dammam Limestone aquifers was carried out using the hydrogeological database, recently created for the State of Kuwait, and the facilities provided by the ARCGIS system. The information available in the database have been provided by Mukhopadhyay *et al*. (2005). The details of the steps followed are presented here.

Extraction of Basic Data

The available data on transmissivity, total dissolved solids content and depth to water were extracted from the Microsoft ACCESS hydrogeological database for Kuwait through the formulation of appropriate queries.

Analysis of Data by ArcMAP (an ARCGIS Component)

The tables created under MS-ACCESS facility were analyzed by ArcMAP for the selection of suitable recharge sites, utilizing the criteria set forth above. At the outset, the Geostatistical Analyst (containing tools for geostatistical analyses), Maplex (facilities for generating maps with labels) and the Spatial Analyst (for creation, querying, mapping, vector-data integration and analysis of cell-based raster data) extensions of ArcMAP were enabled to carry out the required analyses. The previously created Water fields, political boundary, soil map and Dual Highway layers for Kuwait, saved in the hydrogeological geodatabase of Kuwait on the KISR central server, were imported in ArcMAP. The newly created tables for transmissivity, TDS, depth to water and potentiometric head for the target aquifer (Kuwait Group or the Dammam Formation) were added to these layers. With the help of the Geostatistical Wizard, the concerned parameter (transmissivity, TDS, depth to water or potentiometric head) in these tables were interpolated using ordinary kriging for the creation of a surface map for each of these parameters. The potentiometric head surface map was also used as the source data for the computation of slope (%) for each cell (500 m x 500 m) with the help of the Surface Analysis tool under Spatial Analyst. The surface maps were converted to raster grid files with cell sizes of 500 m x 500 m by exporting the maps to Raster. A 20-km corridor on either side of the main dual highways of Kuwait was calculated in Raster grid format (dst_20_hiway) with the same cell size using Straight Line distance calculation facility under Spatial Analyst.

Once all the raster grid maps for transmissivity, TDS, depth to water, lateral potentiometric gradient and 20-km corridor on both sides of the major dual highways were ready, the Raster Calculator was invoked under Spatial Analyst. Following expressions were used in the Raster Calculator to select suitable areas for artificial recharge in each of the aquifers, based on the selection criteria established above.

<u>Expression for selecting suitable areas for artificial recharge in the Kuwait Group Aquifer</u>

[Calculation] = ([grd_avg_wdep] <= 150 & [grd_avg_wdep] >= 50) * ([grd_avg_kgtrn] <= 1000 & [grd_avg_kgtrn] >= 250) * ([grd_avg_kgtds] <= 5000) *([kgwhgtslope_p] ≤ 0.2) *([dst_20_hiway]) (Exp. 1)

<u>Expression for selecting suitable areas for artificial recharge in the Dammam Formation</u>

[Calculation] = ([grd_avg_wdep] <= 150 & [grd_avg_wdep] >= 50) * ([grd_avg_dmtrn] <= 500 & [grd_avg_dmtrn] >= 150) * ([grd_avg_dmtds] <= 5000) *([dmwhgtslope_p] ≤ 0.2) *([dst_20_hiway]) (Exp. 2)

where

grd_avg_wdep: Average depth to water in a grid block

grd_avg_kgtm: Average transmissivity of Kuwait Group aquifer in a grid block

grd_avg_kgtds: Average TDS contents of Kuwait Group aquifer in a grid block

kgwhgtslope: Average potentiometric surface slope of the Kuwait Group aquifer in a grid block

grd_avg_dmtm: Average transmissivity of Dammam Formation aquifer in a grid block

grd_avg_dmtds: Average TDS contents of Dammam Formation aquifer in a grid block

dmwhgtslope: Average potentiometric surface slope of the Dammam Formation aquifer in a grid block

dst_20_hway: A logical variable indicating whether a grid block is within 20-km distance from a dual-carriage highway

and

[Calculation] is the logical output (either 1 or 0) of (Exp. 1) or (Exp. 2) indicating whether a grid block is suitable (indicated by 1) or not (indicated by 0) for artificial recharge in the Kuwait Group or Dammam Formation, respectively.

The selected areas were further graded according to the degree of match of the different selection criterion to the range of values specified. The Raster calculations utilized for this purpose are as follows:

<u>Calculations for grading of the suitable areas for artificial recharge in the Kuwait Group</u>

[Calculation] * (([grd_avg_kgtds] <= 4000) + ([grd_avg_kgtrn] <= 1000 & [grd_avg_kgtrn] >= 500) + ([grd_avg_wdep] >= 50 & [grd_avg_wdep] <= 100) + ([kgwhgtslope_p] <= 0.1)) (Exp. 3)

Calculations for grading of the suitable areas for artificial recharge in the Dammam Formation

[Calculation] * (([grd_avg_dmtds] <= 4000) + ([grd_avg_dmtrn] <= 500 & [grd_avg_dmtrn] >= 250) + ([grd_avg_wdep] >= 50 & [grd_avg_wdep] <= 100) + ([dmwhgtslope_p] <= 0.1)) (Exp. 4)

The grading assigns highest score (4, most suitable, when the grid block has passed through the original screening by Exp. 1 or Exp. 2 with a value of 1 for the output, [Calculation] and each of the logical expressions within brackets return a value of 1) to the areas that have met all the above conditions i.e., total dissolved solid contents of less than 4000 mg/l, transmissivity values in the range of 500 - 1000 m^2/d for the Kuwait Group and 250 - 500 m^2/d for the Dammam Formation, depth to water in the range of 50 - 100 m below the ground and lateral hydraulic gradient ≤ 0.1% (1 m per 1 km). The score reduces as one or more of these conditions are not met. Thus, an area with a score of 3 where three of the four conditions have been met has been graded as 'fairly suitable'; an area with a score of 2 where two of the conditions have been met has been graded as 'moderately suitable'; and an area with a score of 1 where only one of the above conditions have been met has been graded as 'least suitable'. The results of these grading are presented in Figs. 2 (Kuwait Group) and 3 (Dammam Formation). However other considerations like the size of the areas selected and their current land use, groundwater pollution potential and aquifer vulnerability, distance from the main population centers, actual availability of water conveying pipeline(s) and electrical network, distance from source of water for injection, electrical energy supply, and other similar factors are to be taken into account before a final selection is made. On a preliminary basis, based on the grading and considerations mentioned above, three areas, prioritized as AR-KG-1, AR-KG-2 and AR-KG-3 have been demarcated on Fig. 2 as suitable for recharge in the Kuwait Group Aquifer. For the Dammam Formation, only one area, AR-DM-1, between Kabd and Umm Gudair water fields appears suitable for recharge (Fig. 3).

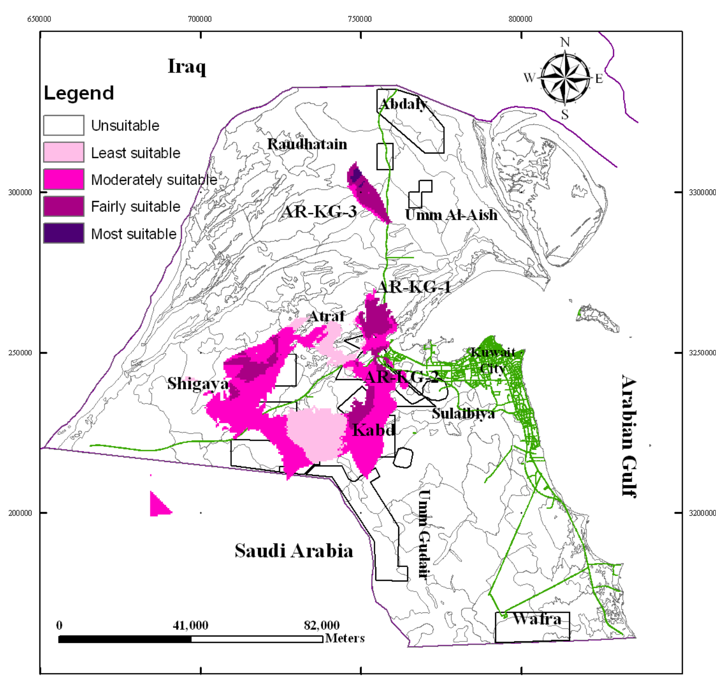

Fig. 2. Suitability gradation of areas selected for artificial recharge in the Kuwait Group aquifer.

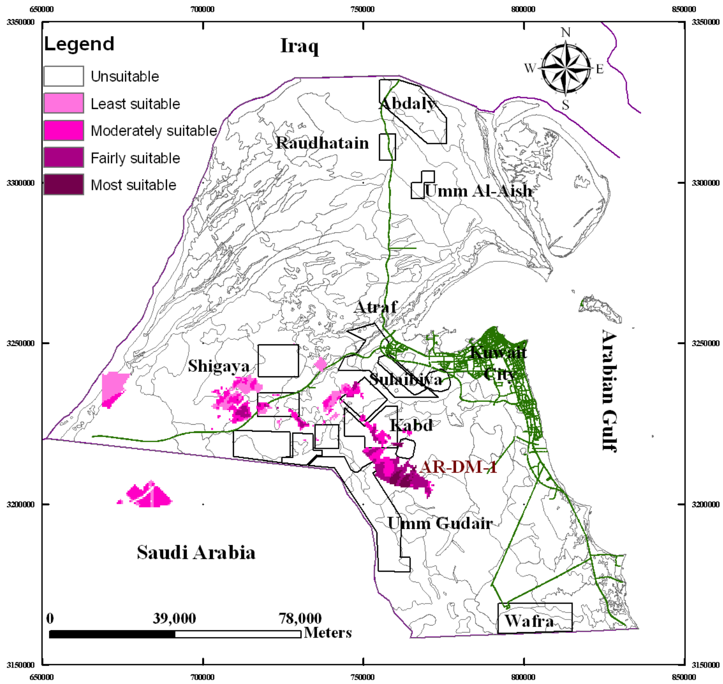

Fig. 3. Suitability gradation of areas selected for artificial recharge in the Dammam Formation aquifer.

Conclusions and Recommendations

Based on the distribution pattern of their hydrogeological characteristics an attempt has been made for the selection of suitable areas for artificial recharge in the Kuwait Group and the Dammam Formation, the two aquifers providing useable brackish groundwater in Kuwait. The help of the ARCGIS system in analyzing the relevant data and information stored in the relational hydrogeological database developed for Kuwait has been taken in the application of the multiple criteria developed for the selection of the suitable sites. Considerations have also been given to other non-hydrogeological criteria like distance from the major highways and population centers in choosing the most suitable sites. After careful considerations of all available data and information, one site near Kabd area (AR-DM-1) has been selected for recharge in the Dammam Formation aquifer. For the Kuwait Group aquifer, three areas have been delineated as suitable, the most favorable of which is the Mutla area, designated here as AR-KG-1, followed by the Sulaibiya area designated as AR-KG-2, and the least favorable one is located to the southwest of the Raudhatain freshwater field designated as AR-KG-3. It is recommended that further on-ground verification of the selected areas for the current use for other activities, actual existence and distance of water conveying pipelines and electrical network and practical considerations like topographic constraints is undertaken before the final selection of one of these areas for pilot scale recharge.

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ASR as a Strategic Resource -
Challenges in the Arabian Gulf Region

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Topic: MAR in Arid Environments

Keywords: ASR, strategic drinking water resource, Arabian Gulf, scaling, location groundwater protection zones.

Abstract

The countries along the Arabian Gulf are undergoing a rapid economical development. Major coastal cities depend on desalinated seawater as main drinking water supply. They are vulnerable to interruption of the water supply due to technical failure and other unforeseeable events. Aquifer Storage and Recovery (ASR) can provide a reliable strategic resource. This paper discusses planning aspects that are of special interest for the implementation of a strategic drinking water resource in the Arabian Gulf Region. Focus is set on scaling, location and protection of the ASR site. The paper concludes that strategic drinking water storage is feasible within the arid environment of the Arabian Gulf. Successful implementation requires synchronization with local water suppliers and regulatory bodies.

Introduction

The countries along the Arabian Gulf coast have experienced tremendous economic growth over the last decades. Regional infrastructure is continuously extended and upgraded to keep up with the pace of urban growth, agricultural expansion and industrial developments. To guarantee for uninterrupted drinking water supply within this arid region, large-scale seawater desalination plants have been implemented. This dependence on artificially produced drinking water makes the economies vulnerable to failure of these plants. Current drinking water reserves do not exceed five days in most Gulf Countries (Kamel et. al. 2008). The creation of reliable water storage is vital for the development of these countries. As viable and cost-effective alternative to water storage in surface facilities, the implementation of large strategic drinking water resources by means of Artificial Storage and Recovery plants (ASR) is in its initial phase, with the most advanced country being the United Arab Emirates. The implementation of a strategic resource is confronted with a variety of challenges that are specific for the unique economic and environmental conditions in the Gulf Countries.

This paper discusses challenges of implementing a strategic resource that shall bridge the time gap starting with the failure of the classical public supply system to its repair on a level that it can provide for the defined water need of the population. The main objectives of the planning of a strategic resource are the scaling and the provision of a suitable location. The respective methodologies are discussed in the following.

Methods

Project Scaling – Forecast of Demand

A strategic resource cannot be planned based on the current demand. Instead, it should take account of the future demand for the next decades to come. The future water demand mainly depends on the projected population growth and the specific per capita supply rates. Most Gulf Countries are carrying out regular census of the population that allows for a good estimate on the current population number and the historic growth rates (World Bank 2010). Broadly, two growth rate patterns occur, as can be seen from Figure 1 below. The first group shows high, however, strongly oscillating, growth rates, while the latter group has stabilized their growth rates during the last decades to relatively low growth rates after the boom of the commencing oil era.The first group consists of smaller countries with high rates of foreign labour forces like U.A.E. and Qatar, while the second group consists of countries with a high percentage of native work force like Saudi Arabia and Oman. Apparently, the prediction of the population to be supplied in the future is difficult for the countries showing a high variation in their growth rates.

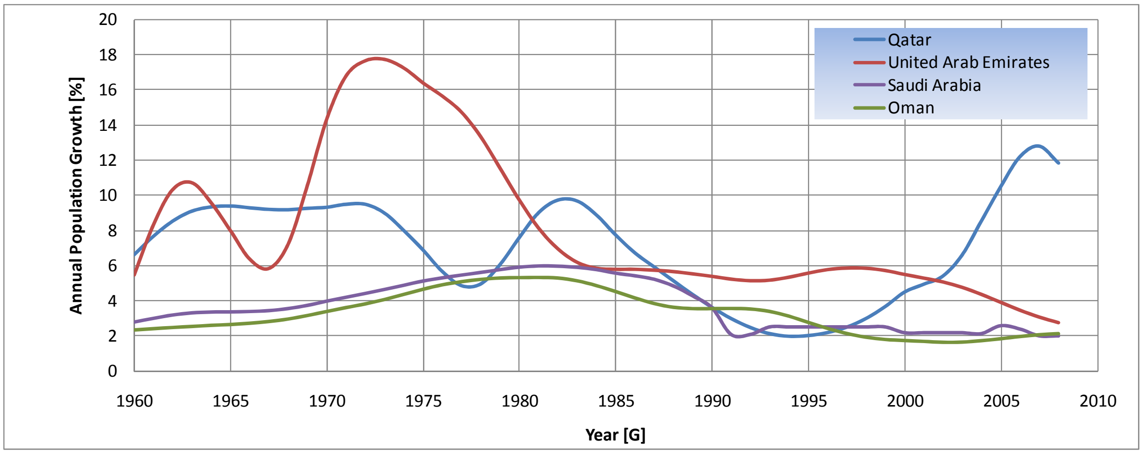

Figure 1: Variation in population growth rates for selected Countries (World Bank 2010)

A strategic resource should be scaled in a way that the social and economic life can continue in the emergency case. Thus, the per capita water need does not depend on the mere drinking water requirement, but should take account of the need for hygiene and of small businesses and public entities. Regarding landscaping, which is a significant water consumer in arid urban environments, the city of Abu Dhabi, for instance, is targeting a 100% water reuse policy to supply their parks and green areas already with treated sewage effluent (Anderson 2008). Other urban areas are following this trend. Thus the water need for urban green or industrial use may not add to the scaling of the strategic resource, in most cases.

The total water need depends further on the specific per capita consumption. Per capita water need is not the same as current water demand. In an ideal case, the water need, in the emergency case, for an urban area in Gulf State could be on the level of the Middle European household consumption, as low as 124 l/c/d (BDEW 2008). This is much lower than the current consumption rates in the Gulf States of around 240 l/c/d to 520 l/c/d (FAO 2010), which include water demand for landscaping.

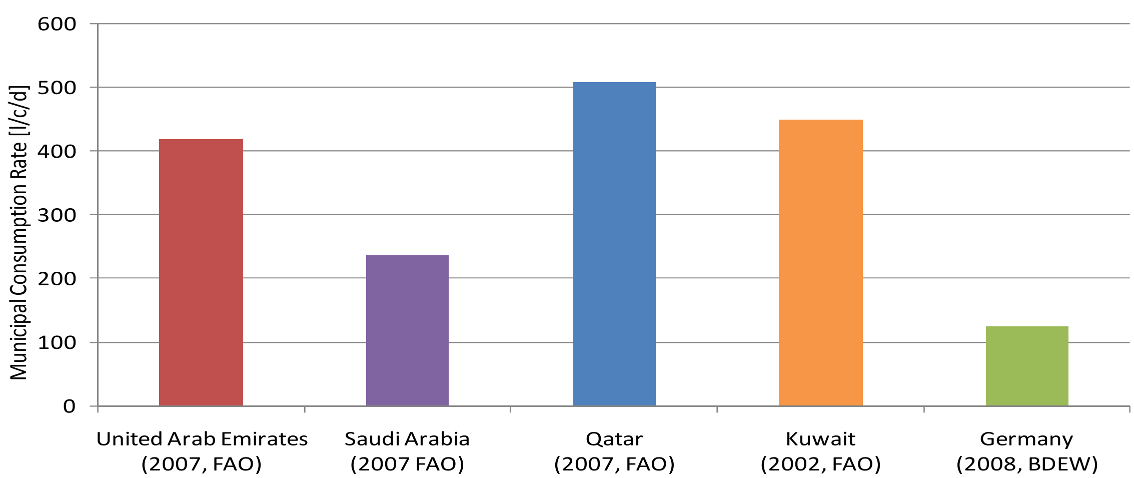

Figure 2: Municipal Water Consumption for Selected Countries (Source: Modified after FAO 2010, BDEW 2008, note for U.A.E., KSA and Qatar data for municipal water demand and population were from different years, adjustment factors were added based on annual population growth data by World Bank 2010))

It is typical for the discussed region that the rapid development of the urban areas in Gulf States has put stress on the quality of the water infrastructure. Extensive water losses are common in many cities

(Bhalas 2009) and are visible to the public as they can trigger additional problems like rising groundwater levels (Al-Hajri et al. 1992, Amer 2008). Thus, water network losses may lead to a somewhat higher water need in respect to the cited Middle European rates and have to be included in the computation of the water demand for strategic resource.

Project Scaling – Time scale

As discussed above, the main water supply source for the cities along the Gulf Coast is seawater desalination. Seawater desalination plants are vulnerable to mechanical failure as any industrial plant. However, all Gulf countries supply their population from several desalination plants (ADWEC 2010, ESCWA 2001, World Bank 2005); failure of only one plant may not trigger an emergency. More severe effects may be caused by an environmental catastrophe like oil spills or red tides if the seawater intake of desalination plant is unprotected (direct seawater intake). In such a case, the interruption may last up to several months. Thus, a strategic resource should be scaled to cover a couple of months, including a safety margin even up to a year. This is a time frame currently being implemented in the U.A.E. (Dawoud 2005, GSEC 2009).

Handling Uncertainties in Project Scaling

The discussed uncertainties regarding future population growth and water demand makes it imperative to synchronize the applied demand figures between the planners of the strategic resource and the major infrastructure and supply entities in the respective country to avoid divergence in the scale of the future planning. Uncertainties should be accepted as integrative part of the planning process. Consequently, the potential for up-scaling of the system should be included in the planning process by selecting an ASR site that offers capacity for extension. If this is not possible, a back-up site for future extension may be identified. The extension zones should benefit from the same protection scheme as the main implementation site and should be ear-marked as protected zone in the long-term planning of the land development. From the technical point of view a modular recharge and recovery system may be of advantage, which allows for capacity adjustments by adding additional modules or even reducing capacity if the water demand decreases.

Site Location

Regarding the choice of the ASR site, the specific configuration of water supplier, demand centre and the location of a suitable aquifer for storage are the main parameters. The main water suppliers for an ASR project in the region under discussion are the seawater desalination plants along the Arabian Gulf Coast. In Gulf States like the United Arab Emirates the State of Qatar or the Kingdom of Bahrain, seawater desalination has become by far the main provider of drinking water over the last decades (Abdulrazzak 1997, Hamouda 2001).

More than 50% of the large desalination plants are designed to cogenerate drinking water and electricity (IEA 2005). In case of technical failure, not only the drinking water production but also parts of the electrical power grid supplying the water pumping stations may break down. Water conveyance requires energy, which has to be supplied by back-up systems in the emergency case. The shorter the conveyance distance, the more reliable the ASR system can be operated as it reduces the need for independent power supply. In addition, as both water suppliers and most drinking water demand centres are located along the coast, it would only be reasonable to locate also the ASR site within an aquifer in the coastal areas. On a regional scale, the Tertiary aquifer system consisting of the Neogene, the Dammam and Umm Er Radhuma aquifers are the first choice, both in terms of their depths as of their thickness (MAW 1984). These aquifers are part of the sedimentary platform of the Arabian Peninsula and dip to the east from Saudi Arabia towards the other Gulf countries (Alsharhan et al. 2001). They can be mainly characterized as fractured and karstic limestone aquifers. Along the northern and central coastal areas the Dammam aquifer and to some degree the underlying Umm Er Radhuma, can provide a water resource of fresh to brackish water quality (World Bank 2005). In addition, some shallow aquifers exist that are of local importance due to their high water quality, for instance along the Omani Mountains in the U.A.E. and in the Western Region of the Abu Dhabi Emirate (Brook et al. 2006). Figure 3 provides an overview of the main urban water demand centres and the water quality of the discussed aquifers.

However, directly at the coastline, and in the deeper aquifers water quality tends to become highly brackish to saline in both the deep and shallow aquifers (Alsharan 2001, Kamel et al. 2003, BGR UNESCO 2009). A fact that, together with the hydrogeological conditions, may reduce the recovery efficiency of the system, especially for low-maintenance, long-term storage.

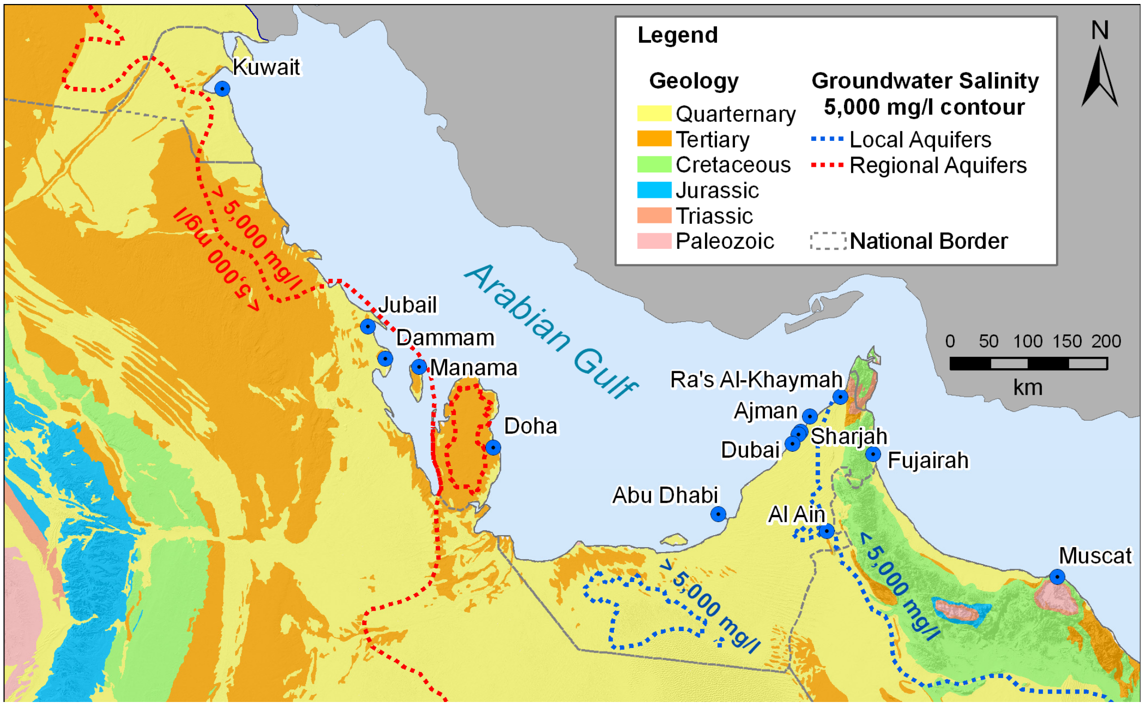

Figure 3: Overview of geology and water quality along the Arabian Gulf (Source: Modified after BGR UNESCO 2009, Brook et al 2006, USGS 1963).

Thus, ASR sites for strategic resources may have to make do with locations in the inland as being planned in Abu Dhabi Emirate (GSEC 2009) and proposed for Qatar (Streetly & Kotoub 1998). The cost for the construction of the conveyance system can be kept on an acceptable level by using existing infrastructure, wherever possible.

Impact of Future Landuse

As for the water demand estimate, locating the strategic water resource should not only consider the current settings but also on the future development. Urban and industrial areas spread rapidly around existing cities. New industrial and urban centres are under planning along the Gulf Coast, a representative example being the Abu Dhabi Capital City development (DoT 2009) in Abu Dhabi Emirate. Predictive groundwater modeling studies by researchers of the King Fahd University (Abderrahman et al 2007) for the Dammam aquifer in the urban areas of Dammam and Al Khobar predict not only a further extended cone of depression within the aquifer but also a rotation of the groundwater flow direction due to ongoing abstraction. Such a development would impact any ASR system in the affected area.

In addition, the largest groundwater consumer in most Gulf countries, agriculture, may extend existing cones of depression due to ongoing groundwater abstraction (World Bank 2005), potentially putting strategic groundwater resources at risk. As side effect, groundwater quality decreases as consequence of overexploitation by agricultural use as documented for Qatar (Kamel et al 2008, Amer 2008).

Coping with Conflicting Landuse – Groundwater Protection Zones

The implementation of groundwater protection zones in the urban, industrial and agricultural planning may mitigate or even prevent the processes described above. The establishment of groundwater protection zones around strategic resource locations are an instrument to regulate the two potential utilisation conflicts (resource protection versus agricultural and municipal demand). A groundwater protection zone protects the area of influence of the strategic resource by law similar to a national park. The size of the groundwater protection zone can be estimated using groundwater flow models under application of the predicted water abstraction over the period of the lifespan of the strategic water resource. Its size and the restrictions applied may vary depending on the local characteristics of the hydrogeology. Groundwater protection zones are generally subdivided in different zones of increasing restriction in use in dependence of their distance to the well head (KOSCHEL et al 1996). In adaptation for strategic storage ASR sites it may be formulated as follows:

- Wider protection zone (Zone III): Provides protection from chemical contamination and groundwater abstraction.
- Nearer protection zone (Zone II): Provides protection from microbiological and viral contamination by natural attenuation within the aquifer.
- Well head zone (Zone I): Highly restricted use, protection of immediate surrounding of ASR well.

It should be stated here that agricultural development and groundwater resources protection may not exclude each other, if groundwater protection zones are respected and alternative water supply like reclaimed water is used or high-efficiency greenhouse farming applied. In case that existing agricultural landuse has to be restricted within a protection zone, payments for the financial losses are a viable solution as common practice in countries like Germany, where compensation in different states may range between approximately 10 Euros up to around 350 Euros per hectare and year (Anlauf 2009, BOR 2010). The proposed protection scheme may need adaptation of the legal environment in the respective country, underlining the need for integration of all relevant stakeholders in the planning process for a strategic drinking water resource.

Conclusions

The creation of a strategic drinking water resource in the countries adjoining the Arabian Gulf Coast has to meet several, regionally specific challenges, of which the highest lies in the prediction of future water demand and the identification of a location that will remain suitable for aquifer storage also within the rapidly environment of rapidly expanding urban, industrial and agricultural developments.

To keep up to these challenges a design approach for a scalable ASR scheme is recommended. The process of the planning and implementation of a strategic resource using the ASR technique should be taken as opportunity for the introduction and enforcement of the concept of groundwater protection zones.

The need for collaboration and synchronization of planning and implementation between planners, the responsible national authorities and water suppliers is key to a successful implementation of a long-term strategic resource. Large-scale ASR projects should be embedded in the overall water sector planning of a country to synchronize and optimize the implementation of different infrastructure projects.

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EFFECTS OF ARTIFICIAL RECHARGE VIA INJECTION WELLS ON GROUNDWATER QUALITY IN A SHALLOW ALLUVIAL AQUIFER, CENTRAL DAMASCUS BASIN

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Abstract
Isotopes and hydrochemistry have been applied to study the effects of artificial recharge via injection wells on the groundwater quality of the shallow Quaternary alluvial aquifer in Damascus basin. The injectant water, entirely taken from the admixed surplus fresh water of Figeh Springs, was diverted to the injection well without any treatment. The total volume of injected water during the study period (2006-2008) amounts to 0.24156 Million m^3, whereas the total volume of pumped water from this site was ≈12.333 M m^3. Water samples, taken from the ambient groundwater of three observation wells drilled in the vicinity of injection well, show that nitrate ($40<NO_3^-<57$ mg/L), sulphate ($180<SO_4^{2-}<270$ mg/L) and TDS (940<TDS<1155 mg/L) were the chemicals of concern, as their concentrations exceed the established standards given for drinking water. After injection, nitrate concentration substantially decreased to values around 20-30 mg/L, sulphate dropped down to ≈100 mg/L, and TDS decreased to levels close to the permitted maximum level for drinking water. The significant difference in ^{18}O concentration between the injectant water and ambient groundwater revealed that this natural tracer is useful and very sensitive for accurate estimate of water mixing. Relationships between the different parameter values and total injected water volumes show slight increases in the case of pH and dissolved O_2. With the exception of HCO_3^-, which shows the worse correlation (R^2=0.005), the remaining parameters show remarkably good correlations, especially chloride (R^2=0.995) and ^{18}O (R^2=0.992). Although the climatic conditions prevailing in the area during the study period did not much help to inject further amounts of water, the findings were encouraging for continuing this artificial recharge scheme, as an alternative water management method, essentially for raising the potential groundwater reserve and improving the groundwater quality of this aquifer.

Keywords
Artificial recharge, aquifer storage and recovery, hydrochemistry, stable isotopes, Damascus basin.

INTRODUCTION

The technique of aquifer storage and recovery (ASR), as a storage scheme of excess surface water underground, via special wells, has proved to be a useful water management option (Pyne, 1995 and 2002). This scheme of artificial aquifer recharge, based on specially designed wells suitable for high injection rates during storage process and ease of recovery of the stored water when it is needed dates back to the 19th Century and has become now more and more popular in many parts around the world (Dillon, 2002; UNEP, 2002; NNC-IAH, 2003). However, this technique operates more successfully where groundwater salinity is limited to about 1 g/L in Cl^- concentration and to less than 10 m/a of groundwater velocity (Tuinhof et al., 2003).

Water shortages in most Arabian countries necessitates collection of every drop of precipitation. Nevertheless, the ASR technique was only recently introduced to this region. Morocco was the first country to apply such a water management method in 1958 (ACSAD, 1995). Later, this technique was introduced to most of the Arabian Gulf countries (UNEP, 2002), and to a lesser extent to Jordan and Syria (University of Jordan, 1993; Malakany, 1999), but with a main objective that is to test the feasibility and effectiveness of ASR scheme under such dry environments (Rasoul Agha, 1994).

Water supply to the city of Damascus represents a big challenge for the government during the 21st Century, and a major concern for the Damascus City Water Supply and Sewerage Authority (DAWSSA), responsible for finding out sufficient water resources in terms of quantity and quality. As most of the water supply to this city essentially comes from the famous Figeh Springs (average discharge rounded 7.7 m^3/s, Fig. 1), this source of water becomes insufficient to meet the increasing water demand (Fallouh, 2004). Therefore, the ASR technique was proposed as an alternative method to meet future water supply demands.

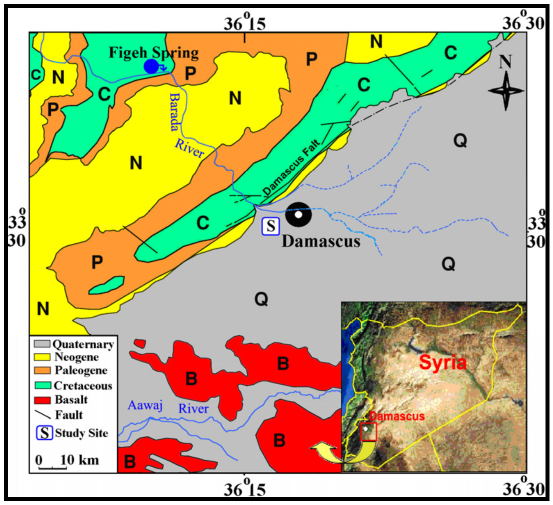

Figure 1. Geological map of the Damascus sheet showing location of the study site.

Accordingly, a number of injection and observation wells were drilled in several injection fields within the shallow alluvial Quaternary aquifer, that are all completely managed by the DAWSSA. For the purpose of this specific study the investigations were focused on a single site (Fig. 1) located in central Damascus basin precisely inside the Damascus University Campus (DUC). The main objective of this paper is to present the results obtained regarding the groundwater quality changes of the shallow Quaternary alluvial aquifer that occurred as a consequence of three injection events conducted during the period 2006-2008.

SITE DESCRIPTION

The selected site is located in the Damascus basin at an average altitude of about 730 m above sea level, where the climate is generally of a Mediterranean type, and rainfall averages ≈212 mm/a. This site contains a sufficient number of production and observation wells (≈22), together with a single injection well (IW), with a diameter of 80 cm and a depth of 45 m below the land surface (Fig. 2). The injection well is fully equipped for artificial recharge purposes, and thus it is associated with three injection pipes permitting water injection at different depths. The main advantage of such pipes is to eliminate the formation of air bubbles and facilitate the water penetration within the aquifer away from the injection well. The well IW can also be used outside the injection periods for pumping the stored water mostly during extended dry periods. The remaining wells were drilled to depths ranging from 30 to 65 m, within the shallow Quaternary alluvial aquifer, consisting of pebble, gravel, loam and sandy clays, that change laterally to pebble conglomerates with carbonate and clay cement or to lenses of clayey sand, with a percent of sand from top to bottom.

MATERIAL AND METHODS

Four monitoring wells were used in this study, and consist of the injection well (IW) and another three observation wells (W13, W14 and W15) located in the vicinity of IW (Fig. 2). The well W15, which is located westward the well IW, was selected to act as unaffected upgradient reference site. The injection water is taken from the admixed surplus water of Figeh Springs during flood periods (March to May), that amounts to an average of approximately 90 million m^3 (Fallouh, 2004). Historically, this huge flow of very fresh water (EC<300 μS/cm) was diverted to the Barada River, which is usually polluted and more saline. Recharge of the alluvial aquifer began in March 2006 and continued during 2007 and 2008. Because of the low content of suspended matter and organic compounds in the Figeh water, this water was diverted for injection without any treatment, and injected at a rate of 20-90 m^3/h. The total amount of injected water in the DUC site during the study period (2006-2008) was approximately 241557 m^3, distributed as follows: 61250 m^3, 86892 m^3 and 93415 m^3, for 2006, 2007 and 2008, respectively. Whereas, the groundwater amount pumped from this site during the same period was remarkably large (12.333 M m^3), and thus, the injected water ratio was less than 2% of the total pumped volume.

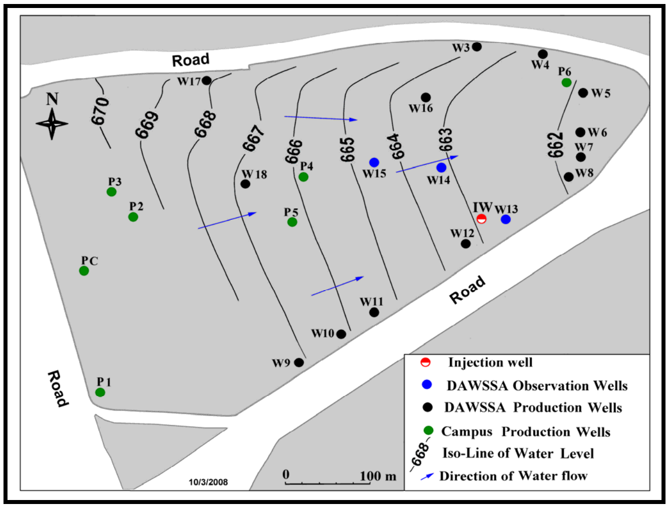

Figure 2. Location map showing the different wells in the DUC ASR site, together with the groundwater piezometric levels measured on 10 March 2008.

More than 145 water samples were collected from the injectant Figeh water and groundwater of three different DUC observation wells (W13, W14 and W15) during a three years period from April 2006 to September 2008. Water samples were generally analyzed for major ions (Ca^{2+}, Mg^{2+}, Na^{+}, K^{+}, Cl^{-}, SO_4^{2-} and NO_3^{-}) and stable isotopes ($\delta^{18}O$ and $\delta^{2}H$) using conventional analytical procedures described in more details by Kattan (2010). The related diverse analyses were carried out in the Geology Department of the Atomic Energy Commission of Syria (AECS), where quality assurance procedures, according to the ISO/IEC 17025, are highly respected. The water temperature, pH, electrical conductivity (EC), dissolved oxygen (DO_2), together with the total alkalinity (i.e. HCO_3^{-}) values of all water samples were measured in the field during sampling.

CHEMICAL AND ISOTOPIC COMPOSITIONS OF INJECTANT WATER

Table 1 summarises the averages and standard deviations (± values) of the hydrochemical and isotopic data of the injectant water in the DUC site calculated for the three injection events during the period 2006-2008, together with the amount of injected water during each event.

Table 1. Mean hydrochemical and isotopic data injectant water in the DUC site calculated for the three injection cycles during the period 2006-2008.

Parameter	event of 2006	event of 2007	event of 2008
Injected water volume (m^3)	61250	86892	93415
T (ºC)	15.9±1.6	13.0±1.8	18.3±1.8
pH	7.66±0.1	7.97±0.1	8.02±0.1
E.C (µS/cm)	256±14	231±16	264±14
DO_2 (mg/L)	6.49±0.6	9.62±1.6	8.33±0.9
Na^{+} (mg/L)	1.9±0.1	2.2±0.2	2.4±0.1
K^{+} (mg/L)	0.4±0.05	0.5±0.05	0.5±0.1
Mg^{2+} (mg/L)	8.3±1.4	10.5±1.0	8.8±0.9
Ca^{2+} (mg/L)	40.6±1.3	39.7±4.2	43.6±4.0
HCO_3^{-} (mg/L)	151.8±14.2	156.1±12.6	166.7 ±7.6
Cl^{-} (mg/L)	3.3±0.5	4.5±0.9	3.6±0.7
NO_3^{-} (mg/L)	4.7±1.0	7.9±0.8	7.1±0.5
SO_4^{2-} (mg/L)	3.5±0.4	6.5±1.5	5.8±0.9
TDS (mg/L)	214±18	228±16	238±4.0
$\delta^{18}O$ (‰, VSMOW)	-8.90±0.1	-8.64±0.1	-8.63±0.1
$\delta^{2}H$ (‰, VSMOW)	-49.46±1.7	-48.14±0.7	-48.78±0.2

The data shows that the injectant water was generally fresh (TDS <240 mg/L), and clearly of low concentrations in most major ions. The pH (7.66-8.02±0.1) and DO_2 (6.5-9.6±0.1) values reflect the lithological nature of the aquifer system, which belongs to the Cenomanian-Turonian complex, mostly composed of thick karstified strata of dolomite, dolomitic limestone and limestone (La-Moreaux, 1989). The reason that DO_2 concentration in the injectant water is significantly higher by ≈2±1 mg/L compared with the DUC ambient groundwater is probably because of the colder water emerging from such a karstified system has had more contact with the atmosphere.

The concentrations of both $\delta^{18}O$ (-8.90 to -8.63 ±0.1 ‰) and $\delta^{2}H$ (-49.5 to -48.1 ±1 ‰) in the admixed Figeh water (injected water), were slightly depleted compared with the other groundwaters in the Damascus basin (Kattan, 2006). However, these concentrations were in good agreement with the previous values given by Kattan (1997) for the Figeh main spring (≈-9.02±0.16‰ and ≈-52.5±1.1‰, for $\delta^{18}O$ and $\delta^{2}H$, respectively) and Figeh side spring (≈-8.52±0.36‰ and ≈-47.8±1.7‰, for $\delta^{18}O$ and $\delta^{2}H$, respectively). The reason that the Figeh Springs dispose relatively depleted isotopic values is simply because of the higher elevation of recharge zones, exceeding ≈1750 m above sea level in the case of Figeh main spring, and ≈1500 m in the case of Figeh side spring (Kattan, 1997).

CHEMICAL AND ISOTOPIC COMPOSITIONS OF AMBIENT GROUNDWATER

Average values of the hydrochemical and isotopic parameters of groundwater samples collected from the DUC site during the period 2006-2008 are compiled together with the standard deviations (± values) in Table 2. The data shows that the groundwater quality in all wells (TDS>750 mg/L) are significantly higher by a factor of 3.7, at least, compared with the injectant Figeh water. The reason that the groundwater in the DUC site is brackish, and especially of more nitrate and sulphate concentrations ($35<NO_3^-<49$ mg/L and $55<SO_4^{2-}<209$ mg/L), is likely because of the anthropogenic pollution that affects the groundwater in the Damascus basin. Noting that this basin is generally recharged by water from the Barada River through direct infiltration from the riverbed, or through penetration of irrigation water, mostly taken from this river, which is usually polluted and of more mineralized water (Kattan, 2006).

Table 2. Mean hydrochemical and isotopic data of the groundwater samples collected from the different DUC observation wells (W13, W14 and W15) during the period 2006-2008.

Parameter	W13	W14	W15
T (ºC)	17.2±0.5	18.2±0.4	17.7±0.6
pH	7.2±0.2	7.1±0.1	7.1±0.2
E.C (µS/cm)	976±233	1213±155	1262±82.4
DO_2 (mg/L)	5.7±0.8	5.0±0.7	4.9±0.7
Na^+ (mg/L)	71.1±20.5	91.4±24.0	105.0±16.3
K^+ (mg/L)	3.9±0.7	4.8±1.1	3.9±1.1
Mg^{2+} (mg/L)	26.0±5.4	31.8±2.9	34.0±3.2
Ca^{2+} (mg/L)	107.0±21.7	129.9±11.4	134.2±13.2
HCO_3^- (mg/L)	290±38.0	318±24.0	291±27.0
Cl^- (mg/L)	88.3±34.8	125.4±28.9	159.0±20.1
NO_3^- (mg/L)	35.5±10.5	48.6±7.0	45.9±6.3
SO_4^{2-} (mg/L)	146.5±43.5	185.2±50.1	209.1±37.6
TDS (mg/L)	768±156	934±112	982±107
$\delta^{18}O$ (‰, VSMOW)	-8.31±0.2	-8.23±0.2	-8.20±0.1
$\delta^{2}H$ (‰, VSMOW)	-47.1±1.5	-47.2±1.5	-47.0±1.5

TEMPORAL VARIATIONS OF INJECTANT WATER AND AMBIENT GROUNDWATERS

Figure 3 illustrates the temporal variations in EC values, and $\delta^{18}O$, SO_4^{2-} and NO_3^- concentrations of the injectant water and ambient groundwaters of the different observation wells during the study period (March 2006-June 2008). This plot shows clearly remarkable decreases in all parameter values during injection events, mostly the groundwater of wells W13 and W14. These decreases reflect the dilution THAT occurs as a result of mixing between the fresh injectant water and the brackish ambient groundwater. The higher dilutions that can be observed in the case of well W13 compared with that of well W14, suggest more effectiveness by water injection.

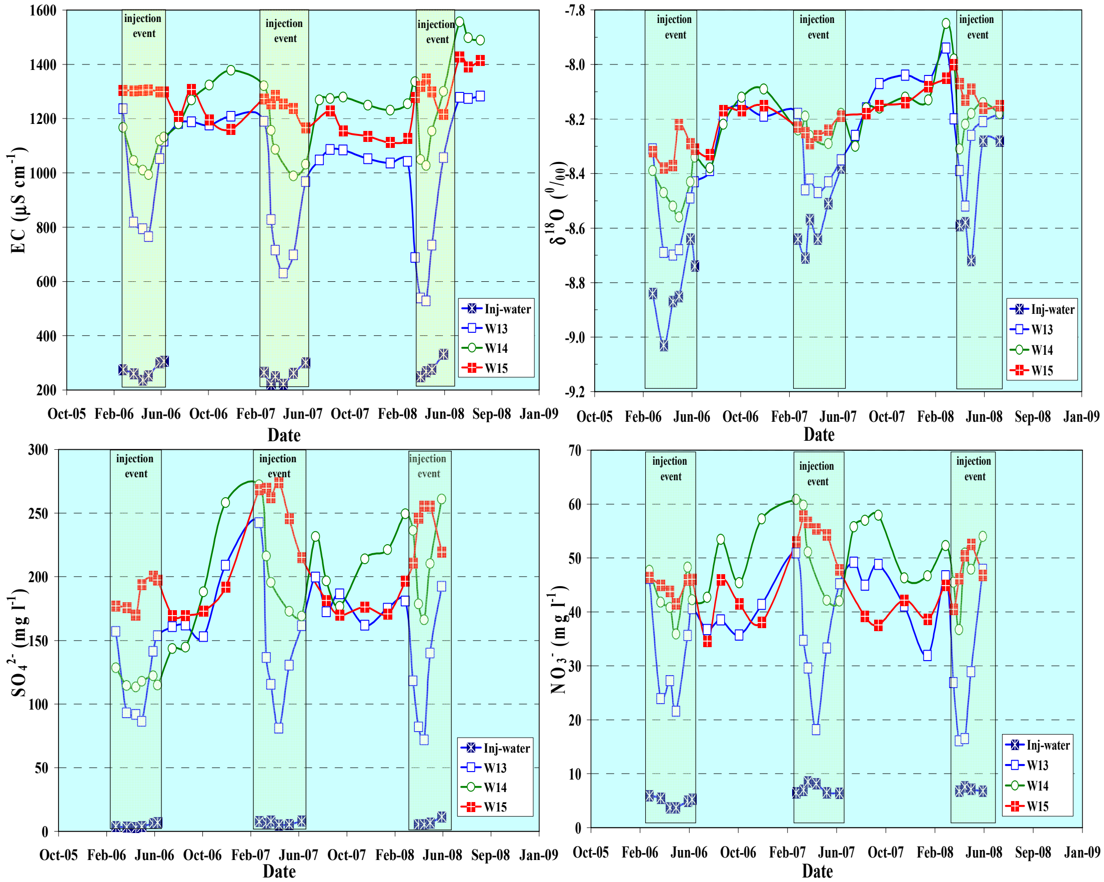

Figure 3. Temporal variations in EC values, $\delta^{18}O$, SO_4^{2-} and NO_3^- concentrations of the injectant water (Inj-water) and ambient groundwaters of the different observation wells (W13, W14 and W15) during the period (March 2006-June 2008).

This result agrees well with the geographic positions of wells W13 and W14, located at distances of ≈30 m and ≈80 m, respectively, from the injection point. Although the well W15 is located very close to wells W13 and W14, its temporal evolutions in all parameters, were evidently without sensitive changes, suggesting hence ineffectiveness of this well by injections. This is because of the longer distance that separates this well from the injection point (≈145 m), and also because of the systematic groundwater flow, usually from west to east. The important point that needs to be stressed here is the remarkable decrease of nitrate and sulphate concentrations in the case of well W13, where the concentration of both parameters becomes half the initial value registered before the injection events.

EFFECTS OF ARTIFICIAL RECHARGE ON GROUNDWATER QUALITY

Figure 4 illustrates variations of the average and ± standard deviation values of TDS, pH, $\delta^{18}O$, DO_2, Na^+, Cl^-, SO_4^{2-} and NO_3^- concentrations in the injectant water and ambient groundwaters of the different observation wells, calculated for water samples collected during and after the three injection events of 2006, 2007 and 2008.

With the exception of pH and DO_2 parameters, which increase as a result of artificial recharge activities, the concentrations of remaining chemical and isotopic parameters decreased as a consequence of such injections. However, the concentrations of monitored chemicals in the ambient groundwater in well W13 were the lowest compared with those of other observation wells (W14 and W15), and stay even lower than those calculated for the periods outside the injection events.

Monitoring of the groundwater quality at the DUC site prior to recharge events indicated that nitrate, sulphate and TDS were the chemicals of concern relating to artificial recharge activities (Ziegler et al., 1999).

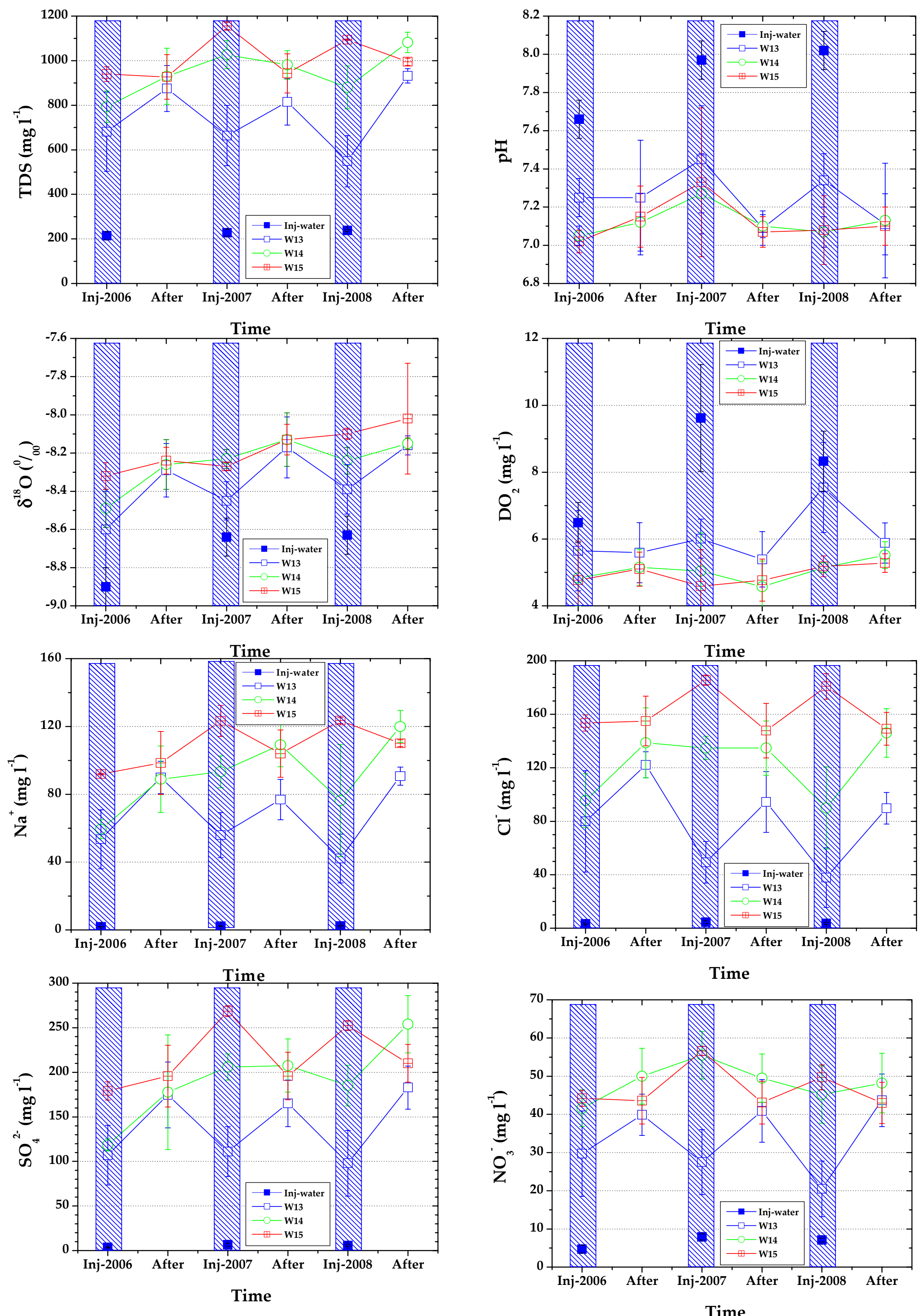

Figure 4. Variations of the average and ± standard deviation values of TDS, pH, $\delta^{18}O$, DO_2, Na^+, Cl^-, SO_4^{2-} and NO_3^- values in the injectant water (Inj-water) and ambient groundwaters of the observation wells (W13, W14 and W15), calculated for the injection and after injection *events during 2006-2008.*

The concentrations of these chemicals exceed, in a large number of groundwater samples, the established standards for drinking water (USEPA, 2001). Nitrate concentrations, as an anthropogenic chemical species, were rather high ($40<NO_3^-<57$ mg/L) at the DUC site prior to artificial recharge began. After artificial recharge began nitrate concentrations in the ambient groundwater of well W13 substantially decreased to values in the range of 18-30 mg/L, and remained higher than 40 mg/L in the unaffected ambient groundwater (i.e. well W15). Noting that nitrate concentrations in the injectant water were below 7 mg/L during all artificial recharge events.
Sulphate concentrations, as a naturally occurring substance in drinking water, were also high in the DUC ambient groundwater ($180<SO_4^{2-}<270$ mg/L) in many samples exceeding the maximum contaminant level (250 mg/L), established for standard drinking water (USEPA, 2001). After artificial recharge began sulphate concentrations decreased in the ambient groundwater of well W13 to a level of ≈100 mg/L. Similarly, the relatively high TDS concentrations (940<TDS<1155 mg/L) in the ambient groundwater of well W13 dropped down to values slightly higher than the maximum permitted level (500 mg/L), given for drinking water (USEPA, 2001).
The overall effects of artificial recharge on groundwater quality in the Quaternary alluvial aquifer at the DUC site can substantially established by studying the relationships that exist between the average values of the different parameters and the total injected water volume at each artificial recharge event. Such correlations suggest that geochemical reactions and dilution process occurred in the ambient groundwater at this recharge site. Accordingly, slight increases were observed for pH and DO_2 values in the ambient groundwater with the increasing volume of injectant water. However, with the exception of bicarbonate, sulphate and sodium, which show the worse correlation coefficients ($R^2<0.3$), when correlating their average concentrations with the amounts of injected water volume (Table 3), the remaining major chemicals remarkably display good negative correlations ($R^2>0.5$) as a consequence of artificial recharge.
The best correlation coefficient was found for the chloride ion ($R^2=0.995$), reflecting its chemical behaviour as a conservative mobile ion (Hem, 1992). A similar good correlation was also found for $\delta^{18}O$ ($R^2=0.992$), proving also the usefulness of such stable isotopes in calculating the proportions of injectant water reaching affected wells (Kattan, 2010). The values of correlation coefficients, calculated for the remaining hydrochemical parameters, decrease in the following sequence (Table 3): $EC>Mg>Ca>^2H>K>NO_3>DO_2>pH>TDS>Na>SO_4>HCO_3$

Table 3. Values of the linear equation constants, together with the correlation coefficient (R^2), calculated for the different relationships between the different parameters and the injectant water volumes.

Relationships	**a[1]**	**b[2]**	**R^2**
Cl-injectant water volume	-1.2762	158.43	0.995
$\delta^{18}O$- injectant water volume	0.0063	-8.9902	0.992
EC- injectant water volume	-8.7307	1445.3	0.950
Mg-injectant water volume	-0.218	38.753	0.944
Ca-injectant water volume	-0.900	163.7	0.909
K-injectant water volume	-0.0167	4.8437	0.895
δ^2H- injectant water volume	0.0406	-51.552	0.878
NO_3-injectant water volume	-0.2294	44.374	0.659
DO_2-injectant water volume	0.0461	2.6864	0.611
TDS- injectant water volume	-3.1514	883.47	0.550
pH-injectant water volume	0.0042	7.0075	0.511
Na-injectant water volume	-0.2278	68.842	0.276
SO_4-injectant water volume	-0.1566	117.95	0.160
HCO_3-injectant water volume	-0.102	286.25	0.005

a[1] and b[2] values mean the calculated slope (a) and intercept (b) values of the linear equation (Y=a.X+b) constants, respectively. X and Y denote the injectant water volume in m^3 and average concentration of considered parameter, respectively.

The low correlation coefficient observed for sodium ($R^2=0.276$) reflects most probably the effect of ionic exchange with bivalent ions, within the aquifer matrix (Hem, 1992). Whereas, the worse correlations found for bicarbonate and sulphate are most likely because of the rather complex biogeochemical reactions that could occur in the aquifer matrix. The small changes in major ion concentrations in ambient groundwater at the DUC site, that were observed during the monitoring period, can be explained by the huge amount of groundwater extraction from this aquifer for water supply purposes. Nevertheless, the results were promising and help to conclude that continued injection of fresh water in this site will lead without any doubt to reduced concentrations of chemicals of concern to values closer to drinking water standards.

CONCLUSION

Although, there have been little changes in major ion concentrations in the groundwater sampled from the DUC recharge site during the period 2006-2008, the overall effects of artificial recharge scheme on the groundwater quality in the alluvial Quaternary aquifer were substantial, especially when comparing the average concentrations prior to recharge began to those calculated during the periods of recharge events. The positive implication related to artificial aquifer recharge by using water of low concentrations such as that of the Figeh Springs is represented by diluting the high concentrations of chemicals of concern (nitrate, sulphate and TDS). However, slight increases in pH and DO_2 values, together with remarkable decreases in the concentrations of most major ions in the ambient groundwater at the DUC recharge site, mainly the well W13, can also be observed.

The average concentrations of chemicals of concern, calculated for the periods after recharge events, were all substantially higher than their respective drinking water standards. The concentrations of these chemicals decreased in the case of ambient groundwater of well W13 during recharge events to concentrations closest to those measured in the injectant water. Because of extraction of the stored water in this aquifer, the concentrations in most ions tend to increase again. In all the cases, the concentrations decrease in the affected ambient groundwater in different rates, and become, if not similar, slightly lower than those found before recharge began.

Because of the significant difference in stable isotope compositions, mostly in ^{18}O concentration, between the injectant water ($-8.9<\delta^{18}O<-8.6$ ‰) and ambient groundwater ($-8.3<\delta^{18}O<-7.9$ ‰) at the DUC site, application of such an isotope revealed that it is a sensitive marker that responds quickly on any hydraulic connection between the injectant water and ambient groundwater. Chloride as a mobile conservative ion has also shows similar behavior to that of ^{18}O. Thus, both natural tracers can successfully be used for accurate estimation of the proportions of injectant water reaching the affected observation wells.

Although the dry climatic conditions prevailing in the area during the study period, the results were encouraging for continuing this scheme of artificial recharge as an alternative water management method. The potential benefit of such an application option is not only due to raising up the potential groundwater reserve, but also for preventing degradation of groundwater quality in this shallow alluvial aquifer.

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Stormwater artificial recharge method through vadose injection into volcanic aquifer in Jeju, South Korea

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Artificial groundwater recharge is a promising technology to address the threat to water supplies posed by climate change. An artificial groundwater recharge project, which directs water from seasonal flash flooding into a volcanic aquifer, is being conducted in a high-elevation area, at a site adjacent to Han Stream in the northern part of Jeju Island, South Korea. Jeju is a volcanic island composed of overlapping permeable structures, such as clinker and scoria, and less permeable structures, such as tuff, massive volcanic rock body and fine-grained volcanic material. There is Mt. Halla at the center of the island and the area above approximately 600m contour is protected as a national park that is good for source water quality which is stream discharge occurring in case of precipitation over about 80 mm/day. Jeju island has a very thick vadose zone more than 100 meters at high-elevation area which is good condition to inject source water through vadose zone and then to naturally filter the injected water during long-term transport. A Jeju-friendly aquifer recharge technology (J-ART) is conceptually designed based on these hydrological, meteorological, and geological properties. This artificial recharge system using reservoir adjacent to the stream could reduce threat of flood and secure groundwater resources to Jeju Island.

Keywords
Artificial recharge, vadose injection, Jeju Island, clinker zone, volcanic aquifer

Introduction

Jeju Island is a young volcanic island and the biggest one, about 1800 square kilometers, in Korea. Groundwater is a very important resource for the island because up to 99.5% of total supply for all of the domestic, agricultural, and industrial use is supplied by groundwater including spring water which comes from perched groundwater. Natural recharge of rainfall is in the trend of decreasing because of a recent climate change of less annual rainy days and heavier rain under similar amount of annual precipitation and an expansion of urbanized area. On the other hands, water demand is increasing due to the development of tourism industries such as golf club, resort, hotel, etc.

The Jeju Special Autonomous Province government enacted a groundwater law and ordinance to promote artificial recharge of rainwater collected from roof of green house or building and to support partially construction budget for the rainwater catchment or recharging systems. And the local government feels strongly the necessity of artificial recharge using other resources like stream water to secure groundwater resources.

The objects of this study are to develop groundwater artificial recharge method using stream water which is favorable to the geological, hydrogeological characteristics of Jeju volcanic island and to secure groundwater resources for the future increased demands.

Hydrogeological characteristics of Jeju

Jeju is a volcanic island of 1.7 M years old and mostly consists of volcanic rocks such as basalt, trachyte, trachy-basalt, tuff, and etc and some sedimentary rocks such as seogwipo-formation which is known to be a reworked sediment of old tuff cones (Park, et al., 2006). The permeable structures in the volcanic rocks such as clinker, lava tube, and scoria cone in Jeju are favorable for the infiltration of rain water and transport, underground storage, and development of groundwater. The impermeable structures like fine sediment and massive rock body are imbedded repeatedly between the permeable structures. Because the lava sequence made of overlaying each other permeable and impermeable structures plays an important role by acting as a natural filtration system, the infiltrated water or artificially injected water can be filtered to be pure and clean during the movement through thick unsaturated zone into deep groundwater body. The tuff and seogwipo-formation sited deep above the basement rock and u-formation (unconsolidated formation) acts as lower impermeable boundary keeping the groundwater.

Groundwater occurs at shallow depth in coastal area except for southern area where there exists the low permeable Seogwipo formation. However, owing to high infiltration rate of surface and subsurface materials and high lateral transmissivity of permeable structures, the unsaturated zone is getting much thicker toward the central mountain, ‘Halla-san’, resulting in deep groundwater table at higher elevation area (Figure 1). For example, the water table of the well located at the elevation of 300 m is generally about 100 m – 150 m above sea level. This feature makes a perched aquifer at high elevation area appearing as a spring or as falling water within the wellbore after drilling. The main aquifer in Jeju Island consists of multiple variably-confined small aquifers, each of which is isolated by low permeable layers, resulting in strong vertical movement within the well, static water level change during drilling, discontinuity of groundwater-saline water interface at some coastal area.

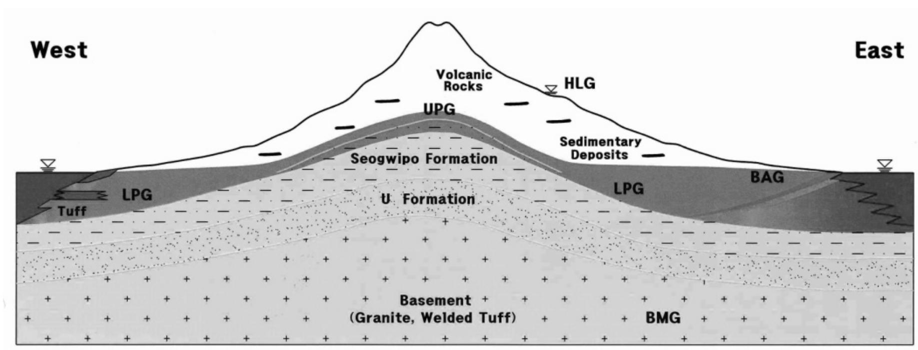

Figure 1. Geological cross section and groundwater occurrence in Jeju Island (Koh, 2008)

Hydrological characteristics of Jeju Island

Jeju Island has one of the highest levels of rainfall in Korea. The annual mean rainfall for 30 years as measured by the meteorological observatories in Jeju City, Seogwipo, Gosan, and Seongsanpo is 1,597mm. Annual mean rainfall for the whole island from 1993 through 2002 is 1,975mm, which is higher the national average of 1,283mm (Korea Water Resources Corporation, 2003). Rainfall varies greatly by regions. The annual rainfall is 2027 - 2339 mm in the south, east, and north area while being just 1299 mm in western area because of monsoon and orographical effect. The annual precipitation depends also elevation and the increase rate of the annual precipitation along the elevation is about 273 mm for the increment of the elevation of 100 m resulting in precipitation of more than 3000 mm at the center of the Island where there is the Halla-san (mountain).

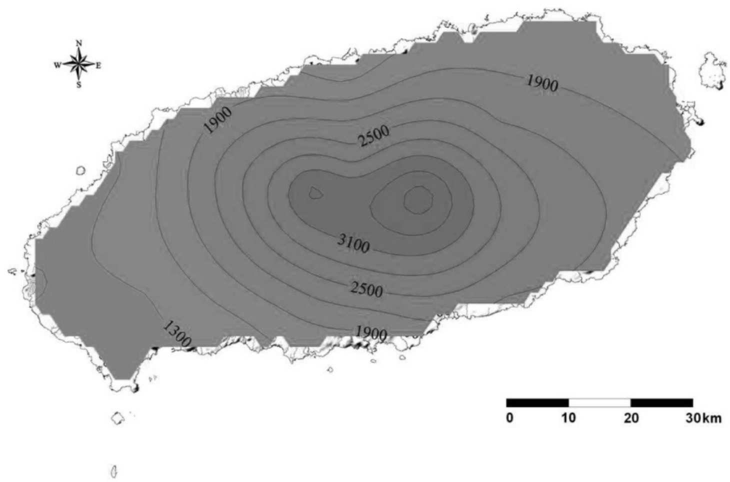

Figure 2. Distribution map of precipitation in 2006 (Korea Institute of Geoscience and Mineral Resources, 2007)

According to water budget analysis of 2003, total amount of precipitation is 3.39 billion cubic meters and about 35% of it is evapotranspirated, 46% of it is recharged into aquifer, and about 19% of it is discharged through stream to the sea (Korea Water Resources Corporation, 2003). The stream is normally dry except during heavy rain of more than 50 mm a day although the precipitation criteria depend on previous surface condition before precipitation. However, the total annual discharging amount to the sea is 640 million cubic meters which can be a good source for artificial recharge with aids of diversion facility. Based on the data collected from the stream discharging monitoring station at Hancheon which is located at northern Jeju, the frequency of stream discharging days is in average 13.2 days a year, stream discharging water level of 0.2 m – 4.03 m , and the average discharge duration is 23 hours (Korea Institute of Geoscience and Mineral Resources, 2008). The amount and the rate of stream discharge at Hancheon for last 5 years variedfrom 0.62 million m^3 to 12.3 million m^3 and from 2.6 % to 33.7%, respectively.

The stream water at Hancheon has a good quality except for turbidity which might attribute soil particles eroded by running surface water and reworked from construction action (Table 1). Because the central area of the Island above the elevation of about 600 m is protected from contamination by Halla-san National Park, the stream water with good quality could be a good source for artificial recharge if the turbidity is properly treated. Fortunately, the turbidity of the stream water was decreasing with time as the discharge rate went down and the peak values of the turbidity decreased since September, 2007when new recordable precipitation and storm water with highest water level followed by Typhoon NARI (PAGASA name Falcon, international designation: 0711, JTWC designation: 12W) that resulted flood of City Jeju located at down-gradient. (Figure 3).

Table 1. Stream water quality of Hancheon

Parameter (mg/L)	Values	Parameter (mg/L)	Values	Parameter (mg/L)	Values
EC (μs/cm)	14.0~38.0	Cl^-	0.86~4.39	T-N	0.12~1.52
DO	6.08~14.37	SO_4^{2-}	0.79~3.08	T-P	0.01~1.32
Ca^{2+}	0.66~6.06	NO_3^-	0.01~0.81	Cu	0.00
Mg^{2+}	0.23~0.76	TDS	7.80~18.23	Cd	0.00
Na^+	0.20~4.10	SS	0.30~23.20	Pb	0.00
K^+	0.10~1.84	COB	1.60~8.80	Turbidity(NTU)	1.57~2,000
HCO_3^-	0.14~10.98	BOD	0.10~3.71		

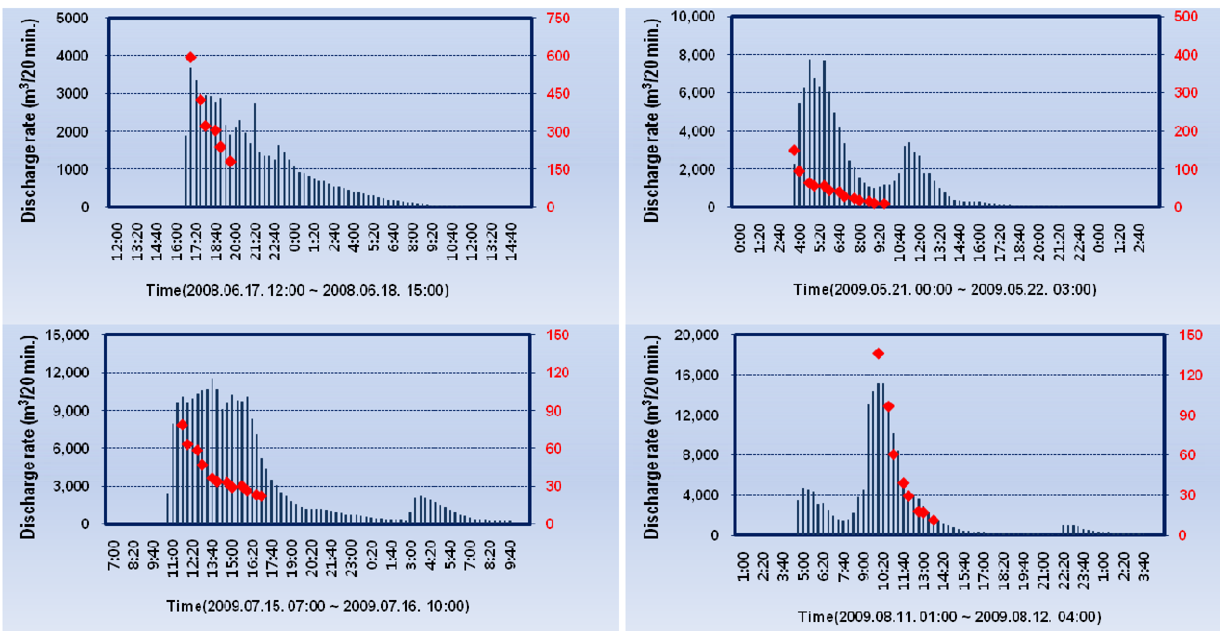

Figure 3. Change of the discharge rate (bar graph) and turbidity in NTU (solid diamond) with time at Hancheon.

Flood mitigation reservoir as stream water diversion facility

After the flood in Jeju at September, 2007, the local government established a comprehensive master plan for flood control and flood mitigation reservoirs near the previously flooded streams were planned to be built at the upper gradient area from the city including Hancheon. Hancheon reservoir-I and reservoir-II with capacity of 459000 m^3 and 463000 m^3, respectively. They are located at the elevations ranging from 280 m to 370 m, and were constructed in 2009 and 2010, respectively (Figure 4). These reservoirs can be used as storm water diversion facility for artificial recharge facilities.

Multi-depth Injection tests

The groundwater level is about 150 m to 200 m deep from the surface depending on season which means thick unsaturated zone exists there. Multi-depth injection tests were performed during drilling of an injection well at the bottom of the each reservoir to evaluate the unsaturated zone permeability above the regional aquifer in the vicinity of the proposed artificial groundwater recharge site. Infiltration characteristics indicate that the clinker zones around 40 meters from the land surface has injection capacity of about 15000 m^3/day and would be a suitable for artificial groundwater recharge within the vadose zone with high utilization of natural geological filtration system (Figure 5). Once drilling depth for vadose zone injection was decided, total 10 injection wells are drilled at each reservoir.

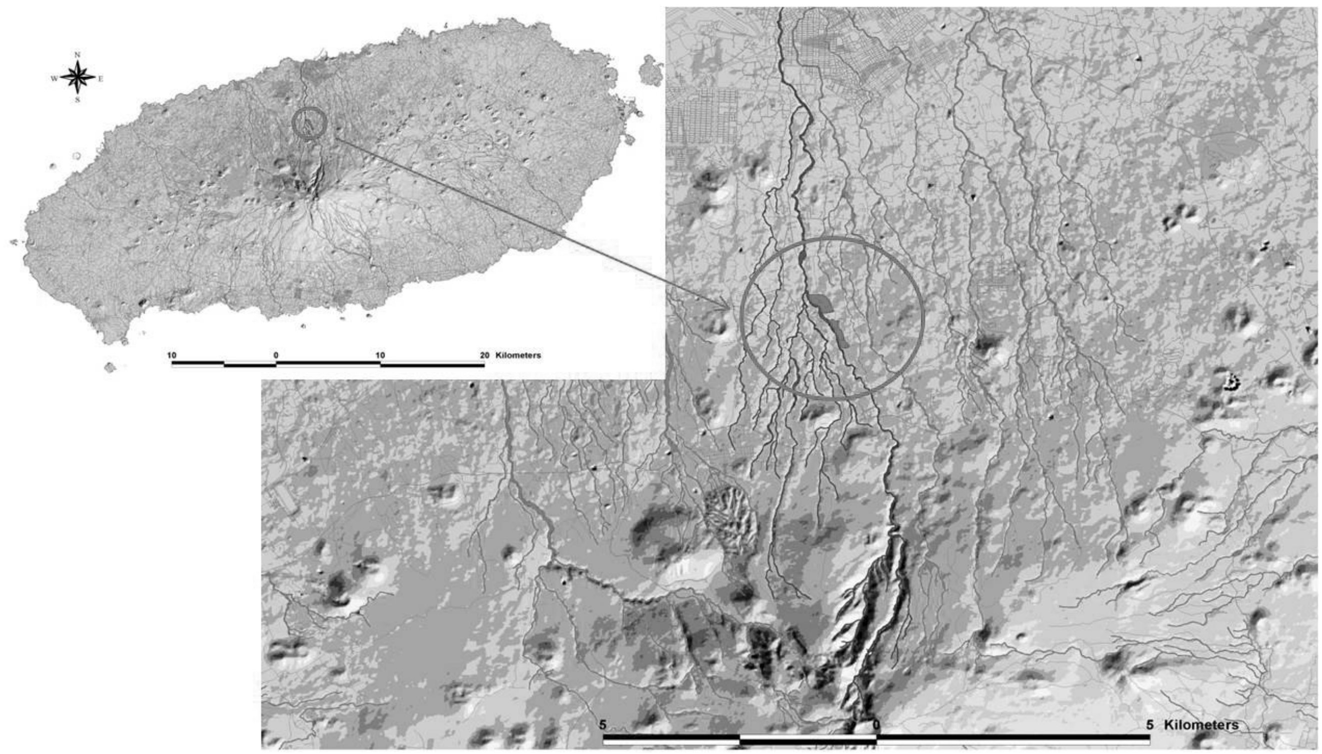

Figure 4. Artificial recharge site coupled with flood mitigation reservoir at Hancheon

Jeju artificial recharge technology (J-ART)

A Jeju-friendly aquifer recharge technology (J-ART) is conceptually designed based on the hydrogeological, hydrological, and meteorological chracteristics. Stormwater discharge monitoring station and flow control valve in the injection well inlet, which is linked with turbidity sensor via control panel, are equipped to control the whole system. J-ART is a technology for securing sustainable water resources by capturing ephemeral stream water, which is normally stormwater, with no negative interference in environment such as natural recharge or eco-system, recharging it through vadose zone injection wells, and making it to be used at down-gradient production wells (Figure 6).

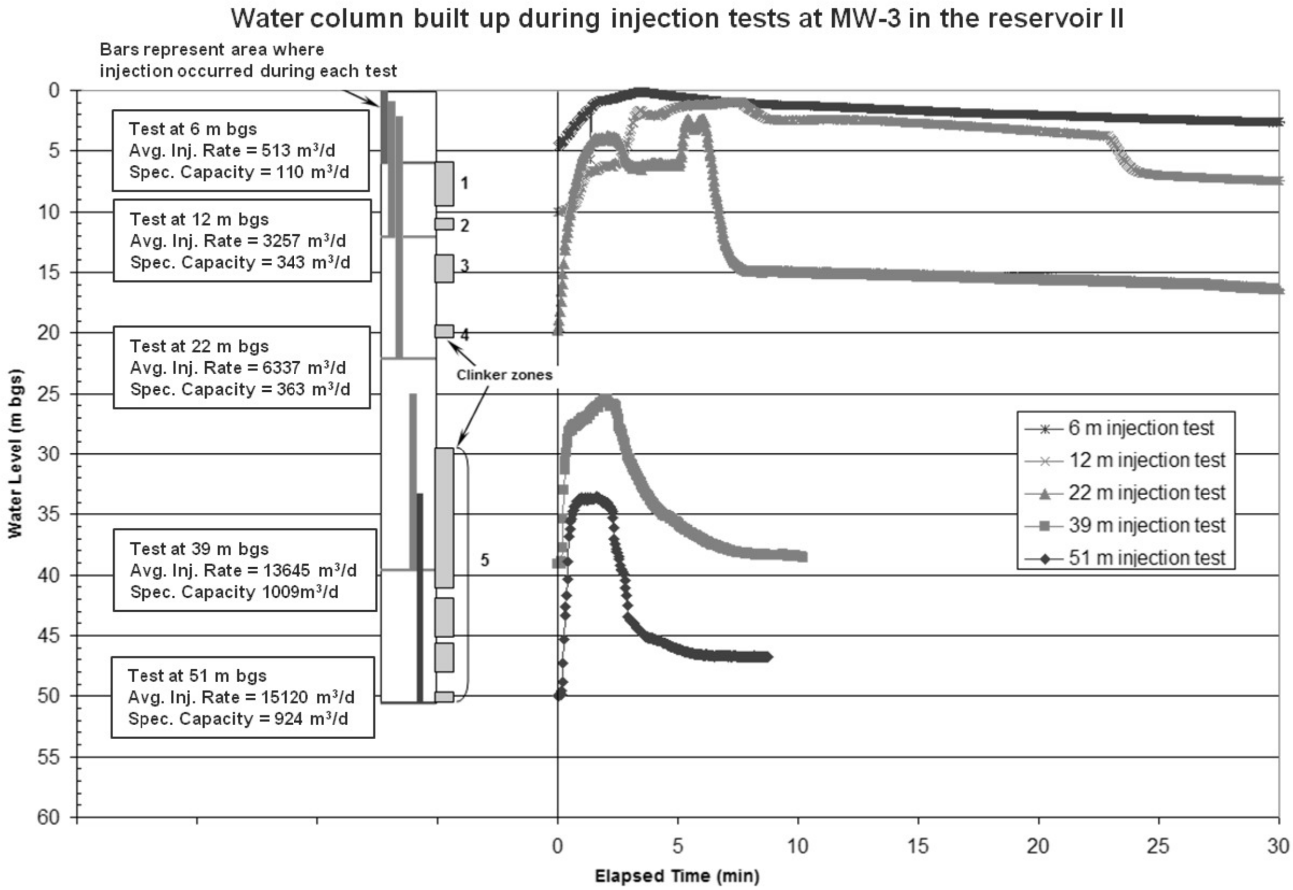

Figure 5. Location of clinker zones (left) and the change of water column built up during injection tests. (right).

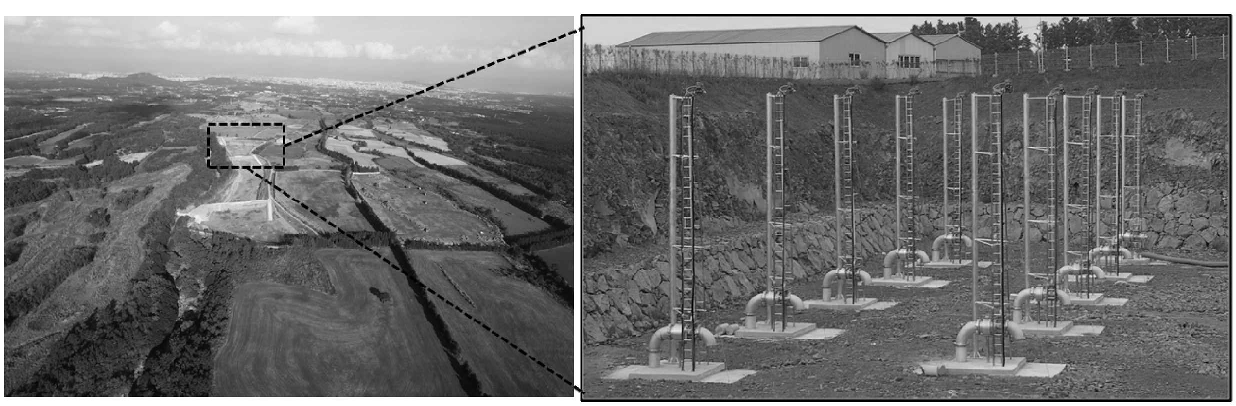

Figure 6. Aerial view of Hancheon flood mitigation reservoir-II (left) and recharge wells drilled at the bottom of reservoir being equipped with injection control valves (right).

Conclusions and Discussions

An artificial recharge method which is an aquifer storage, transfer, and recovery type (Rinck-Pfeiffer, et al., 2005), suitable for Jeju volcanic Island based on the hydrogeological an hydrological properties is designed and full scale test sites are developed to secure additional water resources. J-ART has advantages as follows:

- It uses stormwater discharging quickly to the ocean and stores it at subsurface through vadose zone injection;
- Due to vadose zone injection through the thick unsaturated zone it has large storage capacity, long transport time, and good natural filtration system;
- The vadose zone injection wells have the capacity of about 15000 m^3/day for a well of 40 m depth;
- It adopts an injection control system linked with turbidity sensor via control panel;
- It can secure additional water resources and can promote the flood mitigation capacity of the reservoirs by recharging the water in the reservoirs to the subsurface reservoir and reduce the threat of flood;
- It could be an adapted method to the climate change using subsurface buffer effect compared to surface water resources.

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Fresh Storage Salines Extraction (FSSE) wells, feasibility of freshwater storage in saline aquifer with a focus on the Red Sea coast, Egypt

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Introduction

Storage of fresh water is necessary strategically as well as to reduce cost of desalination for potable water production, by bridging periods of high demand with water stored in the subsurface during low demands. Especially in dry areas like the Red Sea Coast tourist resorts are booming which need fresh water; because no fresh water is available, it is produced by desalination of local groundwater, which is generally about as saline as sea water. Because of strong demand fluctuations with the tourist seasons combined and the high cost of these plants, water storage can save money. Subsurface storage has the benefit of the large space available and the absence of evaporation and sunlight heating. Subsurface water storage may also be substantially cheaper than water storage above ground using expensive man-made tanks of steel or concrete. However, subsurface storage of the fresh desalinated water is a challenge due to the lower density of this water compared to the ambient saline groundwater. This density difference causes the stored lighter fresh water to float up to the ceiling of the aquifer and spread out, and so, to be lost within days to weeks if just left to its own. This study shows how the fresh water can be kept in place around the well by continuous pumping of saline water at a limited rate from below the stored cone-shaped freshwater bubble. This continuous low-rate pumping presents no loss for these systems as a rate fresh water production is required even in low-demand times and is sold to the consumer. Therefore, this way of storing fresh water is an innovative way of making use of the economic settings and the groundwater physics that go with the density differences between fresh and salt water.

Aquifer Storage Recovery

In situations with seasonal fluctuations in water availability and water demand, storage could be used to bridge peak demands. Adequate storage may be the key to sustainable water management, by overcoming water shortages and drought spells. There may be sufficient overall amounts of water available in many cases; however, storage is necessary to be able to use it [Pyne, 1995]. When water is stored in an aquifer with the purpose to extract it later, the storage principle is called ASR, or Aquifer Storage Recovery, which may be defined as the storage of water in a suitable aquifer by a well during times when water is available, and subsequent recovery of water by the same well during times when it is needed [Pyne, 1995].

ASR refers to a time-limited storage of water in aquifers. The water is injected by wells and usually extracted again at a later time by the same wells. In Florida, USA, ASR is used for seasonal storage of river water and rainwater [Buros and Pyne, 1994]. In the Netherlands, province of South Holland, a hundred or so horticulturists inject rain water from the roofs of their green houses in a shallow semi-confined brackish aquifer for use during the summer growing season.

The Red Sea coastal area

Since the Red Sea region was first targeted for massive tourism development around 1990, the number of rooms has grown spectacularly, reaching 10.500 in the year 2000 (22% of the country) with 140.000 as the target for 2012. The majority of the resorts are in a 300km long coastal stretch with about 50-300m coastal setback depending on the shoreline conditions [Shalaan, 2005.]. Fresh groundwater is absent or insufficient to supply the tourist areas with potable water making desalination of seawater and groundwater necessary to satisfy freshwater needs [El-Sadek and Mabrouk, 1992].

The desalination plants along the Red Sea coast typically produce water for one resort or a cluster of hotels, which have a freshwater demand of less than 1500m3/d, i.e. Nefertary (500m3/d), Equinox (300m3/d) and Cataract (1400m3/d). This situation favors the small-size Reverse Osmosis (RO) plants, with production capacities ranging between 200 and 3000m3/d. RO-plant capacities beyond 3000m3/d are limited to the main towns, i.e. El Gouna (6000m3/d), Hurghada, Safaga, Port Ghalib (3000m3/d) and Mersa Alam. The RO-desalination capacity in the Red Sea area increased from less than 20.000m3/d in 1980 to about 140.000m3/d in 2001 [Hafez and El-Manharawy, 2002].

Reverse Osmosis is used to remove virtually all dissolved salts and organic micro-pollutants from water. It is always preceded by a pre-treatment step to remove particulate matter. Lack of suspended solids is one factor making groundwater an attractive resource compared to direct use of seawater.

According to Dabbagh (2001) the efficient management of desalinated water supply is becoming more and more important. While further improvements in desalination processes are certainly desirable and need to be researched and developed, it is becoming even more important that desalinated water be managed efficiently given the already large number of existing desalination plants in Egypt.

The seasonal fluctuation of freshwater demand of the hotels, villages and resorts along the Red Sea coast is large and coincides with the seasonal character of tourism. The data acquired during the field trips in April and May 2007 to the drinking water plants of El Gouna, Nefertary, Port Ghalib, Equinox and Cataract is given in Van Ginkel, (2007).

Storage could improve the efficiency of the desalination plants and help bridging production interruptions. Instead of increasing the production capacity every year to match peak demand, the plants may run on a more even production if enough water can be stored during low demand periods for use in times of high demand.

Problem

Subsurface storage of desalinated water using ASR seems to be the perfect solution to improve the efficiency of the desalination plants along the Red Sea coast of Egypt. Injected freshwater displaces the native saltwater in the aquifer during injection. On the fringes mixing will take place, due to advection and dispersion, and a mixing zone between the two water types is present, separating the injected freshwater in the bubble from the saltwater in the surrounding aquifer.

However, the density difference between fresh and saltwater poses a challenge as it tends to float the stored freshwater upward to the top of the aquifer where it may be hard or impossible to recover at a later stage. This problem may be less evident in well-known systems with continuous recharge of fresh rainwater, causing a permanent freshwater lens to float on saline deeper water in a permanent dynamic fashion. In Egypt, given the size of the production sites, we would only store small amounts of freshwater and further, we do not inject continuously during the storage phase. Moreover, there is no precipitation to maintain a permanent freshwater lens. Hence, in the Egyptian situation the freshwater bubble will naturally float up due to density difference between fresh and saltwater, so that the freshwater bubble would be lost as a thin layer on top of the saline aquifer in a matter of days to weeks.

Analysis of a FSSE well

According to Bear (1978) the storage of a bubble of freshwater in a saline aquifer is possible if the interface is maintained dynamically. The essence of the matter is that a groundwater velocity difference across the interface between the stored freshwater and the present saltwater must be maintained. This is expressed by

(1.1) $$\sin\alpha = \frac{\rho_f}{\rho_s - \rho_f}\frac{q_f - q_s}{K}$$

Where α is the inclination angle of the interface ρ_s and ρ_s are the densities of the fresh and saltwater respectively, K is hydraulic conductivity of the aquifer, q_f and q_s are the groundwater discharges on either side of the interface and directed long it. This formula is the heart of the analysis and assumes a steady-state, hence equilibrium interface position..

This implies that the inclination of the interface is maintained as long as there is a groundwater velocity difference between the stored fresh and the present saltwater. The interface will be horizontal when the velocity difference equals zero, i.e. when no injection of freshwater or flow of saltwater occurs. We may trap the stored water around the well as long as we inject freshwater or extract saltwater or do both simultaneously to maintain this velocity difference.

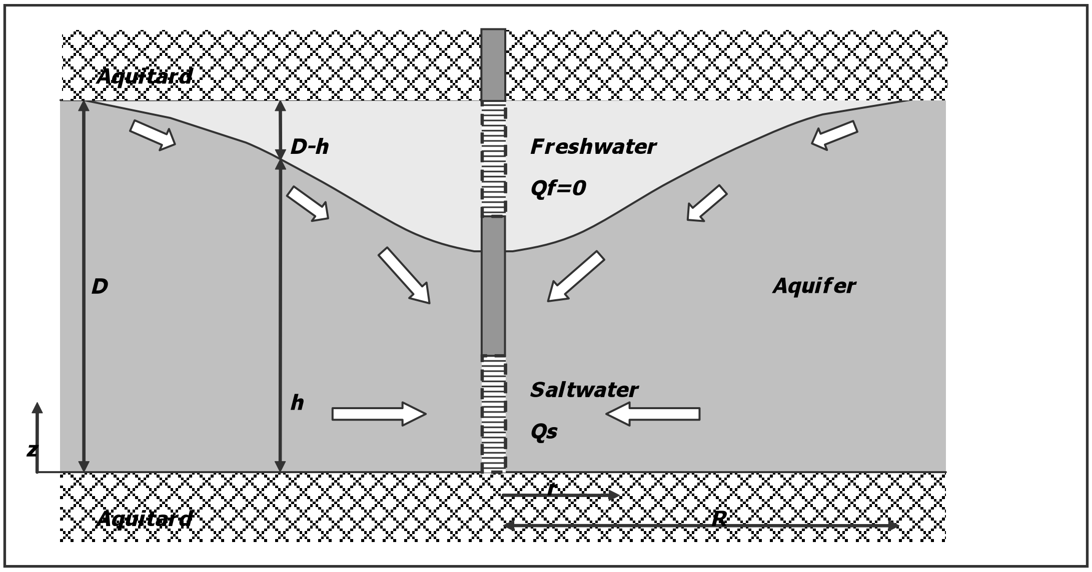

Figure 1: Fresh Storage Saline Extraction (FSSE) well in a confined aquifer during the storage phase.

The principle of saltwater extraction is visualized in Figure 1, showing an aquifer with a Fresh Storage Saline Extraction (FSSE) well with two screens. The upper screen injects freshwater, but in this figure, the freshwater is in its stagnant storage phase, while saltwater is being extracted with the lower screen. A velocity difference between stored fresh and present saltwater is thus maintained keeping the interface in place. This is similar in technology to the technique of scavenger wells in Pakistan but now with the objective to keep a fresh water volume in place instead of maximizing continuous extraction of fresh water.

The dynamic storage around a FSSE well may be analyzed analytically for the confined and unconfined situation (Figure 2 and Figure 3) in axial symmetric or flat cross section conditions, while a computer code like SEAWAT (ref 2008) is required to study more complex, multi-well cases which includes effects of diffusion and dispersion, ambient groundwater-flow and operation dynamics.

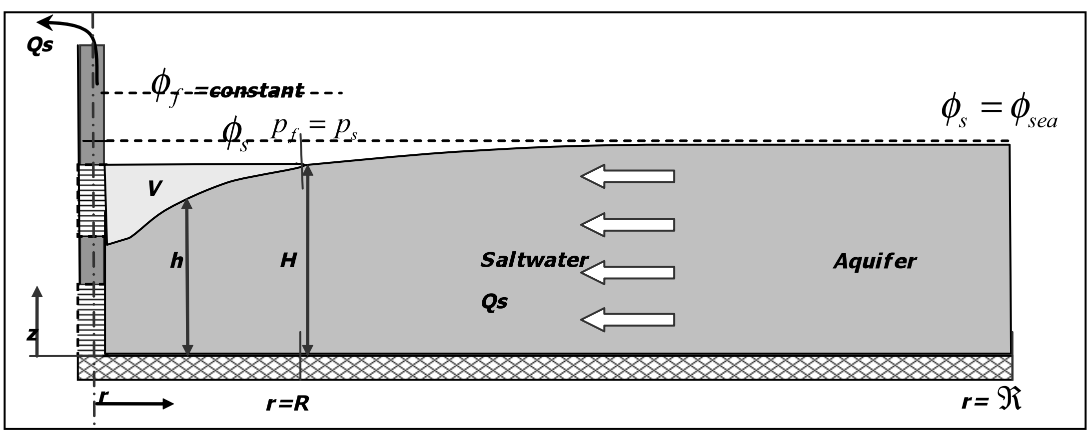

Figure 2: Radial symmetric cross-section of a FSSE-well in an unconfined aquifer.

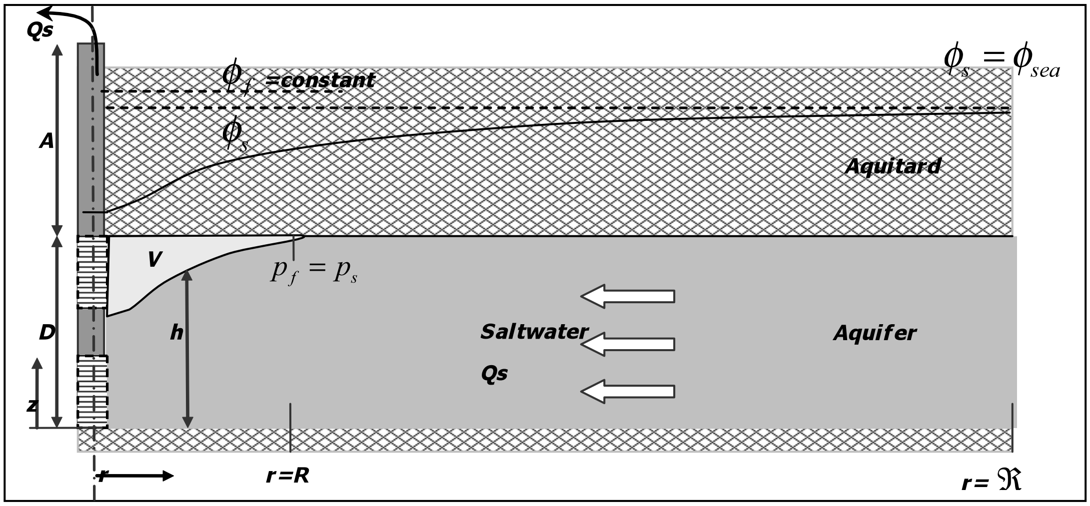

Figure 3 Radial symmetric cross-section of a FSSE-well in confined aquifer.

Analytical solution

The analytical solution of the steady flow of salt water towards a well keeping in place the stored volume of fresh water on top of it, is fairly straightforward, yet, to our knowledge, not found in the literature, where many papers study and discuss the important subject of storing fresh water in a saline or brackish environment, but not in a way to actually capture and fix the stored fresh water in the vicinity of the well, despite or, with the help of the gravity difference that normally renders this storage impossible for extended periods. The analysis presented, shows that the bubble of fresh water stays fixed around the well and will be little if at all affected by ambient flow other than a change of its shape.

This situation is assumed steady state; the salt groundwater flow Q_s [L^3T^{-1}] towards the well is constant at any distance $r<R$ [L] and further given by Darcy's Law:

(1.2) $$Q_s = 2\pi rkh\frac{d\phi_s}{dr}$$

Where K [LT^{-1}] is the horizontal conductivity , h [L] the distance of the saline water table above the base of the aquifer and ϕ_s [L] the saltwater head .

The freshwater pressure head is equal to the saltwater pressure head at $r=R$

(1.3) $$\left(\phi_f - H\right)\rho_f = \left(\phi_s - H\right)\rho_s$$

Where H [L] is the static freshwater head with respect to the base of the aquifer.

Where $r<R$ the freshwater pressure at the interface equals the saltwater pressure, $\left(\phi_f - h\right)\rho_f = \left(\phi_s - h\right)\rho_s$, so that the saltwater head can be written as

(1.4) $$\phi_s = \frac{\rho_f}{\rho_s}\phi_f + \frac{\rho_s - \rho_f}{\rho_f} h$$

Combining yields

(1.5) $$hdh = \frac{Q}{2\pi k v_s}\frac{1}{r}dr$$

Solving with boundary condition that *h=H* at *r=R* yields

(1.6) $$h = \sqrt{H^2 + \frac{Q_s}{\pi k v_s}\ln\left(\frac{r}{R}\right)}$$

Or, for the confined case with as boundary conditions *h=D* at *r=R*, we get

(1.7) $$h = \sqrt{D^2 + \frac{Q_s}{\pi k v_s}\ln\left(\frac{r}{R}\right)}$$

The volume of the freshwater bubble, *V*, can be calculated by integration of its thickness between the well radius r_w and *R*

(1.8) $$V = 2\pi r\varepsilon \int_{r_w}^{R} (D-h) r dr$$

to yield

(1.9) $$V = 2\pi n \int_{r_w}^{R} \left(D - \sqrt{D^2 + \frac{Q_s}{\pi k v_s}\ln\left(\frac{r}{R}\right)} \right) r dr = 2\pi\varepsilon \int \left(a - \sqrt{b \ln cr}\right) r dr$$

Where for the unconfined case $a = H$, $b = Q_s / (\pi k v_s)$ and $c = \exp\left(H^2 \pi k v_s / Q_s R\right)$ and for the confined case $a = D$ and $c = \exp\left(D^2 \pi k v_s / Q_s R\right)$

The result of the integral is

(1.10) $$V = \frac{1}{4}\pi\varepsilon\left[\left(a - \sqrt{b\ln(cr)}\right)4r^2 + \left(2\sqrt{2}\sqrt{b}r^2 F(x)\right)\right]_{r_w}^{R}$$

With $F(x)$ the Dawson function, which is defined as

(1.11) $$F(x) = e^{-x^2}\int_0^x e^{-t^2} dt$$

With $x = \sqrt{2}\sqrt{\ln cr}$

The freshwater volume may thus be calculated for different saltwater extraction Q_s and radius of the bubble, R.

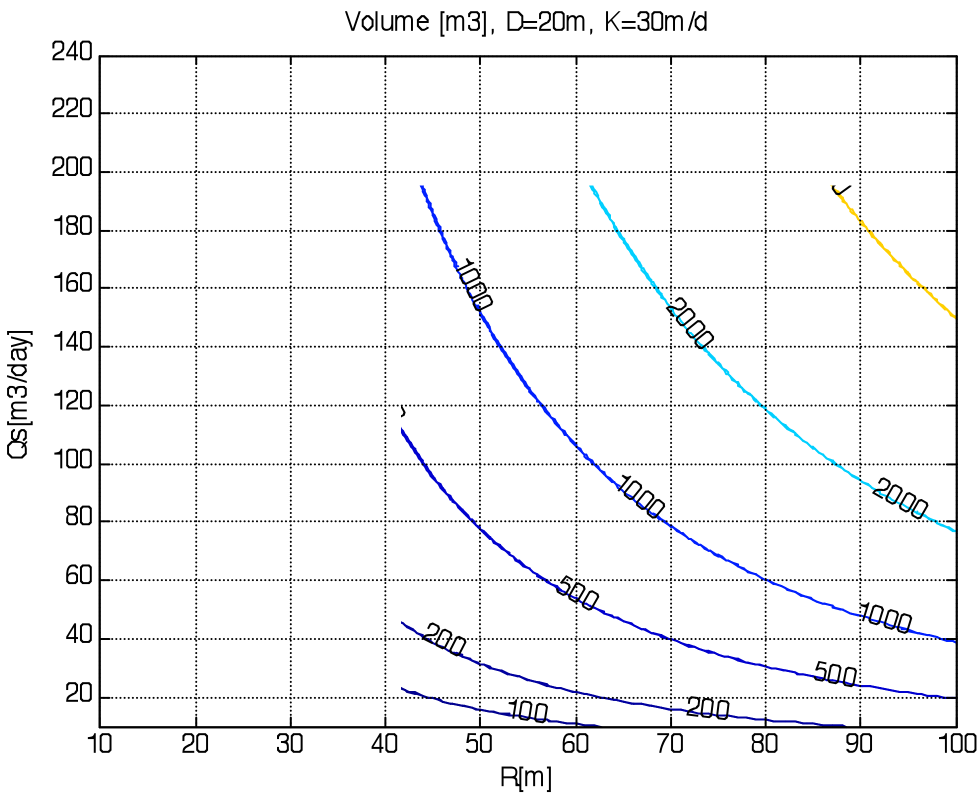

Figure 4: Freshwater storage volume as a function its radius R and the saltwater extraction Q_s for D=20 m, and K=30 m/d, porosity 35% (the numerical and analytical results overlap almost perfectly.

This analytical solution and its graphical representation in Figure 4 may be used as guidelines. It determines the maximum freshwater volume that can be stored under the given steady state circumstances ignoring any losses due to dispersion and diffusion and assuming a homogeneous subsurface.

Numerical analysis

Clearly the analytical approach is limited to a steady state situation with a stagnant floating freshwater body, without the influences that occur under real circumstances. A numerical approach is necessary to better quantify such effects. For this to work the numerical code must allow for transport of water with varying quality and density, and allow the quantification of dispersion, diffusion and dynamics as well as the ubiquitous heterogeneity of the subsurface. Even though the setup and concepts are simple and the previous analytical solution is straight-forward, the combination of the just mentioned processes makes successful operation of FSSE wells a challenge, until enough experience has been acquired in practice. The complexity however, makes it impossible to design and operate such wells on intuition; numerical simulation is an essential tool for the understanding, design and operation of such systems.

In this study we simulated the functioning of the FSSE well by means of the SEAWAT computer code [Guo and Langevin, 2002], which allows the mentioned processes to be included. We did so using axial symmetric and cross section models. Future work will include full 3D modeling, which requires a lot more computing power.

The goal of the modeling effort is to find out how a freshwater bubble in a saline aquifer, stored by means of a FSSE-well, can be recovered; how it behaves under different circumstances (conductivity, depth, heterogeneity, ambient groundwater flow and calamity); and what the recovery rate is.

This research focuses on the situation pertaining to the desalination plants in tourist resorts along the Egyptian Red Sea coast. The model parameters are based on the data collected during the field trips in April and May 2007 to the drinking water plants of El Gouna, Nefertary, Port Ghalib, Equinox and Cataract in Egypt.

This area is virtually without natural recharge. The groundwater has salinity equal to seawater quality (42000ppm) and the aquifers are connected to sea generally with depths thickness less than 30m, consiting of gravelly sand, rocks, and limestone with sand, with K-values ranging from 10 to 50m/d [Shata, 2007].

The model is meant to be representative of the situation in the five focus locations: The distance to sea is 1000m, the salinity of the aquifer is 42000ppm, the aquifer confined thickness D=20m with average hydraulic conductivity K=30m/d, 35% porosity and an initial head of =35m above the base of the aquifer, which is virtually equal to sea level.

SEAWAT makes it possible to simulate 3D variable density transient groundwater flow. SEAWAT couples the flow and transport equations of two widely used codes (MODFLOW [Mc Donald and Harbaugh, 1996] and MT3D [Zheng and Wang, 1999]) with some modifications to include density effects.

Rapid buoyancy of fresh water infiltrated to float on top of saline groundwater was illustrated using a partially penetrating well of 5m length, injecting 100 m3/d for 60 days after which the bubble was left to its own. The obtained bubble radius measures 105m its maximum thickness is reduced from 12 m at the moment of stopping the infiltration to 4 m after 140 days later. This demonstrates that buoyancy is an important process on the time and spatial scale relevant to the storage; without a mechanism to preserve the shape of the bubble no long-term storage will be feasible.

Instead of just leaving the bubble to float, we can extract some fresh water with a deeper screen in hte same borehole as a method to keep the stored freshwater in place.

Hydrodynamic dispersion is important process as it causes mixing when water flows. Its effect can be summarized by the standard deviation of the mixing front, which, in this simulation reaches 3.5m. This would imply a loss of 1000m3 water, 16% in our example. However, the mixing zone can may also be considered a pre-investment equivalent to the ‘walls’ of an alternative ground-surface storage basin [Pyne, 1995], as it kind of prevents losses in future cycles due to reduced salinity gradients.

Recovery of the freshwater

During the recovery of the freshwater bubble, a velocity difference across the interface between fresh and saltwater must be maintained to counteract buoyancy. While injection of freshwater may be done without simultaneous extraction of saltwater, extraction of the freshwater requires a way to maintain the required velocity difference across the interface.. Therefore, a first design proposal is to increase the saltwater extraction when extracting freshwater, which may be done without problems during peak demand. In case of emergencies, saltwater pumping of the extraction station may have to be focused on the storage well, to ensure the necessary saltwater pumping during freshwater recovery.

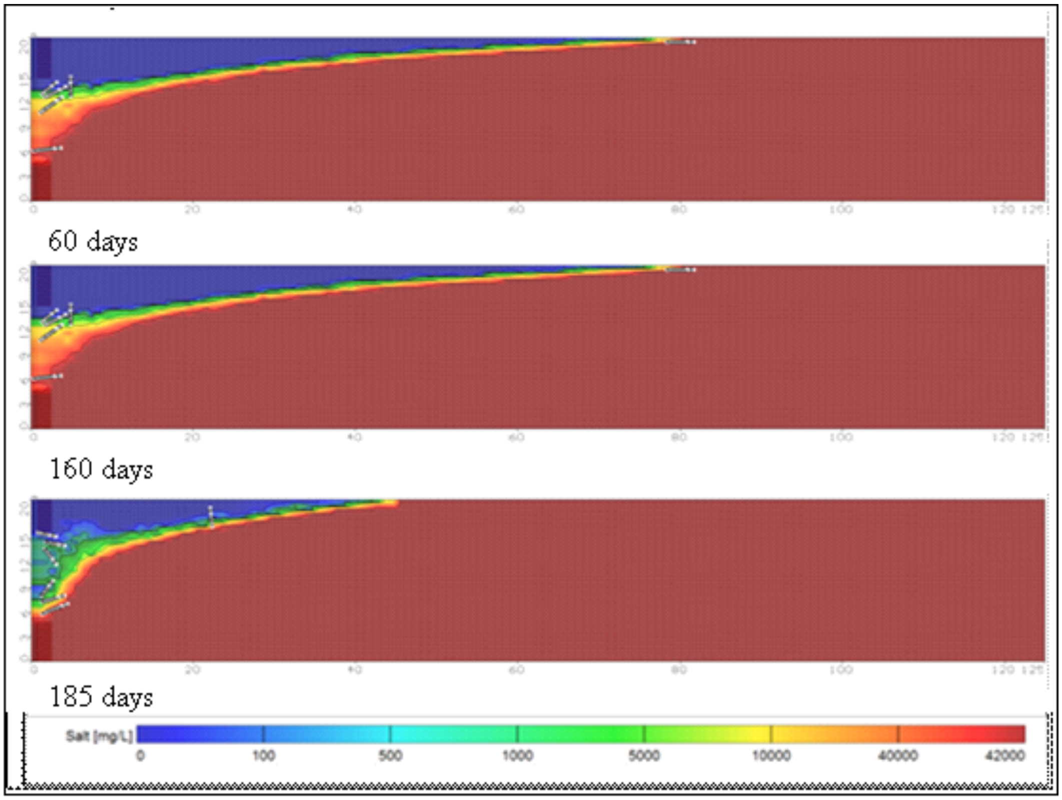

Figure 5: SEAWAT simulation result where 60 days marks the end of the period during which freshwater was injected, 160 days marks the end of the period of steady-state storage of freshwater with continuous extraction of saltwater at Q=240 m3/d. Finally 185 days shows the situation after extraction of freshwater between since day 160 of simultaneous extraction of fresh and saline water at the moment when the interface intersects the lower screen and reduction of the saltwater extraction rate is necessary The colorbar indicates salinity. The horizontal scale is 0-120 m, vertical scale 0-20 m.

During extraction the interface has to be kept between the upper freshwater screen and the lower saltwater screen. Tie interface elevation near the well turns out to be quite sensitive to the the ration of the extraction of fresh and saltwater, but in practice this may be overcome by automation based on an EC sensor in both screens. Including dispersion, it thus seems that 50% recovery is feasible in the first storage cycle with 6000 m3 stored around a single wells and 3000 m3 recovered before salinity increases above standard.

In cycled operation where, after an initial storage of 6000 m3 of fresh water recovery takes place until salinity reaches a standard and then infiltration-recovery cycles are repeated

The simulation of the behavior of the bubble for different cycles of one month duration showed that, although recovery efficiency of the first cycle is only 50%, that of subsequent cycles increase to 100%, mainly due to the initial “loss” which, should rather be regarded as an investment in a “salinity wall”. Which makes further development of this method of freshwater storage in saline aquifers viable. Clearly, the circumstances are special, in the case of the Red Sea resorts, the saltwater extraction to keep the stored freshwater in place is never lost.

Seepage face

Seepage faces are well known from inflow of groundwater into an open well in an unconfined aquifer. However, seepage faces always occur when two fluids meet at a fixed pressure boundary no matter whether it concerns water and air or freshwater and saltwater. In the case of freshwater floating on saltwater a seepage face develops naturally at the well bore [Olsthoorn, 2007]. In the Figure 6 the reason for this seepage face is visualized.

The groundwater velocity component into the well caries along the seepage face and reaches a maximum at the point of the interface inside the well, at the bottom of the seepage face. This implies that saltwater will enter the freshwater column in the well, as part of it enters above the elevation of the interface in in the well. The length of the seepage face is dependent on the pumping rate, the well radius, the radius of influence and the hydraulic conductivity [Chenaf and Chapuis, 2007].

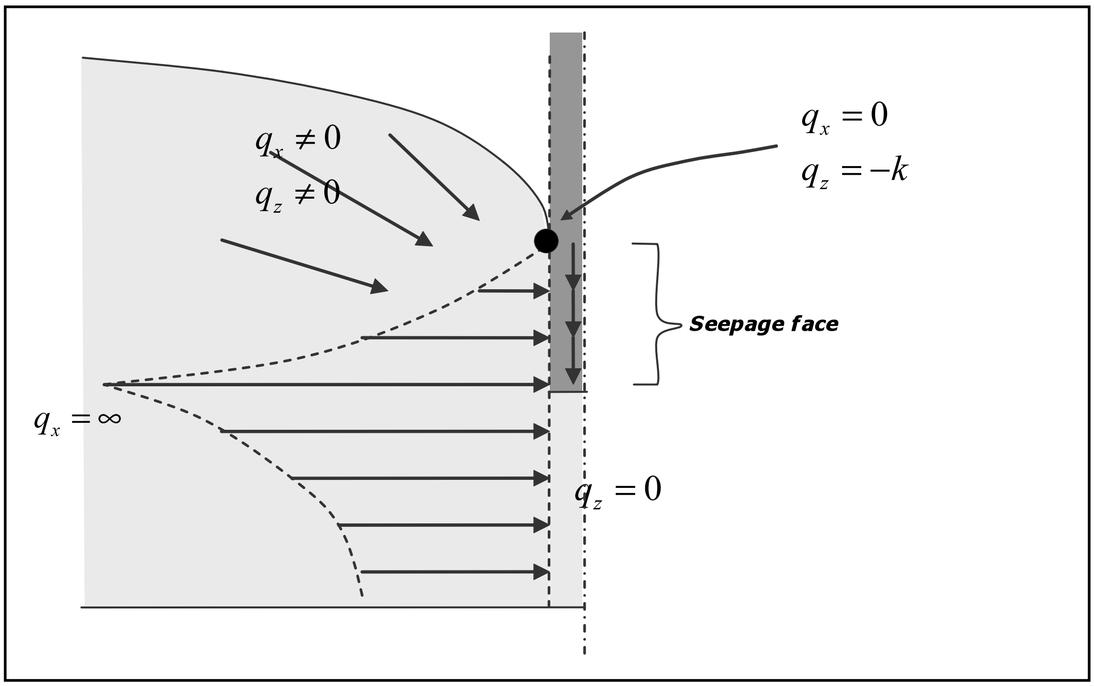

Figure 6: Seepage face showing the grey saltwater, the curved interface above which the stagnant freshwater resides. The dark grey is freshwater inside the well resting on light-grey saltwater. Part of the saltwater directly enters the freshwater inside the well at the seepage face. The arrows indicate the horizontal component of the saltwater inflow, which reaches as maximum at the point the interface of fresh and saltwater inside the well. Therefore, the seepage face has to be blinded off.

The seepage flow is an important factor to avoid or else it would lead to a continuous mixing of incoming saltwater and resident freshwater. Therefore a blind casing must be used over the height of the fluctuation of the interface to prevent the inflow of saltwater directly into the freshwater. Because, even then there is still the risk that the interface inside the well descends into the lower screen, a packer inside the casing to completely separate the fresh and the saltwater is essential.

Sensitivities

Some sensitivities are worthwhile and important in considering this type of storage. The most important ones are discussed below.

Hydraulic conductivity

The steepness of the interface is directly proportional to the inverse of the conductivity of the aquifer. Hence the importance of this parameter from the perspective of FSSE-well storage. If the aquifer is too conductive, the bubble of freshwater will be flat and extended and, therefore, hard to recover. On the other hand, if it is low, it not only limits the capacity of the wells, but it my make the interface too steep to store large volumes, while on the other hand, lower conductivities reduce buoyancy and make it easier to exploit the full thickness of the aquifer with lower overall losses.

Aquifer thickness

The aquifer thickness is important in combination with its conductivity, as, salt water pumping may not be feasible; the interface will hit the bottom of the aquifer. It may not be too big a problem, to cope with; saltwater pumping comes in action when the interface hits the well near the bottom of the aquifer, which may happen soon after starting the extraction, because the interface tends to rapidly turn backward when extracting freshwater alone..

Aquifer heterogeneity

Heterogeneity influences dispersion and, therefore, affects recovery efficiency negatively. On the other hand, it facilitates building a diffuse buffer zone which raises efficiency in later cycles. See the discussion on regarding first-cycle losses as an investment to raise efficiency in future cycles due to the forming of a diffuse buffer zone. Heterogeneity may vary with depth, as is the case with conductivity.

Layers of different conductivity

Aquifers generally possess a small-scale layered structure, which reduces vertical over horizontal conductivity. This is generally an advantage for freshwater storage in saline aquifers as it reduces the buoyancy.

On the other hand, aquifers also tend to possess a larger scale layered structure with distinct layers of higher and of lower conductivity on different elevations and of thicknesses in the order over decimeters to tens of meters .This layering will cause the injected water to spread out at different rates creating a ragged and fingered outer limit. This character is likely to disturb the interface, which may even locally become reversed, and, therefore will cause more transverse dispersion, also compared with single-density fluid storage and a much more complicated pattern of density flows. Such effects are likely complex and need not only to be investigated in models, but definitely also in practice, but only after good local data have been obtained and the model was used to support the design thus honoring the encountered complexity of the subsurface.

Operations

Operations largely influence the position of the interface and, therefore, are essential to manage in accordance with the time of storage of the freshwater body and the characteristics of the aquifer. An unused stored fresh water volume floating on saline groundwater will degrade in quality due to ongoing dispersion and diffusion. Hence, storage duration must be in relation to storage volume and the mixing processes that occur, including operations.

Calamities

What happens when saltwater pumping is interrupted due to emergencies, power breaks or even maintenance? The freshwater bubble will float up; so the question is then to what extent can it be recovered later, or, to what extent can the original shape of the stored volume be reestablished afterwards by accurate pumping? The modeling shows that it is indeed possible to restore the bubble after an interruption of short duration, but after a month without saltwater pumping, the buoyancy has reduced the thickness of the stored volume to such an extent that recovery is no longer feasible, according to the model.

Ambient flow

An important feature of this method is, that the stored volume is relatively unaffected by ambient groundwater flow. Normally this background flow is a major disadvantage for subsurface storage systems as it causes losses, which either have to be accepted or must be counterbalanced by proper layout of the wells and adequate operations to counterbalance the gradual movement of the stored volume of water. Ambient flow, which includes the floe caused by neighboring wells is always one of the most important factors in the design of subsurface storage systems. However, the modeling shows that with FSSE wells, ambient flow only distorts the shape of the volume, while the stored volume is effectively fixed around the well by saltwater pumping, thus preventing ti to drift along with the surrounding groundwater.

Conclusions

Freshwater storage in saline aquifers seems possible by means of Fresh Storage Saline Extraction (FSSE) wells. Such wells can be beneficial in situations prevailing along the Red Sea coast of Egypt, where resorts generally fully depend on desalinated groundwater and no freshwater sources are available.

The main characteristic of an FSSE well is continuous pumping of salt water at a small rate effectively glues the stored freshwater to the well , keeps it in place, maintains the interface inclination and prevents it from drifting away the the background groundwater flow.

The derivation of the analytical solution is presented and numerical modeling was carried out with SEAWAT to study dynamic behavior of the stored freshwater including effects of operation dynamics, dispersion, diffusion and, especially density differences.

The study shows that it is feasible to store freshwater in and recover freshwater from a saline aquifer by means of Fresh Storage Saline Extraction wells under favorable conditions and proper design and operations.

The seepage face analysis shows that this type of storage cannot be successful without a separate screen for the fresh and one for the saltwater extraction. It can therefore not done with existing wells.

The stored and recoverable volume of freshwater depends on many factors whose influence on the storage capacity could be investigated using the numerical model. These factors are porosity, depth, hydraulic conductivity, heterogeneity, buoyancy and ambient groundwater flow and operations. The sensitivities are shortly discussed in the text.

Mixing causes losses during the first storage-recovery cycle, but they increase efficiency during subsequent ones. Thus, these losses should be regarded an investment to be included in the a priori cost estimation and not at losses at all (Pyne, 2005)..

It is possible to recover the freshwater after an interruption of the saltwater pumping for about a week during which the freshwater bubble floats upward due to its buoyancy, but it seems not recoverable when the interruption lasts for around a month because it has become to thin a layer in the top of the aquifer.

The study was carried out by two field visits to 5 sites along the Red Sea Coast, where measurements were carried out and data collected for this study.

Acknowledgement

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Significance of Natural and Induced Vertical Leakage for an MAR Scheme: Geelong, Australia

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Abstract
This paper investigates the potential impact of an MAR scheme located in an upper aquifer on a water supply borefield screened in an underlying aquifer. This is achieved by assessing vertical fluxes and drawdown in the upper aquifer using a two layer dimensional analytical model. While over time, the modelling suggests that most of the water to the borefield is sourced from the upper aquifer, the maximum potential contribution from the MAR scheme to the borefield is less than 2%, due to the distance of the borefield from the MAR scheme. It is further shown than minimum travel times from the MAR site to the borefield were 40+ years, and therefore the risk of any substances or species reliant on biodegradation or die off (e.g. pathogens) surviving to the borefield is practically nil. Using dimensionless numbers developed by Ward et al. (2009) for assessing recovery efficiency, in combination with the analytical modelling, it was shown that the risk of significant recovery efficiency issues was relatively low, and indeed site conditions appeared reasonable favourable for high recovery rates of the injected water. Gradients induced by the borefield in the upper aquifer were generally less important than the storage time of the injected water. Storage times of more than half a year may result in reduced recovery efficiencies. The approach developed in this paper is a useful first pass tool for indicating that the likelihood of impacts on the borefield are very low and that that there is a reasonable likelihood of favourable conditions for recovery of the injected water.

Keywords
Analytical models, Australia, Geelong, Managed aquifer recharge, recovery efficiency, vertical leakage

INTRODUCTION

Barwon Water and MAR

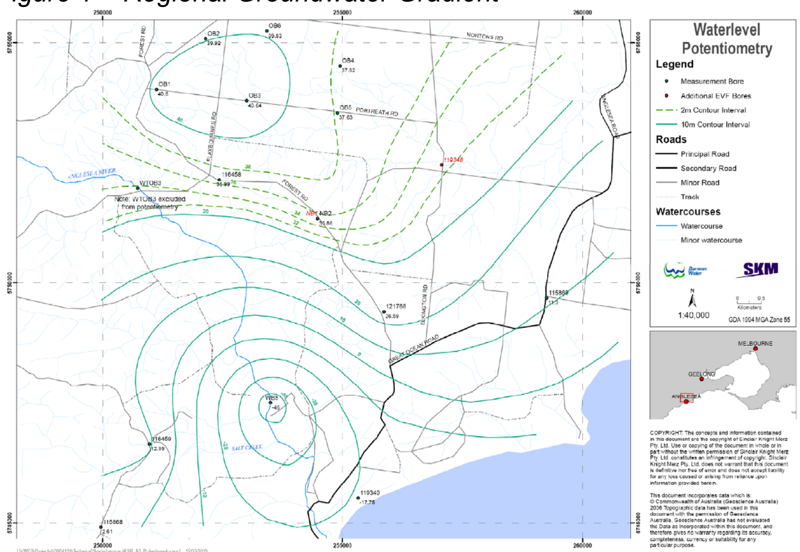
Barwon Water Authority supplies around 270,000 people in the Greater Geelong area (60km south west of Melbourne, Australia) with water and sewerage services. During the current prolonged dry period, groundwater comprises up to 50% of the Greater Geelong water supply. Managed aquifer recharge is therefore a logical non-conventional technology for investigation. Barwon Water is currently pursuing three investigation areas in the region. This paper uses early data from a two year investigation program in one of these areas (the Upper Eastern View Formation (UEVF) in the Portreath Rd area), to assess the significance of vertical leakage on a potential Managed Aquifer Recharge scheme.

The exact source water for the MAR scheme has not yet been decided; the two main options include high quality recycled water or stormwater from a nearby town. Options for end use of the water are also still open and range from supplying local irrigation demand, third pipe water for garden water/toilet flushing in new housing developments, or potentially adding the water back into Geelong's water supply system. However this last option is currently not possible, as it is against existing Victorian policy. The approximate minimum size of the MAR scheme that would be considered viable is 3 GL/yr, although a scheme of up to 10 GL/yr would be preferred.

Hydrogeological setting based on results to date

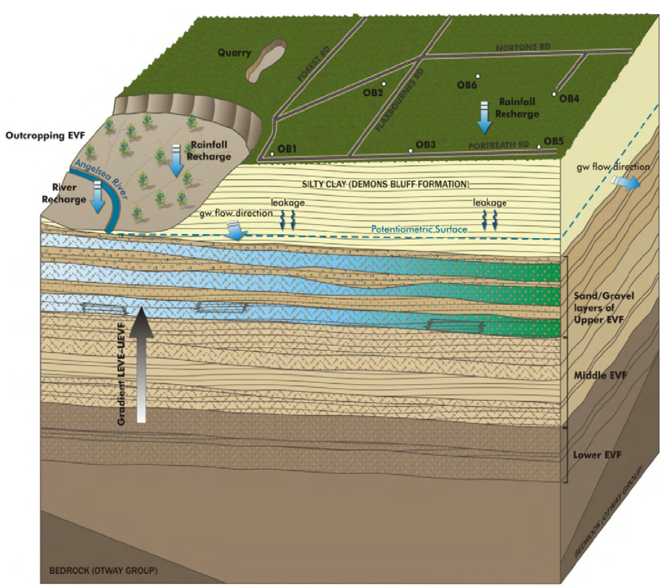
The field investigation program to date has involved drilling and hydraulic testing of six observation bores and water quality sampling. The program confirmed the presence of the UEVF Aquifer in the area and the fact that, hydraulically, it is suitable for MAR. The aquifer consists of interbedded sands/gravels, clay and coal seams. Three sand/gravel layers were intersected at depths between 80 and 140m below ground level. These layers dip to the east (towards the coast) and to the south (towards the Jan Juc syncline). The thickness of these layers range between approximately 5 and 20m. Preliminary hydraulic testing indicated the sand/gravel layers have a (combined) transmissivity in the range of 200 – 400 m^2/day and hydraulic conductivities in the range of 10 – 20 m/d. (An average rate from of 11 m/day is used in this analysis).

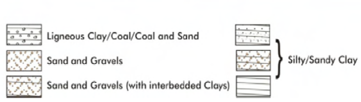
Figure 1 – Regional Groundwater Gradient



Within the investigation area the aquifer is confined by a 60-100m aquitard (Demons Bluff Formation), with a potentiometric level 40-70m below ground level. However, the two eastern bores (OB1 and OB2) are close to the confined – unconfined boundary of the aquifer system. The investigation area is also located immediately east of outcropping EVF. The ambient groundwater gradient within the study area (i.e. around OB2) is to the south and east, as shown in Figure 1. Groundwater salinity ranges from less than 1,000 mg/L TDS to 4,000 mg/L TDS, with a salinity gradient from east to west across the investigation area, probably reflecting increasing distance from the recharge area. Within the study area, groundwater levels are below the Anglesea River (currently approximately 7m below river level). The preliminary modeling undertaken suggests that the likelihood of groundwater levels being raised to the river bed level during the injection cycle are very low. Hence any significant groundwater-surface water interaction occurring is considered unlikely. The Lower EVF is separated from the Upper EVF by the Middle EVF aquitard. The Middle EVF in the study area is comprised of clay layers separated by sandy layers, rather than a continuous low permeability layer. Nested hydrographs indicate an upwards gradient from the Lower EVF to the Upper EVF of around 11 – 12 metres (over a vertical screen interval difference of around 100m). A conceptual model of the study area is presented in Figure 2.

Figure 2 – Conceptual Model of Study Area





The area around OB2 was selected as the preferred site for the focus of further field investigations. The two primary advantages of this site are the high transmissivity of the aquifer and the moderate groundwater salinity. Preliminary analytical modeling indicates an ASR scheme in the vicinity of OB2 is expected to be able to store between 4 and 7 GL/yr (11 to 19 ML/d). Up to twice as many recovery bores compared to injection bores may be required due to drawdown constraints during recovery. An ASTR scheme is anticipated to be able to transfer around 10 GL/yr (27 ML/d), with less than 10 bores.

Aim of this paper

The MAR investigation at the site will necessarily involve assessment of a wide range of technical issues to determine the viability of the scheme. The aim of this paper is to assess one aspect of the MAR scheme feasibility - the significance of the natural and induced vertical gradient at the site. There is a natural upward vertical gradient from the upper (UEVF) to the lower (LEVF) aquifer. There is however significant groundwater extraction approximately four kilometres south of the site in the LEVF which has recently commenced as part of Geelong's water supply, and which will reverse this gradient locally. The vertical gradient is potentially significant in that:

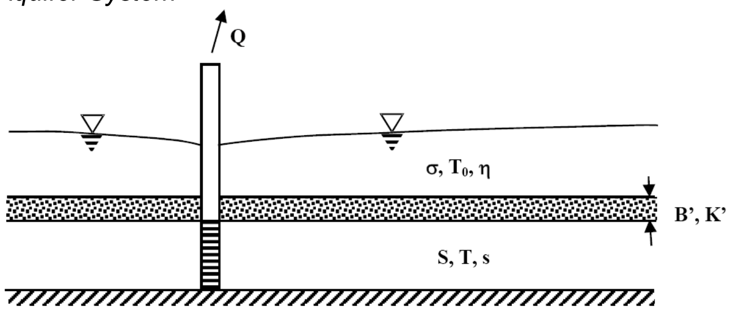
1. The MAR scheme could impact on the bore field to the south. The source water for the MAR scheme has not yet been decided. If a lower quality water (fit for a lower quality end use than potable, for example, irrigation) was used in the MAR scheme, then the question of possible impact on the bore field is a potential issue. This is comprised of two sub-issues:
    a. First is the question of the straight volumetric impact on the bore field. This in turn relates to the impact of conservative species in the injected water (e.g. salinity, nitrate etc). For example, if the injected water contained an average salinity of 1,000 mg/L TDS, and the bore field to the south is producing water at approximately 500 mg/L TDS, then there is the potential for the salinity of the bore field to increase as the borefield captures some of the MAR injected water. This paper examines the significance of this effect.
    b. Second is related to the travel time between the MAR scheme and water supply borefield. This is of relevance in terms of potential water quality impacts from species such as pathogens in the injected water, which are subject to die-off within the aquifer system over time. This paper examines whether the vertical gradient between the two aquifers could shorten travel times such that species subject to biodegradation and decay within the aquifer could be problematic for the bore field.
2. The bore field to the south could impact on the MAR scheme – the vertical gradients induced by the extraction in the lower aquifer could impact adversely on the recovery efficiency of the MAR scheme, i.e. the pumping could induce significant leakage away from the MAR scheme which cannot be recovered and could effect MAR viability.

This paper presents an analytical approach to assess these issues. Given the geological complexity of the site, a numerical modelling approach would obviously be preferred. However, constructing and running a numerical model and collecting the information necessary for proper calibration of the model is expensive. There is a need for a first pass assessment of the issues, to identify if there is any obvious impediments which may suggest that there is a low probability of the MAR scheme being successful at the site. Numerical modelling will be conducted in the next stage of the project if these initial results indicate at least a reasonable likelihood of success. The numerical results, when complete, will provide an interesting comparison to the analytical results presented in this paper.

METHODS

The primary analytical model used in this paper is the analytical solution of Zhan and Zlotnik (2002) to the two layer aquifer model, as presented in Figure 3. This aquifer system is similar to a Boulton-type semi-confined aquifer: it consists of a pumped aquifer overlain by either a single aquitard or a series of aquitards containing a standing water table. For small values of time after turning on the pump, the semi-confined pumped aquifer behaves as a fully confined (Theis-type) aquifer. At intermediate values of time water seeps downward through the overlying aquitard (or aquitards) to recharge the pumped aquifer, which appears during this second period to respond as a Hantush leaky aquifer. Finally, as pumping from the well continues, a third phase is reached in which free surface drawdowns in the top layer begin to drop and, ultimately, approach the same values that occur in the pumped aquifer. During this third phase vertical leakage decreases and drawdowns in the pumped aquifer approach values predicted from the Theis (1935) solution when the elastic storage coefficient (storativity) is replaced with the effective porosity (specific yield) of the material containing the watertable (Hunt, 2008).

Figure 3 – Conceptual Model for Analytical Assessment of Vertical Flow and Drawdown in aTwo-Aquifer System



The main limitation of the Boulton solution is that it is not applicable where other layers have a transmissivity of more than 5% of the pumped aquifer. This is because the solution cannot model the occurrence of horizontal flow in overlying strata. The effects of horizontal flow in overlying strata are accounted for however in the Zhan and Zlotnik (2002) solution. As described in Hunt (2008), programs have been written to calculate both the volume of water sourced from the pumped aquifer and the corresponding volume of water depleted from storage at the watertable. This paper uses Function.xls to call these functions in combination with the solution obtained by Zhan and Zlotnik (2002) to model flow volumes to the pumped well and drawdown in the overlying aquifer in this two-layer aquifer model. Using volumes derived from the watertable aquifer and drawdown in the watertable aquifer resultant from the borefield to the south, other simple groundwater equations are employed to assess the contribution of water from the MAR borefield and impact on travel times between the two borefields.

The second important tool which is drawn upon in this paper is the work of Ward et al (2009). This work examines the influence of three key factors which can lead to reduction in the recovery efficiency of ASR systems in brackish or saline aquifers: lateral flow, density-driven flow and dispersive mixing. Lateral flow refers to the process of the ambient groundwater gradient causing the injected water to drift away from the ASR well, such that recovery of viable volumes of the injected water is not possible. Dispersive mixing refers to the process of mixing between the advancing fluid (i.e. injected water) and the native groundwater. The dominant cause of this is mechanical dispersion caused by variable flow paths within the aquifer. Density driven mixing refers to mixing due to solute concentrations differences between the two waters, and can include the formation of an unstable fresh-salt interface, with the injected plume trying to ''float" upwards through the aquifer while the denser native groundwater sinks down and inwards, contaminating the ASR well at the bottom and significantly impact on recovery efficiency of the ASR well.

In Ward et al. (2009) four dimensionless parameters are defined to give an approximate characterisation of lateral flow, dispersive mixing, mixed convection (density effects during pumping) and free convection (density effects during storage). A set of numerical models spanning a wide parameter range is then used to develop a predictive framework using the dimensionless numbers. If the sum of the four dimensionless numbers (denoted R_{ASR}) exceeds 10, the ASR operation is likely to fail with no recoverable freshwater, while if $R_{ASR} < 0.1$, the ASR operation is likely to provide at least some recovery of freshwater Ward et al. (2009). There is insufficient scope within this paper to expand on the derivation or meaning behind the dimensionless numbers and the reader is encouraged to read Ward et al. (2009) for further details.

This paper calculates the lateral flow and the dispersive mixing dimensionless number as outlined in Ward et al. (2009) to determine the potential for either favourable or deleterious conditions for ASR at the site. Different ambient groundwater gradients resulting from different aquitard hydraulic conductivities and the groundwater pumping from the borefield to the south [as calculated using Zhan and Zlotnik (2002)] are used to derive different dimensionless numbers, and assess the potential significance of vertical leakage on recovery efficiency at the site. The two density related dimensionless numbers were not calculated, as it was assumed that the relatively small difference in density of the injected and receiving waters (likely to be approximately in the range of 100 – 500 mg/L TDS for the injected water compared to groundwater salinity around 1,300 – 1,400 mg/L TDS) will mean that this form of mixing will not be an issue.

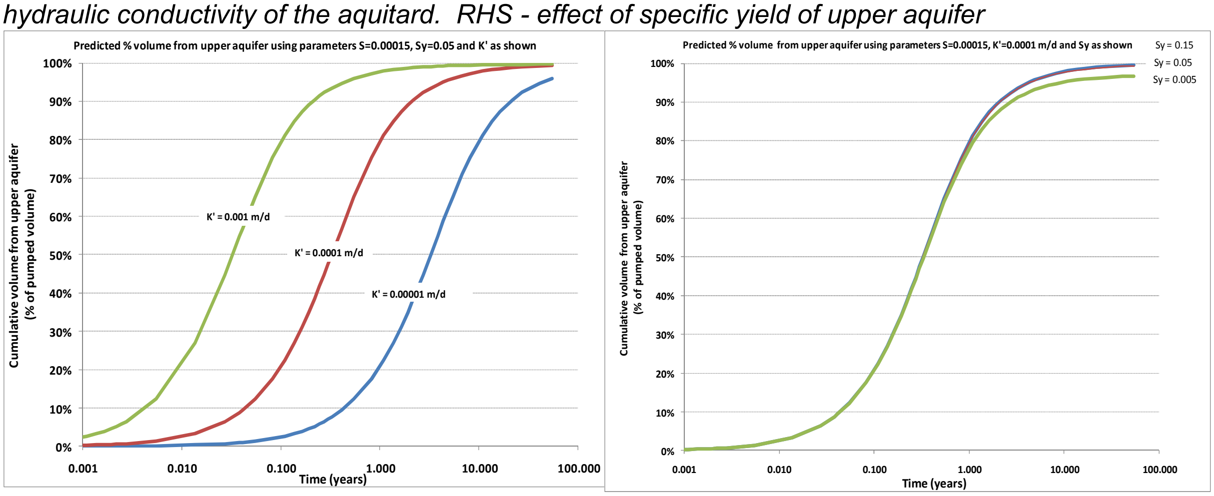
One of the inputs to the dispersivity dimensionless number is the longitudinal dispersivity. Unfortunately for most feasibility assessments of ASR, dispersivity is generally not known until field tracer tests have been carried out, at (or near) the scale of the problem of interest. Scale is very important in the case of dispersivity, as the resulting value of longitudinal dispersivity is found to be approximately correlated to the scale of the physical test. The empirical formula of Schulze-Makuch (2005) is used to provide an approximate upper estimate for dispersivity in order to define a conservative value for this dimensionless number (R_{DISP}) in this assessment.

RESULTS AND DISCUSSION

Impact of the MAR scheme on the Borefield – Volumetric Impact

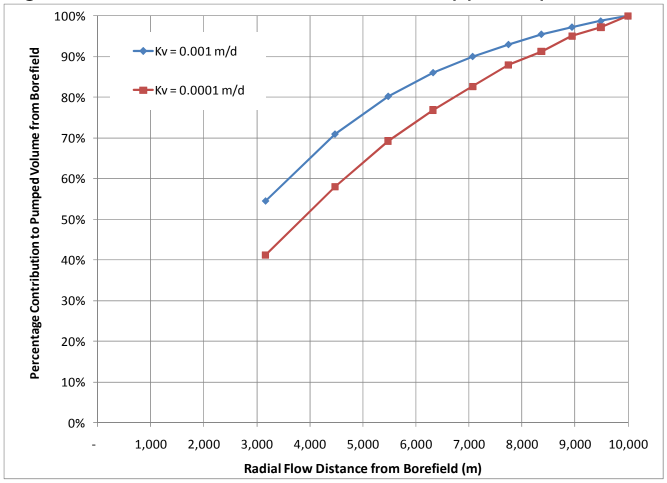
Using Zhan and Zlotik (2002) the volumes derived from the watertable aquifer over time, as a result of pumping from the borefield in the LEVF aquifer were calculated. The results are presented in Figure 4 below. Sensitivity testing for the impact of the aquitard hydraulic conductivity (refer Figure 4 LHS) and the specific yield of the upper aquifer (refer Figure 4 RHS) was undertaken. Figure indicates that, regardless of the hydrogeological parameters used, after a sufficient period of groundwater abstraction, the volume of drainage at the watertable predicted by this solution becomes insensitive to the parameter values (within realistic ranges). The influence of the transmissivity of the upper and low aquifers has not been presented in this analysis, because in this solution, the volume released is entirely independent of transmissivity. While the aquitard vertical hydraulic conductivity does display the most sensitivity to the volumes derived from the upper aquifer, in the long term (i.e. after 10-20 years), even the scenario with the lowest hydraulic conductivity indicates that more than 90% of water will be sourced from the upper aquifer.

Figure 4. Cumulative pumped volume (as percentage of total volume pumped) derived from the upper aquifer resultant from pumping in the lower (LEVF) aquifer. LHS - effect of vertical hydraulic conductivity of the aquitard. RHS - effect of specific yield of upper aquifer



Based on these results, the issue then became one of determining the magnitude of the contribution of different parts of the watertable aquifer to the borefield. Zhan and Zlotnik (2002) was used to determine the watertable gradient in different equal area portions of the upper aquifer (for two different aquitard hydraulic conductivities). This provided an indication of the flow contributions at different radial distances from the borefield (refer Figure 5). A practical cut-off of 10,000m radial impact was assumed (e.g due to the limited extent of the aquifer) even though the analytical model predicted an impact beyond this zone.

Figure 5 – Contribution of Flow from Upper Aquifer to Borefield at Different Radial Distances



The areal footprint of the injected water from the MAR scheme was conservatively estimated (i.e. maximum extent of plume) to sit over a radial distance of 3000 – 5000m from the borefield. As can be seen from Figure 5, this radial area contributes between 20-25% of flow to the borefield (with only a slight difference for the varying aquitard hydraulic conductivities). However, the MAR scheme footprint only occupies 8% of this radial circumference and hence the contribution from the area covered by the MAR scheme is less than 2% (i.e 8% of 20-25%) of flow at the borefield. In other words, the flow at the water supply borefield to the south could not be comprised of more than 2% of water from the MAR scheme. Hence it is considered extremely unlikely that over the long term any ‘contamination’ of the water supply borefield from transfer of conservative species in the MAR injected water could occur, as the potential contribution to total bore field flow is very small.

An important assumption inherent in this calculation is that the long term impressed head (i.e watertable rise) from any MAR operations is effectively zero, as the injection impact is cancelled out by the extraction.

Impact of the MAR scheme on the Borefield – Travel Times

In terms of the possible impact from species such as pathogens, which may be present in the source water, this is essentially a question of travel time between the MAR borefield and the water supply borefield. (Although this risk is already considered very low, as the water will most likely be treated for pathogens prior to injection, this assessment covers the scenario of a failure in the pre-treatment process).

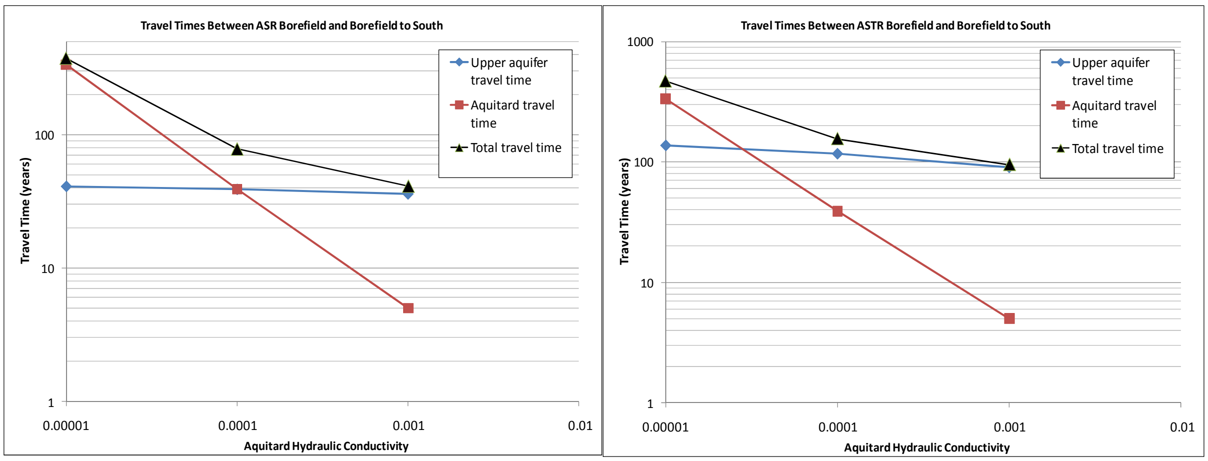
The flow path for the injected water to the borefield is laterally along the UEVF aquifer (approximately 4,000 metres) and then vertically through the MEVF aquitard (conservatively assumed to be 50m for this assessment). Another theoretical flow path is for the injected water to migrate vertically through the MEVF and then laterally within the LEVF aquifer to the bore field. However this second flow path is very unlikely given the upward gradient from the UEVF to the LEVF, as only in relatively close proximity to the borefield will there be a downward gradient across the aquitard. Hence this section presents the results of various travel times for this first mentioned flow path, for various aquitard hydraulic conductivities.

The gradients within the UEVF are a combination of the natural gradient (approximately 4m over 4,000 m) and the induced gradient, firstly from the bore field to the south (calculated using Zhan and Zlotnik, 2002), and secondly from the MAR scheme itself. The gradients across the aquitard are calculated based on the natural gradient (assumed to be 12m, upward) and the gradients induced by the borefield pumping (downward), which change for the various aquitard hydraulic conductivity scenarios. Once the gradients are determined, the travel times are calculated using the Darcy flow velocity equation ($V = Ki/n_e$).

The results are presented below for a 5 GL/yr ASR (Aquifer Storage and Recovery) and a 5 GL/yr ASTR (Aquifer Storage, Transfer and Recovery) scheme in Figure 6. The graphs show the split between the aquitard travel time and the aquifer travel time. The travel times from the ASR scheme are less than the ASTR scheme, as the ASTR scheme operates largely in net equilibrium. A 1.5m impressed head was used in the calculation, resulting from an assumed 10% (500 ML/yr) which was not recoverable in the ASTR calculation. In contrast the ASR scheme was assumed to operate for 6 months of the year, with an impressed head which over the next six months is reduced to (close to) zero during the extraction phase. Hence for half the year there is an increased gradient from the ASR scheme to the borefield. From analytical modelling, using Thies (1935), the average impressed head across the year was calculated to be 15m.

The results indicate that total travel times between the MAR bores and the borefield (for both ASR and ASTR schemes) are very long, and hence the risk even from extremely resistant pathogens, is practically zero. While for the upper end estimate of aquitard hydraulic conductivity (0.001 m/d) the travel time through the aquitard (4 years) may appear relatively short, this is in fact still a very long travel time (in terms of pathogen die off), and when combined with the 30+ year of travel time through the UEVF aquifer, the total travel time for this scenario is 40+ years and the risks from any residual species in injected water reliant on biodegradation and die-off are therefore negligible.

Figure 6 – LHS: Travel times between ASR borefield and bore field to south. RHS: Travel times between ASTR borefield and bore field to south



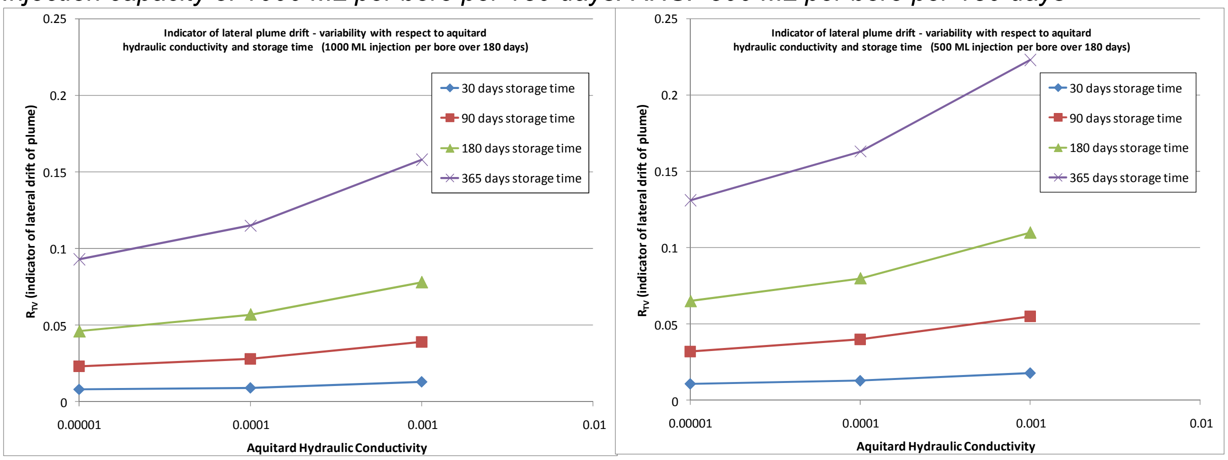
Impact of the Borefield on Recovery Efficiency of the MAR scheme
Figure 7 presents the results of the calculation of the Ward et al. (2009) dimensionless parameter R_{TV} for different aquitard hydraulic conductivities and different storage intervals prior to extraction. The different aquitard hydraulic conductivities alter the ambient groundwater in the upper aquifer (due to the influence of the borefield pumping to the south), which is an input to the calculation of R_{TV}. (The impressed heads resultant from the ASR scheme were not included as the input required is the ambient gradient only). The R_{TV} parameter provides an indication of the likelihood of lateral flow being problematic in terms of recovery efficiency of the ASR scheme. Ward et al. (2009) suggest that sites that have R_{TV} values less than 0.1 are likely to be favourable in terms of lateral drift of the injected water, and sites greater than 10 are likely to be unfavourable.

The results indicate that most scenarios are likely to be favourable (independent of aquitard hydraulic conductivity), with the exception being for long storage times (one year). For the highest aquitard hydraulic conductivity of 0.001 m/day, for bore injection rates of 500 ML/yr, the R_{TV} marginally exceeded 0.1. However, all results are well below 10, indicating the likelihood of major recovery issues related to lateral drift is unlikely. They also indicate that the storage time of the injected water is likely to be more important than the additional gradient in the upper from the borefield pumping (and associated aquitard hydraulic conductivity) for lateral drift of the plume.

The dimensionless number for dispersive mixing was also calculated. The number was relatively insensitive to changes in the ambient gradient, with changes in the gradient apparently cancelled out by other inputs to the R_{DISP} equation. Values of R_{DISP} of around 0.045 were calculated for the 500 ML/yr/bore scenario and 0.063 for the 1,000 ML/yr/bore scenario. Given that both of these values are below 0.1, Ward et al. (2009) suggest significant recovery issues related to dispersive mixing are unlikely, and indeed conditions appear relatively favourable.

Assuming that the two dimensionless numbers related to density driven mixing are essentially nil (refer to *Method* above for rationale), the overall dimensionless number, R_{ASR} (which reduces to $R_{TV} + R_{DISP}$) is typically less than 0.1 for most scenarios, although for storage times of more than 180 days, R_{ASR} will slightly exceed 0.1, but is still well below the value of 10 for which Ward et al. (2009) suggest conditions are likely to be unfavourable. The analysis indicates that there is at least a reasonable likelihood of favourable conditions for recovery of the injected water and therefore that continuing on to detailed numerical modelling of recovery efficiency is warranted.

Figure 7 – Indicator of potential ASR recovery efficiency issues due to lateral drift of plume. LHS: Injection capacity of 1000 ML per bore per 180 days. RHS: 500 ML per bore per 180 days



CONCLUSIONS

This paper has investigated the potential impact of an MAR scheme located in an upper aquifer on a water supply borefield screened in an underlying aquifer, by assessing vertical fluxes and drawdown in the upper aquifer using a two layer dimensional analytical model (Zhan and Zlotnik, 2002). While over time, the modelling suggests that most of the water to the borefield is sourced from the upper aquifer, the maximum potential contribution from the MAR scheme to the borefield is less than 2%, due to the distance of the borefield from the MAR scheme. It was further shown than minimum travel times from the MAR site to the borefield were 40+ years, and therefore the risk of any substances or species reliant on biodegradation and die off (e.g. pathogens) surviving to the borefield is practically nil.

Using the dimensionless number developed by Ward et al. (2009) in combination with the analytical modelling, it was shown that the risk of significant recovery efficiency issues was relatively low, and indeed site conditions appeared reasonable favourable for good recovery of the injected water. Gradients induced by the borefield in the upper aquifer and the associated influence of vertical hydraulic conductivity on these gradients, were generally less important than the storage time of the water. Storage times of more than half a year may result in reduced recovery efficiencies.

All of the above analysis is subject to potentially significant errors related to the assumptions inherent in the analytical model (e.g. infinite lateral extent of the aquifer). However, it has proved useful as a first pass tool for indicating that the likelihood of impacts on the borefield are very low and that that there is at least a reasonable likelihood of favourable conditions for recovery of the injected water. Therefore continuing on to detailed numerical modelling of the scheme is warranted.

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Commonality between MAR water drainage and gas oil gravity drainage: the case of a fractured rock

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Abstract. In MAR schemes planned in Oman, seasonal retrieval of water from aquifers during summer incorporates drainage of highly conductive fractures and adjacent rock blocks (clay lenses) of low permeability. Similarly, in enhanced oil recovery by Gas-Oil Gravity Drainage (GOGD) oil rapidly drains through fractures and slowly retreats from matrix blocks. In both MAR and GOGD one immiscible phase displaces another in a spatially non-uniform manner with the following controlling factors: block/fracture sizes/aperture, capillary-pressure function, relative permeabilities and phase viscosities-densities. The difference between MAR and GOGD is in the perception of aquifer (water): in groundwater hydrology it is an entity to be replenished and protected, in GOGD it is a nuisance causing deterioration in oil production (e.g. upconing). In this work we illustrate the physical and mathematical commonality between hydrological (MAR) and reservoir (GOGD) engineering problems of modeling the drainage of rectangular porous blocks. The left and right block faces are in contact with a film flow of a heavy fluid (liquid), which re-imbibes into the block through the upper segment of the matrix-fracture contact surface and re-drains through a lower segment (the subjacent-superjacent blocks are separated by impermeable barriers). Analytical solutions for steady-state, essentially 2-D liquid motion are obtained for the Vedernikov model of capillarity by conformal mappings and solution of boundary-value problems involving singular integrals. Numerical solutions by HYDRUS2D for VanGenuchten capillarity-permeability functions are in qualitative agreement with the Vedernikov model.

Key Words: drainage, matrix, fracture, capillarity, Darcy law, Vedernikov-Richards' models

Introduction

Standard models of MAR in groundwater hydrology assume either a homogeneous porous medium or a Barenblatt-Kazemi (BK) double-continuum medium as a compartment into which water is injected or infiltrated from recharge wells or surface basins (e.g. Barenblatt et al., 1990). Real MAR aquifer zones are seldom homogeneous on the scale of operation. They are however not so large to allow for many blocks to be spatially averaged into a BK system, as is done on oil field scales. In other words, the size of, say, aquifer limestone or sandstone blocks is so large compared to an infiltration basin such that seepage of recharged water there should be scrutinized in relation to motion of water in ad-, super, -sub- jacent fractures surrounding the block. A typical situation is shown in Fig.1. Here a stack of permeable homogeneous blocks is bordered on the left and right by vertical fractures. A liquid moves vertically downward in the direction of the gravity acceleration vector g. The liquid films partially fill the fracture and contact the block walls. This contact causes slow circulatory-type motion of water similarly to common aquifers (Kacimov, 2006).

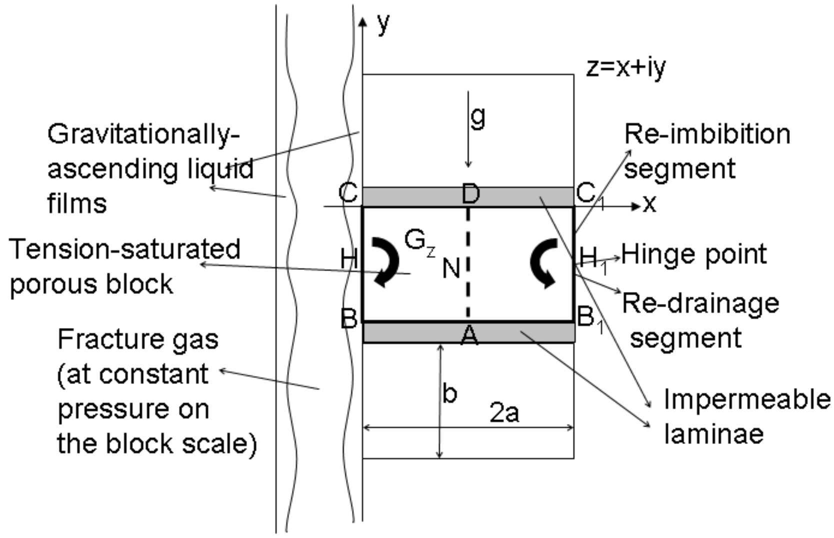

Fig.1

This re-imbibition and re-drainage through the film-porous block contacts is characterized by so-called hinge lines (hinge points H and H_1 in Fig.1). 2-3-D topology of streamlines, isobars and equipotential lines (surfaces) in the block $ABCDC_1B_1A$ emerges. The driving force for liquid to be sucked by the porous medium is capillary tension. A capillary fringe boundary and "gas bubble" can appear in the vicinity of point D in Fig.1 if the blocks are separated by impermeable barriers (a scheme adopted in this paper; "partial barriers" between the blocks can be also tackled by our model).

Gas oil gravity drainage in fractured formations (GOGD-FF) is a mechanism of secondary oil recovery from carbonate rock interspersed by a network of horizontal and vertical fractures similar to the fractured aquifer depicted in Fig.1. Gas as a non-wetting fluid is injected from the pay top and rapidly displaces a wetting oil from vertical fractures with a much slower displacement through blocks. Block-originated oil exudes to the fractures, both horizontal and vertical, and descends through the latter to the so-called oil rim where a horizontal well pumps it out. GOGD phenomenon has been thoroughly investigated by reservoir engineers (see, e.g., Bech et al. 1991, Boerrigter et al., 2007, Evans, 1982, Ghazvini et al., 2009, Habsi et al. 2008, Horie et al. 1990, Labastie 1990, Nejadi and Sajjadiaun 2008, Shariat et al. 2006, Stones 1992, Wit et al. 2002, Ypma 1985).

The petroleum engineering community has not, however, recognized that GOGD-FF is physically and mathematically similar to standard problems of water drainage in MAR. The latter has been studied in civil and agricultural engineering by analytical and numerical models (e.g., Polubarinova-Kochina, 1977, Vedernikov, 1939). To the best of our knowledge, civil engineers dealing with MAR, in particular, with air-water seepage in the vadose and saturated zone of unconfined aquifers are also unaware of GOGD-FF theoretical and practical accomplishments in reservoir engineering. So, the objective of this paper is to bridge the gap between petroleum and civil engineers studying essentially the same processes (oil-gas and water-air motion) and to illustrate that standard MAR models (e.g. the Vedernikov and Richards ones) of gravitational water drainage can serve in predictions of oil recovery through fractured pays.

Methods

We use mathematical modeling and assume that flow in blocks of Fig.1 is Darcian, the fluid is incompressible, the block is rigid, homogeneous and isotropic.

The stack of rectangular blocks in Fig.1 corresponds to the scheme of Saidi et al. (1979). The origin of a Cartesian coordinate system is at point C. We assume that blocks' size in the direction perpendicular to Fig.1 is high such that flow is 2-D in the vertical plane xCy. Due to symmetry with respect to DA in Fig.1 we consider only a half-block, ABCD (zoomed in Fig.2). The sizes of this rectangle are a and b.

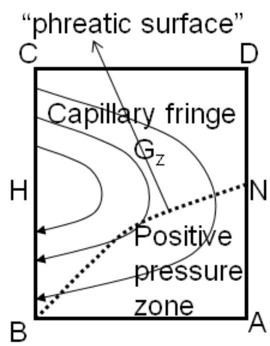

Fig.2

The vertical face BC is a vertical fracture wall. Liquid, oozed from the blocks stacked above BC, drains down as a fracture film, partially intercepted by the block through CH. By conservation of mass, the same liquid quantity is released back into the fracture through HB. The horizontal segments CD and AB are impermeable (e.g. shale or clay layers) and AD is a no-flow line due to symmetry. The dotted line BN is a "phreatic surface" in the sense of Polubarinova-Kochina (1977) i.e. pressure along this line is atmospheric=0. The same zero pressure holds in the unfilled parts of the fractures of Fig.1. Some streamlines (arrowed curves in Fig.2) intersect BN and others do not. A capillary fringe (negative pressure zone) is above BN. Below BN in Fig.2 we have an unconfined "mini-aquifer" on the scale of the block. Points A and D are hydrodynamic stagnation points. In GOGD-FR modeling, Saidi et al. (1979) for the cascade of blocks in Fig.1 experimentally illustrated that oil is reimbibed and redrained into the blocks through BC.

We will take into account capillarity by the so-called Vedernikov (1939) model . The crux of this model, utilised in theoretical studies of capillary fringes abopve the water-table capped groundwater domains, is in the assumption that in the fringe (tension-saturated zone) the hydraulic conductivity is constant but water pressure, p is negative (water is at tension) until a limit of $-p_c$ is reached. The positive constant p_c is a property of the rock (soil) tabulated elsewhere (e.g. Polubarinova-Kochina, 1977). If $p<-p_c$ a porous medium drains instantaneously and hydraulic conductivity drops to zero. The two zones (tension saturated, $p>-p_c$, and dry, $p<-p_c$) are separated by a sharp interface, which is a mathematical free boundary of the problem. In groundwater hydrology and soil physics the lines $p=-p_c$ and p=0 demarcate the capillary fringe boundary and phreatic surface, respectively. The latter in Fig.2 is not a mathematical free boundary of the Vedernikov model.

We present here the simplest case with no free (fringe) boundary, i.e. there is no dry porous zone within the blocks of Fig.1. Although this regime is simplistic, it illustrates all the major flow features which also appear in the schemes with a "dry bubble" near point D in Fig.1 and in genuinely unsaturated flows modeled by the Richards equation. The block half- width a in Fig.1a should be small enough for no-fringe boundary regimes to occur.

Water in the fractures of Fig. 1 is not pressurised and, therefore, in the fractures we can assume that oil pressure is equal to the zero air pressure in the fracture. The air pressure gradient from C to B in Fig.1 is negligible that is reasonable because b in real aquifers/pays is of a size of meters.

We introduce the complex physical coordinate z=x+iy in the rectangle of the flow domain, G_z. According to the Darcy law, the Darcian velocity, of water in the blocks V(x,y) is:

$$\vec{V} = -k\nabla h(x,y) = \nabla\phi(x,y) \tag{1}$$

where the velocity potential $\phi(x,y)$ is defined through pressure as

$$p=-\rho\, g(\phi(/k+y) \tag{2}$$

where k=const is hydraulic conductivity, ρ=const is water density and h(x,y) is the total hydraulic head.

We introduce a stream function $\psi(x,y)$ connected with ϕ by the Cauchy-Riemann equations. The complex potential is defined as $w=\phi+i\,\psi$. This function is holomoprhic in G_z. The horizontal, u(x,y), and vertical, v(x,y), components of V are $u=\phi_x$, $v=\phi_y$ where the subscripts indicate partial differentiation. The complexified Darcian velocity u+iv is antiholomorphic in G_z.

It is well known (Polubarinova-Kochina, 1977) that ϕ and ψ are defined up to an arbitrary constant. As the origin of the coordinate system is at point C in Fig.1a, we set $\phi_c=0$ according to eqn.(2) . Along the streamline CDAB we set $\psi=0$. Then the domain G_w, corresponding to G_z in the w- plane, is sketched in Fig.3a. We note that the image of the curve CHB in G_w is not known in advance. In particular, we do not know what is the inflow-outflow rate Q of water into-out of the block. From eqn.(2) we, however, know that ϕ_B=kb.

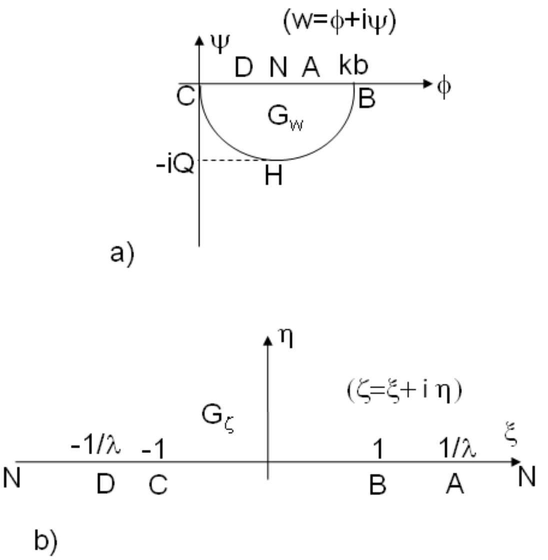

Fig.3

We introduce dimensionless variables as: (x_d, y_d, a_d, ϕ_d , ψ_d, p_d, u_d, v_d, Q) =(x/b, y/b, a/d, ϕ /(k b), ψ /(kb), p/ (ρ g b), u/(k b), v/(k b), Q/(k b)) and for the sake of brevity drop the subindex d (indicates nondimensionality).

The governing equation for water motion in G_z is:

$$\Delta\phi(x,y)=0 \tag{3}$$

Isobaricity of CB and no-flow condition along CDAB give the following corresponding boundary conditions:

$$\phi(x,y)=-y,\quad \psi=0 \tag{4}$$

Eqns.(3)-(4) make a mixed boundary-value problem, which we solve by the Polubarinova-Kochina (1977) method. We introduce an auxiliary complex variable $\zeta=\xi+i\eta$ and map conformally G_z onto the half-plane $\operatorname{Im}\zeta>0$ (Fig.3b) with the correspondence of points: $B,C\to\pm1, A,D\to\pm1/\lambda, N\to\infty$ where $0<\lambda<1$ is a constant to be found later and N is the middle point of the face AD. The Schwartz-Christoffel formula (Polubarinova-Kochina, 1977) gives:

$$z(\zeta)=A_m\int_{-1}^{\varsigma}\frac{d\tau}{\sqrt{(1-\tau^2)(1-\lambda^2\tau^2)}} \tag{5}$$

The mapping constant A_m in eqn.(5) and λ are found from the following geometrical conditions z=-i at z=1 and z=a-i at $\zeta = 1/\lambda$ as:

$$A_m = -\frac{i}{2K}, \quad \frac{\mathrm{K}}{\mathrm{K}'} = \frac{1}{2a} \tag{6}$$

where K and K' are the complete elliptic integrals of the first type with moduli λ and $\lambda' = \sqrt{1-\lambda^2}$. The modulus λ is determined from egn. (7) using the Wolfram (1991) *Matehmatica* built-in function **FindRoot**. Thus, the first holomorphic function z(ζ) is determined.

The second function w(ζ) is reconstructed by the Signorini formula (see Polubarinova-Kochina, 1977, Kacimov, 1996, 2006) as

$$w(\zeta) = \frac{\sqrt{1-\zeta^2}}{\pi i} \int_{-1}^{1} \frac{\phi(\tau) d\tau}{\sqrt{1-\tau^2}(\tau - \zeta)} \tag{7}$$

According to eqns (4)-(5) the kernel of the integral in eqn.(7) is:

$$\phi(\tau) = \frac{1}{2K} \int_{-1}^{\tau} \frac{ds}{\sqrt{(1-s^2)(1-\lambda^2 s^2)}} \tag{8}$$

The Sokhotsky-Plemelj limit applied to eqn.(7) leads to the following stream function along BC:

$$\psi(\xi) = \frac{\sqrt{1-\xi^2}}{\pi} \int_{-1}^{1} \frac{\phi(\tau) d\tau}{\sqrt{1-\tau^2}(\tau - \xi)}, \quad \phi(\tau) = \frac{F[\arcsin \xi, \lambda] + K}{2K} \tag{9}$$

where F is the incomplete elliptic integral of the first type with the modulus λ .

From eqn.(9) at $\xi = 0$ we immediately get the flow rate through the half-block:

$$Q = \frac{1}{\pi} \int_{-1}^{1} \frac{\phi(\tau) d\tau}{\sqrt{1-\tau^2}\,\tau}$$

Expressions similar to eqn.(9) can be written from eqn.(7) for $\phi(\xi)$, and $p(\xi)$ along CDAB. According to the maximum principle (Polubarinova-Kochina, 1977) the harmonic function p(x,y) reaches its minimum on the boundary of G_z and therefore the minimum, p_{min}, along CDAB should be checked for p_{min} >-p_c. If this inequality does not hold, then a free (capillary fringe) boundary is formed. In our case p_{min} is attained at point D and therefore the no-dry-zone condition is very simple $\phi_D < p_C$ that can be checked straight from eqn.(7). Pressure decreases from point C to point D, increases from D to A (almost hydrostatically) and decreases again from A to B.

In computations, we treated the singular (Cauchy-type) integral in eqn.(9) in the sense of principal value along CB. For this purpose we engaged the *Mathematica* routine **CauchyPrincipalValue**. All other integrals following from eqn.(7) are regular and were tackled by the **NIntegrate** routine.

Results

As an example Fig.4 shows the distribution of the stream function along BC for $\lambda=0.8$ (a=1.47).

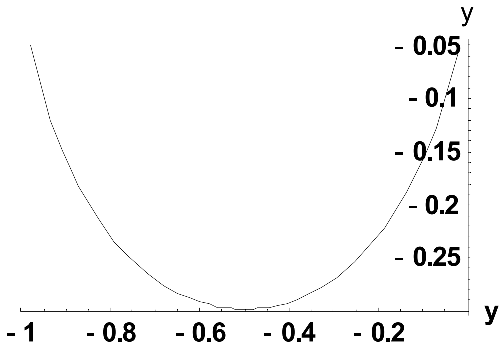

Fig.4

This curve has a single minimum corresponding to point N where $\psi = -Q$ (see Fig.3a). Fig.5 represents Q(a).

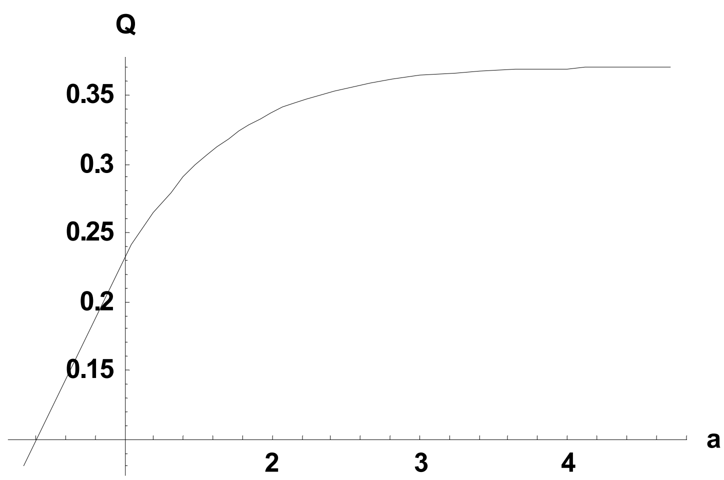

Fig.5

We recall that if a is large a dry "bubble" appears and the solution given above is not valid.

Using HYDRUS2D we studied numerically flow in ABCD of Fig.2 using the Richards equation. Qualitatively the flow pattern in the block and pressure distribution is the same as in the Vedernikov model.

Conclusions

A simple analytical solution is developed for 1-phase, 2-D water motion in a rectangular matrix block, induced by an isobaric film in the fracture. This film causes "shearing" of a Darcian flow inside the block. An internal phreatic surface in the block separates a saturated and tension-saturated zone. Using the Vedernikov model and theory of holomorphic functions we found explicit expressions of the complex potential and complex physical coordinate of the flow through an auxiliary variable. Parametric equations for the stream function, hydraulic head and pressure involve elliptic functions and Cauchy type integrals. Oil flow in GOGD is analogous to the studied water motion, which is controlled by capillarity, gravity and matrix resistance. In both cases an infinite mobility of air (gas) is assumed.

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Optimal Management of Groundwater Resources in Coastal Aquifers Subjected to Seawater Intrusion

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ABSTRACT: Management of water resources in coastal areas is very important because these areas are heavily urbanized with increasing water demand. Seawater intrusion is one of the most important environmental problems that degrade groundwater resources in these areas. It threatens groundwater resources, because mixing a small quantity of seawater with groundwater makes it unsuitable and can result in abandonment of aquifers. Therefore, saltwater intrusion should be prevented or at least controlled to protect groundwater resources. This paper presents a simulation-optimization model to study the control of seawater intrusion in coastal aquifers using three management scenarios; abstraction of brackish water, recharge of fresh water and combination of abstraction and recharge. The objectives and constraints of these management scenarios include minimizing the total costs for construction and operation and minimizing salt concentrations in the aquifer by determining the optimal depth, location and abstraction/recharge rates. The developed model is based on the integration of a genetic algorithm optimization technique and a density-dependant finite element model for simulation of fluid flow and solute transport in soils. The simulation-optimization model is applied to a real case study. The results of the model are presented and discussed. The efficiencies of the three management scenarios are examined and compared. The results show that all three scenarios could be effective but using a combination of abstraction and recharge results in the lowest cost and salt concentration in the aquifer and maximum movement of freshwater/saline water interface toward the sea. This study presents a management model to control saltwater intrusion considering economical and environmental impacts that can be applied in areas where there is a risk of saltwater intrusion. It can be used for sustainable development of water resources in coastal areas.
Keywords: coastal aquifers, genetic algorithm, seawater intrusion, simulation-optimization.

Introduction

Groundwater resource planning in coastal areas requires an approach similar to water resources planning inland such as; aquifer yield, pumping interface, aquifer-stream interaction and contamination. In aquifers situated along the coast, saltwater intrusion adds a significant complication to the process of aquifer planning (Cheng and Ouazar, 2003). Management of coastal aquifers subjected to saltwater intrusion is therefore a challenging task and requires efficient and cost effective measures to control seawater intrusion in order to protect groundwater resources from depletion. Todd (1974) discussed various means of preventing saltwater intrusion including: reduction of the abstraction rates, relocation of abstraction wells, subsurface barriers, natural recharge, artificial recharge, abstraction of saline water, and combination systems. Numerical simulation models can be used to examine a limited number of design options by trial and error. However, optimization methods can be combined with simulation models to search for the optimal solution. The motive for the development of the simulation-optimization S/O model to control the seawater intrusion is the enormous cost of groundwater remediation. The S/O approach can be used efficiently in groundwater remediation system design and also in other groundwater quality management problems. In the last two decades, genetic algorithms (GAs) have received increased attention from academic and industrial communities for dealing with a wide range of optimization problems. GAs have been successfully applied to a number of groundwater management problems including; maximizing extraction from aquifers, minimizing the cost of aquifer remediation and pump-and-treat. Development and application of optimization techniques in association with seawater intrusion was presented by a number of researchers (e.g. Cheng et al., 2000; Das and Datta, 2000 and Bhattacharjya and Datta, 2005).

A limited number of studies have concentrated on the management of coastal aquifers considering the intrusion of seawater. To the authors' knowledge, no attempt has been made in the literature to develop or use simulation-optimization models to study optimal control of seawater intrusion using a combination of abstraction and recharge wells. Rastogi et al (2004) used a simulation model to study the effect of abstraction and recharge on the intrusion of seawater by trial-and-error. To date, no work has been done on reduction of costs of construction and operation of such systems. This paper presents the development and application of S/O model to control seawater intrusion in coastal aquifers. The simulation model is integrated with a genetic algorithm (GA) optimization model to study the control of seawater intrusion in coastal aquifers using three management scenarios; abstraction of brackish water, recharge of fresh water and combination of abstraction and recharge. The objectives of these management scenarios include minimizing the total construction and operation cost and minimizing salt concentrations in the aquifer by determining the optimal depth, location and abstraction/recharge rates. The developed model is applied to a number of cases studies one of which is presented in this paper. The efficiencies of the three management scenarios are examined and compared for the presented case study.

Development of a simulation-optimization model

In this work a simulation-optimization model is developed based on the integration of a GA with a coupled transient density-dependent finite element model for flow and solute transport in soils. In the developed framework, the simulation model is repeatedly called by the GA to calculate the response of the system to each set of design variables generated by the GA. The simulation model is used to compute hydraulic heads and salt concentrations at every node in the domain. These values for heads and salt concentrations are returned to the GA routine and used to evaluate the objective function value and determine the fitness of each solution in competition with other generated solutions. The simulation and optimization models are presented below.

Simulation model

The governing deferential equations for water flow and air flow are obtained, based on conservation of mass in terms of two primary variables; pore-water pressure (u_w) and pore air pressure (u_a) (Javadi et al., 2008):

Water flow equation

$$C_{ww}\frac{\partial u_w}{\partial t}+C_{wa}\frac{\partial u_a}{\partial t}=\nabla[K_{ww}\nabla u_w]+\nabla[K_{wa}\nabla u_a]+J_w \tag{1}$$

Air flow equation

$$C_{aw}\frac{\partial u_w}{\partial t}+C_{aa}\frac{\partial u_a}{\partial t}=\nabla\left(K_{aw}\nabla u_w\right)+\nabla\left(K_{aa}\nabla u_a\right)+J_a \tag{2}$$

The governing deferential equations of solute transport in porous media including various processes such as; advection, diffusion, dispersion, adsorption, chemical reaction and biological degradation can be expressed as (Javadi et al., 2008):

$$\frac{\partial((R\theta_w+H\theta_a)c_w)}{\partial t}+\nabla(v_w c)+\nabla(v_a c)-\nabla(\theta_w D_w\nabla c)-\nabla(\theta_a D_a\nabla c)+(\lambda_w\theta_w+H\lambda_a\theta_a)Rc=0 \tag{3}$$

The coupling of fluid flow and solute transport in unsaturated soil is modeled using two sets of equations. The first set of equations describes water flow and air flow while the second set describes solute transport. Numerical simulation of seawater intrusion, assuming that mixing occurs at the transition zone between seawater and freshwater due to hydrodynamic dispersion, involves the solution of the partial differential equations representing the conservation of mass for the variable-density fluid (flow equations) and for the dissolved solute (transport equation). The nonlinear governing differential equations of fluid flow and solute transport are solved in space using the finite element method and in time using a finite difference scheme. The flow and transport equations are coupled through Darcy's law and a constitutive equation relating fluid density to salt concentration $\rho = \rho_f (1 + \varepsilon C)$ where, $\varepsilon = (\rho_s - \rho_f)/\rho_f$, ρ_f, ρ_s are the freshwater and the seawater densities respectively $[M][L]^{-3}$. The numerical solution of coupled fluid flow is based on solving the governing equations with appropriate boundary and initial conditions to compute pore water and air pressures at all nodes. The pressure head, fluid velocity and fluid density are then calculated. The calculated velocities are used to define the dispersion coefficient for the solute transport equation. The solute transport equation is then solved for concentrations at every node in the domain and this process is repeated at every time step.

Genetic algorithm optimization model

An appropriate management strategy can provide an efficient and cost-effective approach to control seawater intrusion in coastal aquifers. A GA-based optimization approach is used in this work to incorporate a simulation model within an optimization-based management model to evolve an optimal management strategy. Three management models are developed in this work to control seawater intrusion in coastal aquifers; abstraction of brackish water, recharge of fresh water and combination between abstraction and recharge. The main objective of the simulation-optimization model is to minimize the construction and operation costs by identifying the optimal depth, location and abstraction/recharge rates for abstraction and/or recharge wells to control the intrusion of seawater. The objective functions of the three models can be represented as (Javadi et al., 2010):

Management model 1: (Abstraction wells)

$$\text{Min} \quad f = P_1 \sum_{i=1}^{N} C_i + P_2 \sum_{i=1}^{N} Q_{A_i} (C_A + C_T) + P_3 \sum_{i=1}^{N} D_{A_i} (C_{DW}) \tag{4}$$

Management model 2: (Recharge wells)

$$\text{Min} \quad f = P_1 \sum_{i=1}^{N} C_i + P_4 \sum_{i=1}^{N} Q_{R_i} (C_{PW} + C_R) + P_5 \sum_{i=1}^{N} D_{R_i} (C_{DW}) \tag{5}$$

Management model 3: (Abstraction and Recharge wells)

$$\text{Min} \quad f = P_1 \sum_{i=1}^{N} C_i + P_2 \sum_{i=1}^{N} Q_{A_i} (C_A + C_T) + P_3 \sum_{i=1}^{N} D_{A_i} (C_{DW}) P_4 \sum_{i=1}^{N} Q_{R_i} (C_R) + P_5 \sum_{i=1}^{N} D_{R_i} (C_{DW}) \tag{6}$$

where
f : is the objective function in terms of the total cost
N : is the total number of nodes in the domain
D_A : is the depth of abstraction well (m)
D_R : is the depth of recharge well (m)
Q_R : is the recharge rate (m^3/sec)
Q_A : is the abstraction rate (m^3/sec)
c_i : is total amount of solute mass in the aquifer (mg/l)
C_A : is the cost of abstraction ($/$m^3$)

C_T : is the cost of treatment ($/m^3)
C_R : is the cost of recharge ($/m^3)
C_{PW} : is the price of water ($/m^3)
C_{DW} : is the cost of installation/drilling of well ($/m)
P_1, P_2, P_3, P_4 and P_5: are the weighting parameters

The management objectives are achieved within a set of side constraints for well depth, well location and abstraction rate as: $Q_{\min} < Q_i < Q_{\max}$, $L_{\min} < L_i < L_{\max}$ and $D_{\min} < D_i < D_{\max}$. In these management models the costs are introduced based on the available data from literature. According to the literature these costs are considered as; cost of installation/drilling of well per meter depth: $1000, cost of abstraction per cubic meter: $0.42, cost of recharge per cubic meter: $0.48, cost of treatment (i.e. desalination) per cubic meter: $0.6 and price of water per cubic meter: $1.5 (Qahman, 2004; Qahman, and Larabi, 2005).

Application of the simulation-optimization model to control seawater intrusion

In this paper the application of the developed simulation-optimization model to a real case study case study is presented. In this case the developed simulation-optimization model is applied to study the control of saltwater intrusion in Biscayne aquifer at Coulter area, Florida, USA (Lee and Cheng 1974). The aquifer average depth is 30 m below mean sea level and the width is 300 m. The domain is discretized using two dimensional grid ($\Delta X = \Delta Y = 6$ m). The mesh consists of 250 elements and 861 nodes. Coefficient of permeability of 3×10^{-10} m/sec and porosity of 0.39 were considered. The longitudinal and transverse dispersivities α_L and α_T were taken as 5.0 and 0.5 m, respectively. The density of fresh water ρ_f and seawater ρ_s are 1.0 and 1.025 t/m^3 respectively (Lee and Cheng 1974). The schematic sketch of the Biscayne aquifer and the corresponding boundary conditions applied in this case are shown in Figure 1.

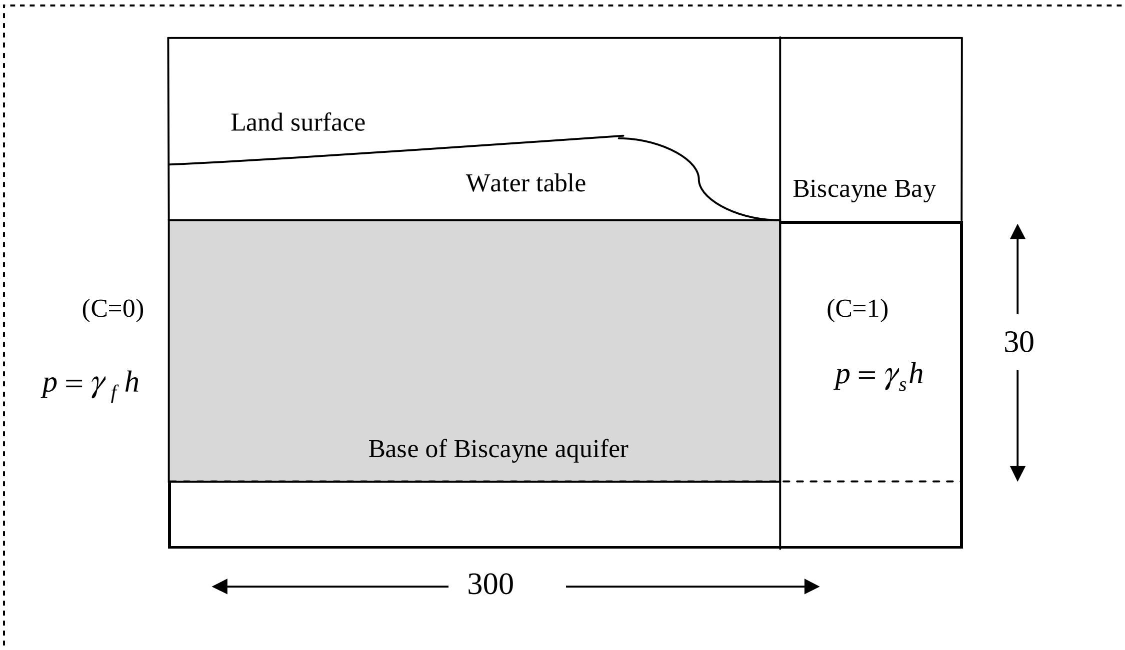


Figure 1 Schematic sketch and boundary conditions of Biscayne aquifer, Florida

The analysis has been done on two steps. In the first step, the simulation model is applied with the initial and boundary conditions to compute pressure head and salt concentration at every node in the interred domain. The second step, the S/O is applied to determine the optimum values of the decision variables; well depth, location and abstraction/recharge rates subject to constraints which have been defined for the three management models presented above. The objective function for the three models is to minimize the total cost associated with well locations, depths and abstraction/recharge rates and treatment. The results of the first step are shown in Figure 2 as the 0.25, 0.5, 0.75 and 1.0 isochlor contour lines. The results obtained from the S/O model for the three management models are presented in Table 1. The 0.5 isochlor lines for the three management models are shown in Figure 3. Comparison between total costs, salt concentration in the aquifer and abstraction/recharge rates for the three management models are presented in Figures 4.

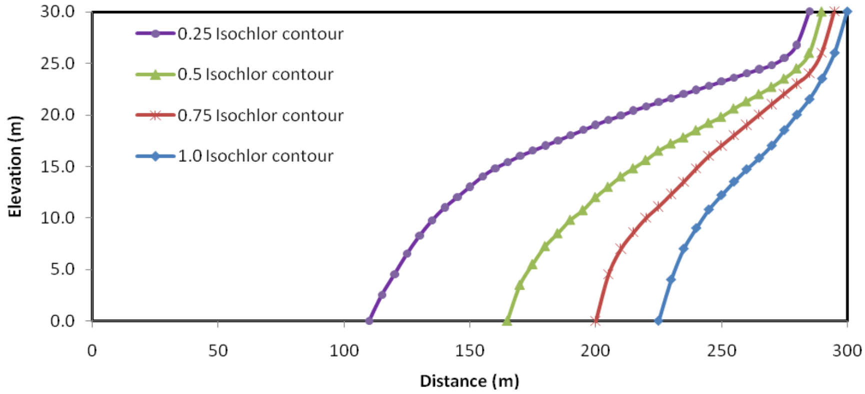


Figure 2 Isochlor counters in Biscayne aquifer, Florida

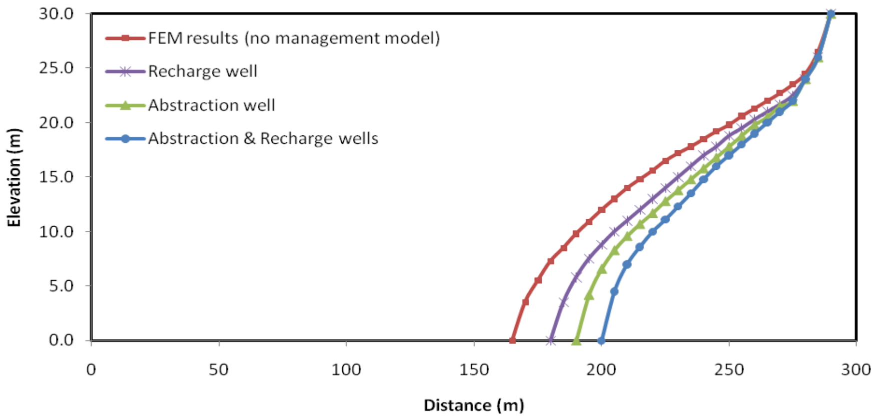
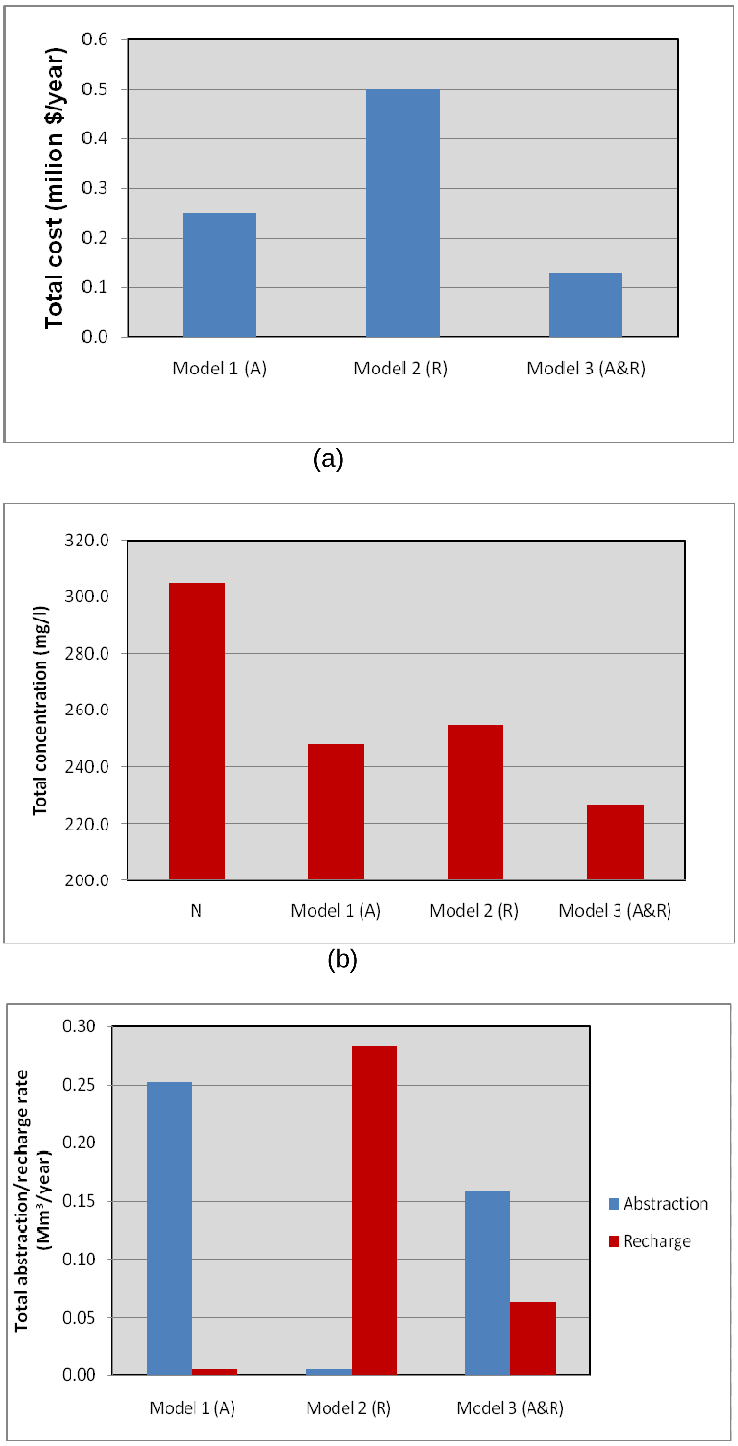


Figure 3 0.5 isochlor distribution from S/O models for the hypothetical case



(c)

Figure 4 Comparison between the results of model 1, 2 and 3 for: (a) total costs, (b) total salt concentrations in the aquifer (c) and total abstraction/recharge rates

Table 1 Summary of the results obtained from the S/O models for the case study

Model	Norm. L	Norm. D	Q (m^3/ s)	Norm. C	Q (Mm^3/ year)	F (cost $/year)
No management model	No abstraction or recharge wells have been used			305		
Abstraction well A	66	24	-0.008	248	0.252	0.25E+6
Recharge well R	156	24	0.009	255	0.283	0.5E+6
Combination A & R						
A	72	24	-0.005	227	0.158	0.13E+6
R	186	18	0.002	227	0.063	

From the above comparison charts it can be seen that using combination of abstraction and recharge wells in model 3 reduced salt concentration from 305 to 227 (mg/l). This amount of reduction in concentration is significantly greater than the amounts reduced by models 1 and 2. The combination of abstraction and recharge (model 3) requires abstraction of 0.158 (Mm3/year) of saline water and recharge of 0.063 (Mm3/year) of fresh water. The total amount of water abstracted and recharged in this method is less than the amount of water abstracted using model 1 (0.252 Mm/year) or recharged using model 2 (0.283 Mm/year). Model 3 also gives the lowest cost ($0.13 million per year) compared with the cost of model 2 ($0.5 million per year) and model 1 ($0.25 million per year). The cost of model 3 (combination of abstraction and recharge) is about 50% of the costs of the abstraction scenario and 25% of the recharge scenario.

Conclusions

Coastal aquifers present very complex and unique management challenges, especially when saltwater intrusion is involved. A simulation-optimization model was developed to study management and control of seawater intrusion in coastal aquifers using three management models; abstraction of brackish water, recharge of fresh water and combination of abstraction and recharge. The developed simulation-optimization model was applied to evaluate the three management scenarios and determine the optimal location, depth and rates of abstraction and/or recharge wells while minimizing the construction and operation costs. The efficiencies of the proposed management scenarios have been examined and compared. The results show that all three scenarios could be effective in controlling seawater intrusion but using model 3 (combination of abstraction and recharge), resulted in the lowest cost and salt concentration in aquifers and maximum movement of freshwater/saline water interface towards the sea. The results also show that for the Biscayne aquifer, the amount of abstracted and treated water is three times the amount of water required for recharge; therefore the remaining treated water can be used directly for different proposes. The developed model is an effective tool to control seawater intrusion in coastal aquifers and can be applied in areas where there is a risk of seawater intrusion. The application of the third model appears to be more efficient and more practical. It can also be used for sustainable development of water resources in coastal areas.

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Transmissivity Sequential Use in Estimating the Efficiency Wells

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ABSTRACT: This paper proposes a simple method for estimating of the wells efficiency, using sequential transmissivity. It is known that, during the pumping test of wells, with constant rate, the drawdown into wells is greater than the drawdown from aquifer. This difference between drawdown is the fundament of two theories: 1- efficiency theory and 2- wellbore storage theory. The author assumes that this difference is generated in the first seconds of pumping, and the size of the difference remains the same until the end of the test. The breaking of pace of the drawdown, that occur inside of the well, is due to a hydraulic shock generated by the screens hydraulic resistance. After exhaust of the hydraulic shock, the drawdown increase inside the well is equivalent to outside drawdown increasing. Thus, a portion of the drawdown from inside the well it is equivalent with the total aquifer drawdown. Now by applying sequential transmissivity relationship, we can determine two distinct values: 1) the apparent transmissivity, which includes the hydraulic jump from inside the well and 2) the real transmissivity, which is determined based on aquifer drawdown. The hydraulic transmissivity is a function of the specific yield, and efficiency can be defined as the ratio of apparent transmissivity and real transmissivity. Maximum error is 1%. The method has an important advantage, because it can be used the dataset from a single stage of pumping.

Keywords: breaking of pace, difference of drawdown, sequential transmissivity, well efficiency

Introduction

The first scientist who founded the efficiency wells theory was Jacob (1947). Starting from Jacob's concept, the present paper develope a new model by estimating the efficiency using sequential transmissivity. This solution can be applied to the pumping test of wells, with constant rate, for the non-steady flow regime. The model suppose the portion identification from the total drawdown, that is equivalent to the drawdown in the aquifer. In the first stage the transmissivity values are calculated for each increase of drawdown. After that it can be established the moment when they begin standardize, and can be determinated both real and apparent transmissivity. In this case, efficiency is defined as the ratio of apparent transmissivity and real transmissivity. The model was applied to a well which was constructed in Devesel area from Romania.

Methods

The main idea of the model presented in this paper is focused on developing drawdown during pumping a well with constant flow. Thus, in first seconds of pumping, the increases of Δs drawdown inside the well, are higher than the increases of Δs' drawdown from aquifer. This concept is known from the theory of the effect of capacity well. The author assumes that this difference it is created in the first few seconds of pumping, due to hydraulic resistance has the screens. After exhausting the hydraulic shock, the difference remains constant until the end of the pumping test. Identifying the equivalent drawdown, can be made after data processing using the relationship Păsărică (2001). The solution is based on Darcy law taken in the form of differential,

$$ds = \frac{Q}{2\pi T}\frac{dR}{R} \tag{1}$$

Where: s is drawdown [L]; Q is discharge rate [L^3T^{-1}]; T is transmissivity [L^2T^{-1}]; R is radius of influence [L]. If we assume that Q/2πT size is constant, for small intervals, then the equation (1) can be integrated on the left from s_{n-1} to s_n on the right from R_{n-1} to R_n, and leads to,

$$\Delta s_{n-1,n} = \frac{Q}{2\pi T_{n-1,n}} LN \frac{R_n}{R_{n-1}} \tag{2}$$

Where: n = 1, 2, 3, n-1; $T_{n-1,n}$ is sequential transmissivity [L^2T^{-1}]; Q is discharge rate [L^3T^{-1}]; $\Delta s_{n-1,n}$ is sequential drawdown [L]; R_{n-1} is radius of influence at the time n-1 [L]; R_n is radius of influence at the time n

[L]. Equation (2) is similar to the relationship Thiem. However, the time is the variable independent and the ratio R_n/R_{n-1} of equation (2) is substituted by the term $\sqrt{t_n / t_{n-1}}$ and leads to,

$$T_{n-1,n} = \frac{Q}{4\pi\Delta s_{n-1,n}} LN \frac{t_n}{t_{n-1}} \tag{3}$$

Equation (3) is comparable with Jacob solution. If we assume that the Δs increases in the well pumped, are equivalent with the Δs' increases in imaginary points located at the distances R_{n-1}, then equation (3) can be written in general form,

$$T_{n-j} = \left(\frac{Q}{4\pi\Delta s_{n-j}}\right) LN \frac{t_n}{t_{n-j}} \tag{4}$$

Boundary condition: $n \geq j$ (n = 0,1,2,3,,, n-1, ; j = 0,1,2,3,, , j-1) and $\Delta s_{n-j} > 0$; the solution (4) is applied for determine to the sequential transmissivity, on each increase has drawdown. Hydraulic transmissivity is a function of the specific yield, and efficiency it can be written as following,

$$E = \left(\frac{\Psi}{T}\right) * 100 \tag{5}$$

Where: E is efficiency [%]; Ψ is apparent transmissivity [L^2 T^{-1}]; T is real transmissivity [L^2 T^{-1}]. In the first stage is applied to equation (4) written as bellow,

$$T = \left(\frac{Q}{4\pi(s_t - s_x)}\right) LN \left(\frac{t_f}{t_x}\right) \tag{6}$$

Where: s_t is total drawdown, s_x is sequential drawdown (x = 0, 1, 2,, ……….x-1), t_f is the time at the end pumping test, t_x is sequential time (x = 0, 1, 2,, ………x-1), Q is discharge rate. Using equation (6) are determined average values. The first value includes the hydraulic jump, and it is apparent transmissivity. At some point the transmissivity values begin to be approximately equal, and they represent the aquifer transmissivity. This is when the hydraulic shock was exhausted. From this time, the Δs increase from inside the well is equivalent with the Δs' increase from aquifer. Real transmissivity is calculated with the equation (4) written as,

$$T = \left(\frac{Q}{4\pi\Delta s_{y,f}}\right) LN \frac{t_f}{t_y} \tag{7}$$

Where: Q is discharge rate; ty is the time at which the values they began to be equal; t_f is the time at the end pumping test; Δs_{y-f} is equivalent drawdown. The hydraulic jump from inside the well, it can be calculate with the following relationship,

$$\lambda = S_t * \left(1 - \frac{\Psi}{T}\right) \tag{8}$$

Where: λ is hydraulic jump [L]; s_t is total drawdown [L]; Ψ is apparent transmissivity [L^2T^{-1}]; T is real transmissivity [L^2T^{-1}].

Results and Discussions

The method was applied to estimate the efficiency of the well 610 W1, built in the Devesel area of Romania. The well has opened an unconfined aquifer, which was continuously pumped 19 hours, with a constant flow of 456.2 m^3/day. Near well, to distance of 15 meters was constructed an observation of well (see Figure1).

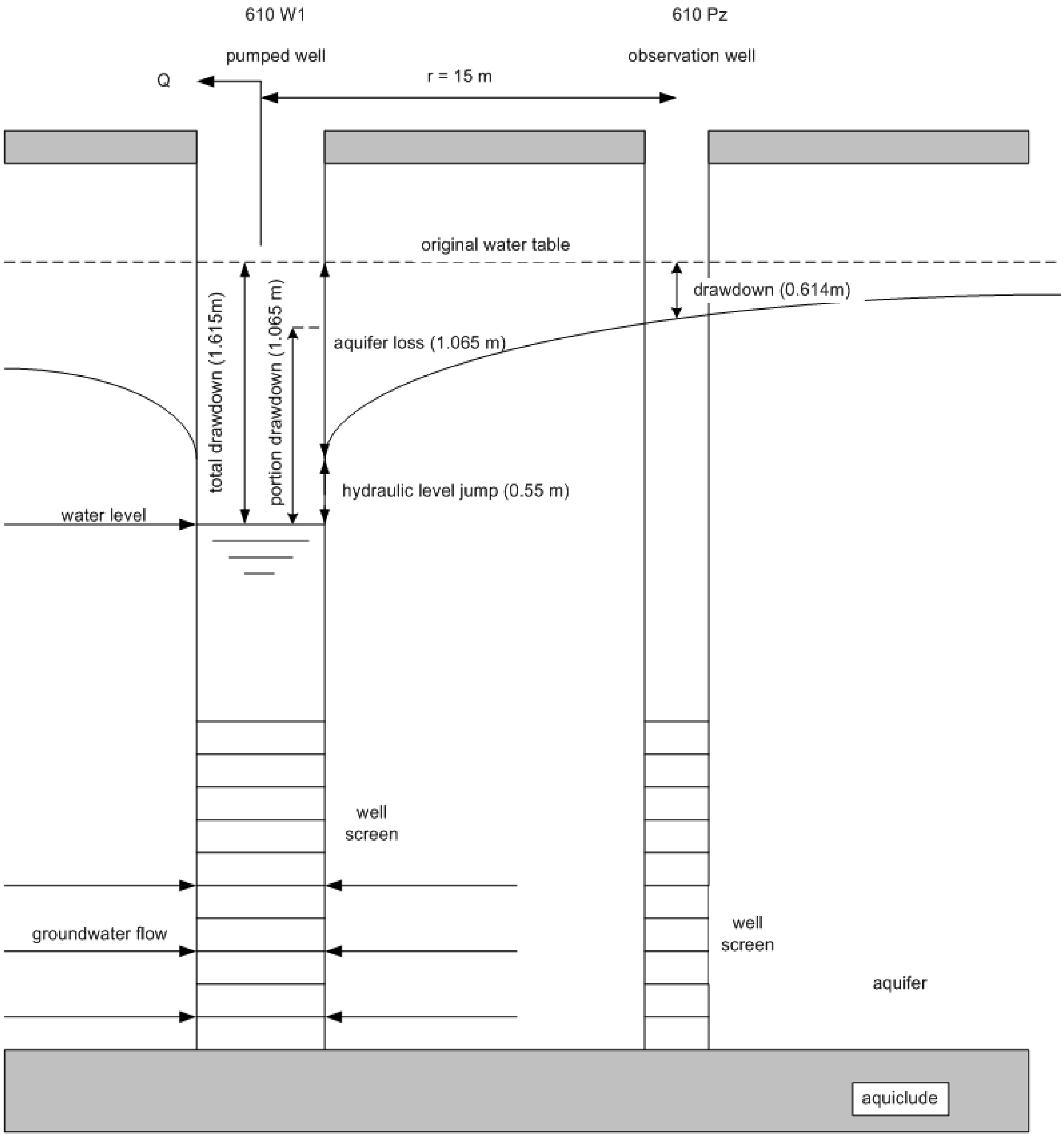

Figure 1. Cross – section at the end-pumping test “Devesel”. From Romania. Not a scale.

Primary data were measured simultaneously both in the pumped well and the observation well (see Table 1).
Data pumping test “Devesel”. From Romania.

t (sec)	s (m)		t (sec)	s (m)		t (sec)	s (m)	
	610-W1 Pumped well	610-Pz Obsv. well		610-W1 Pumped well	610-Pz Obsv. well		610-W1 Pumped well	610-Pz Obsv. well

0,001	0	0	1,200	1.375	0.433	12,600	1.520	0.510
30	1.140	0.220	1,320	1.380	0.434	14,400	1.535	0.520
60	1.200	0.260	1,440	1.385	0.435	16,200	1.540	0.530
90	1.240	0.300	1,560	1.390	0.438	18,000	1.545	0.535
120	1.250	0.320	1,680	1.393	0.440	19,800	1.550	0.540
150	1.260	0.330	1,800	1.395	0.445	21,600	1.555	0.545
180	1.280	0.350	2,100	1.405	0.450	25,200	1.565	0.550
210	1.290	0.360	2,400	1.410	0.455	28,800	1.570	0.560
240	1.295	0.365	2,700	1.413	0.460	32,400	1.575	0.570
270	1.305	0.380	3,000	1.415	0.463	36,000	1.580	0.580
300	1.310	0.390	3,300	1.417	0.465	39,600	1.585	0.585
360	1.315	0.400	3,600	1.419	0.467	43,200	1.590	0.590
420	1.325	0.410	4,200	1.445	0.470	46,800	1.595	0.595
480	1.330	0.415	4,800	1.455	0.475	50,400	1.600	0.600
540	1.340	0.420	5,400	1.465	0.480	54,000	1.605	0.605
600	1.345	0.425	6,000	1.475	0.485	57,600	1.610	0.608
720	1.350	0.428	6,600	1.480	0.490	61,200	1.615	0.610
840	1.360	0.429	7,200	1.490	0.495	64,800	1.615	0.614
960	1.365	0.431	8,400	1.500	0.500			
1,080	1.370	0.432	9,600	1.510	0.501			

Analysis of data, show as in the first 30 seconds of pumping, drawdown in the well pumped, has increased rapidly to 1.14 meters, and in the observation well, has increased by only 0.22 meters. It is remarkable that after the first 30 seconds the Δs increase of drawdown in the well pumped it is equivalent to increase Δs' of drawdown in observation well. This development has drawdown, suggests that the hydraulic jump is formed in the first few seconds of pumping, and the size remained constant until the end test. Dataset which was recorded in the well pumped, was processed with the equation (4), and has resulting values in Table 2.

Table 2. The average values of the sequential transmissivity. "Devesel". From Romania.

t <sec>	s <m>	sequential transmissivity <m^2/day>	t <sec>	s <m>	sequential transmissivity <m^2/day>	t <sec>	s <m>	sequential transmissivity <m^2/day>
1	2	3	1	2	3	1	2	3
0,001	negligible	**406**	1080	1,365	610	8400	1,5	648
30	1,14	590	1200	1,37	607	9600	1,51	664
60	1,2	614	1320	1,375	605	12600	1,52	629
90	1,24	640	1440	1,38	604	14400	1,535	686
120	1,25	629	1560	1,385	604	16200	1,54	675
150	1,26	624	1680	1,39	600	18000	1,545	668
180	1,28	641	1800	1,393	594	19800	1,55	666
210	1,29	644	2100	1,395	596	21600	1,555	668
240	1,295	639	2400	1,405	587	25200	1,565	689
270	1,305	645	2700	1,41	574	28800	1,57	658
300	1,31	643	3000	1,413	561	32400	1,575	632
360	1,315	632	3300	1,415	549	36000	1,58	613
420	1,325	634	3600	1,417	538	39600	1,585	599
480	1,33	628	4200	1,419	587	43200	1,59	592
540	1,34	635	4800	1,445	594	46800	1,595	594
600	1,345	633	5400	1,455	605	50400	1,6	611
720	1,35	620	6000	1,465	620	54000	1,605	665
840	1,36	622	6600	1,475	618	57600	1,61	860
960	1,365	615	7200	1,48	642	61200	1,615	-

We see that the values calculated after the first 30 seconds of pumping are very close . These values correlate with the value apparent has transmissivity (406 m2/day), confirming the hypothesis that the hydraulic jump was formed in the first 30 seconds of pomping, after which its size remained constant until the end test. In this case, were set the drawdown equivalent limits, t_2 = 60 sec; s_2 = 1, 2 m; t_{58} = 64800 sec, s_{58} = 1,615 m; and

by applying equation (4) result,

$$T = \left(\frac{0.08 * Q}{s_{58} - s_2}\right) LN \frac{t_{58}}{t_2} = \left(\frac{0.08 * 456.2 m^3 / day}{1.615 m - 1.2 m}\right) LN \frac{64,800}{30} = 614 m^2 / day$$

As a result, the well efficiency (610 W1) is, $E = \left(\frac{\Psi}{T}\right) * 100 = \left(\frac{406 m^2 / day}{614 m^2 / day}\right) * 100 = 66\%$. Size level jump $\lambda = S_t * \left(1 - \frac{\Psi}{T}\right) = 1.615 m * \left(1 - \frac{406 m^2 / day}{614 m^2 / day}\right) = 0.55 m$. Now we can check if true transmissivity was correctly determined on the basis of the equivalent drawdown. Its size it must be close to the transmissivity size, which is determined on based data measured in the observation well (610 Pz). Thus, the data set in Table 1 were established the following limits, t_1 = 1 sec s_1 = 0.0m, t_{58} = 64 800 sec, s_{58} = 0.614 m. Following the data processing results,

$$T = \left(\frac{0.08 * Q}{s_{58} - s_1}\right) LN \left(\frac{t_{58}}{t_1}\right) = \left(\frac{0.08 * 456.2 m^3 / day}{0.614 m}\right) LN \frac{64800}{1} = 657 m^2 / day$$

. The value by 657 m2/day is very close by 614 m^2/day which is that of the actual transmissivity. Thus, is confirming the accuracy of its determination, based on the equivalent drawdown.

Conclusions

The results obtained are encouraging and show a model that can be applied to the single stage pumping. The practical example presented in this paper it was carefully chose from more experiments to verify the reliability model. Also, and in this case, the pumped flow can influence the efficiency because the hydraulic jump size varies depending on the size of the pumped flow. But we must emphasize the fact that real transmissivity does not depend on the size of the pumped flow. The method involves low production costs because can be used dataset from a single stage pumping. Also, the solution is purely analytical, and the pumping data can be processed in real time with a very simple procedure. The well efficiency W1 610 is 66%, due to actual construction and the true transmissivity. It is remarkable as the real transmissivity (614 m^2/day), is very closer by the determinated value (657 m^2/day) from dataset recorded into the observation of well. Consequently, the method can be applied when the wells are pumped with constant flow in no-steady flow regime, with a single stage pumping. Thus, in real time can be determinated both the efficiency and true transmissivity.

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GROUNDWATER MANAGEMENT IN BRIBIE ISLAND FOR URBAN EXPANSION

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Bribie Island is a sand island located close to Brisbane, Australia. It is located at the northern end of Moreton Bay. Historically much of Bribie Island's town water supply has been sourced from groundwater. Groundwater has historically been drawn from a trench system in dune sands at the island's southern end. The migration of the seawater interface into groundwater is inhibited through a release of treated sewage effluent to sand dune areas south of the trench. This is undertaken to preserve a groundwater mound above mean sea level between the coastline and the extraction trench. An increase in water demand from urban expansion on Bribie Island has led to establishment of a larger capacity well-field in the central area of the island.

Although advanced software is available that can simulate seawater intrusion processes, there is frequently insufficient relevant data available to define hydrogeological conditions in coastal aquifers. This raises questions regarding the predictive uncertainty of complex seawater intrusion models in the face of limited data. The application of sophisticated seawater intrusion models does not necessarily provide sufficient confidence to support environmental and water resource management decision making for coastal aquifers. The environmental implications of the expansion of urban areas of Bribie Island has been studied using a simple suite of software comprising MODFLOW, MT3DMS, MODPATH and Glover's formula to describe seawater interface movement with discharge of freshwater.

A process of numerical groundwater flow modelling combined with analytical modelling of seawater intrusion was used to investigate the impacts on the island's groundwater resources for a series of development scenarios. Since the response of natural systems to hydrological stress is not completely predictable, a robust process of adaptive management is undertaken for groundwater extraction on Bribie Island. This adaptive management system is supported by systematic hydrological and ecological monitoring.

Keywords: Sand island, artificial recharge, seawater intrusion, analytical modelling, numerical modelling, Ghyben-Herzberg, Glover's solution

INTRODUCTION:

Bribie Island is a Quaternary age coastal sand mass island covering approximately 144 km^2. It is surrounded to the east by the Coral Sea and to the south by Moreton Bay (See Figure 1). To the west it is separated from the mainland by Pummicestone Channel. It is generally low lying and has a maximum elevation of only 14 m. The elevated areas of the island coincide with north by north west trending dunes in the central-eastern section of the island. This area of elevated land is separated from low dunes (with an elevation in the order of 4 m AHD) in the central – north section of the island. Juxtaposed with the ridge systems of the island are low-lying swales that host wetlands (Evans et al. 2002). This series of wetlands, as well as other environmentally valuable wetlands on the island have complex groundwater dependency. This island is fringed in part by Ramsar listed wetlands.

Bribie Island experiences a subtropical climate with highly variable rainfall. Mean annual rainfall on the island is approximately 1,340 mm. The lowest recorded rainfall for the area was for 1993 when only 721 mm was recorded. There is a marked wet and dry season on the island. On average approximately 80% of the rainfall falls from October through May. The wettest months are December through February when on average 35% of annual rainfall occurs. Mean monthly pan evaporation exceeds mean monthly rainfall over the period June through February.

Figure 1 – Study area locality

Figure 2 – Location of infrastructure

Bribie Island consists of a blanket of sand deposits of Holocene to Pleistocene age overlying the Mesozoic age Landsborough Sandstone (Evans et al. 2002). The sand deposits range in thickness from 5 to 41 m and are extremely variable. Hard dark coloured humicrete / cemented sand layers locally known as 'coffee rock' are broadly distributed throughout the island. These horizons consist of humic materials and sesquioxides. The origin of the cemented sands is not yet fully understood. It is however suspected that groundwater processes are involved in their formation (Harbison 1996). Figure 2 indicates the location of a schematic hydrogeological cross section provided as Figure 3. Figure 3 illustrates the upper dune sands and cemented sand overlying older Pleistocene age alluvial sediments. It is noted that at the southern end of the island, where artificial recharge is practiced, the cemented sands are largely absent.

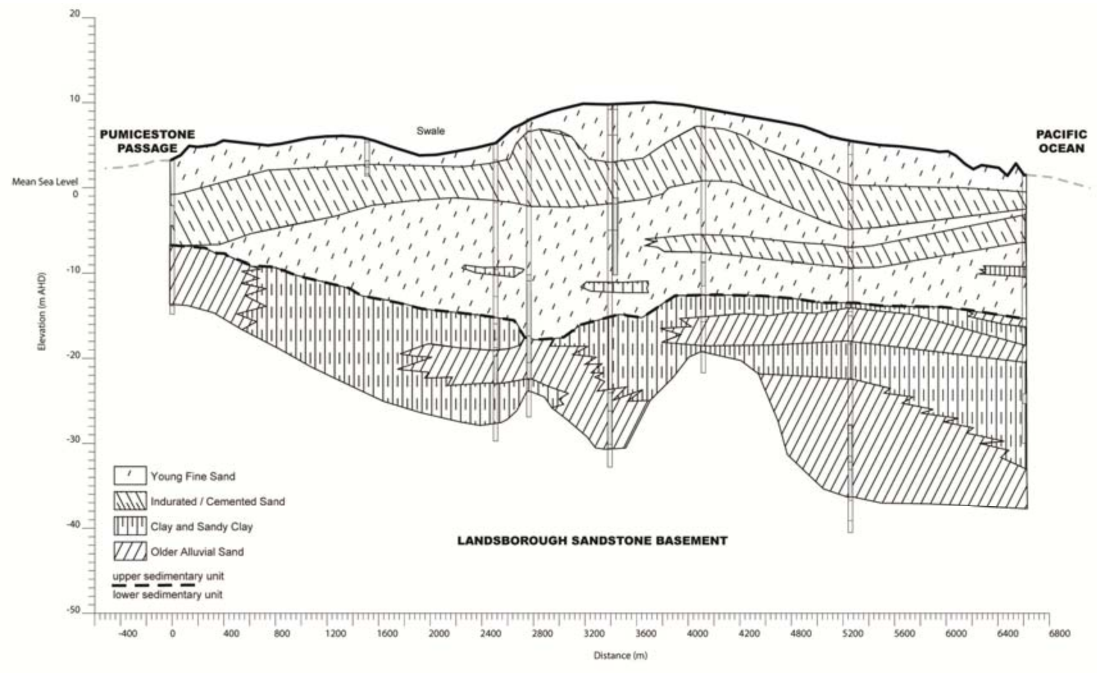

Figure 3 – Schematic geological cross-section

Groundwater occurs within the intergranular spaces of the sand deposits of the island. The aquifer is generally unconfined. However local semi-confining to confining conditions can develop in association with shallow humicrete layers. Recharge to the aquifer occurs via direct infiltration of rainfall into sands and leakage through cemented sands. Discharge from the aquifer occurs via seepage discharge to the ocean and via ephemeral discharge to low-lying areas. In the low-lying areas losses to surface water flow and evapotranspiration occur. Groundwater forms a typical elongated mound with higher groundwater elevations associated with areas of elevated topography. Figure 2 provides groundwater elevation contours that illustrate this.

A major feature of the island is the presence of extensive shallow groundwater perched over the humicrete / cemented sand layers. This perched groundwater is critical to local wetland ecology. Groundwater levels in the perched systems may be up to 4 m higher than groundwater levels in the underlying main body of the sand.

A groundwater extraction system for town water supply has been successfully operated at the southern end of Bribie Island since 1962. Initially the groundwater extraction system consisted of six production bores and a 2.2 ML/d water treatment plant located at Woorim (Harbison, 1996). These bores were supplemented with the installation of a series of 21 additional bores during the period 1966-67. These bores were located in an area that was in 1970, gazetted as a water reserve in the south east corner of the island (Harbison, 1996).

Because of iron fouling of the production bores the groundwater extraction system was converted to an open trench system (see Figure 2). This trench system was approximately 3 km long and 5 m deep extending westward from the previous extraction areas (Harbison, 1996). The water reserve currently occupies approximately 259 ha (Werner, 1998). A subsequent separate, smaller extraction trench known locally as the “black hole” was constructed to the north of the main trench.

Annual groundwater extraction from the main trench system has historically been in the order of 1 to 3 ML, and sometimes reduced to zero. However during spring, when groundwater levels fall significantly prior to summer rains, groundwater extraction from the trench falls significantly to in the order of 1 to 2 ML/d. When Bribie Island cannot produce enough water from local groundwater sources, treated water is imported onto the island via pipelines from treatment plants up to 40 kilometres away.

It was quickly appreciated that there could be a significant threat of seawater intrusion to the trench if groundwater extraction rates were too high (Lumsden, 1964). In order to address the potential threat of seawater intrusion an artificial recharge system was established. This system was located between the southern coastline of the island and the groundwater extraction trench. The artificial recharge system involves the land application of treated municipal sewage effluent to sand dune areas. A series of basins are used for this purpose (see Figure 2) and currently land application to this system is in the order of 5 ML/d. Over time, the standard of treatment of the water applied to the dune system has been improved and the water currently applied is predominantly sewage effluent treated to a tertiary treatment standard and disinfected with chlorine and UV radiation.

The groundwater extraction system supported by the artificial recharge operation functioned well for over 20 years. However there were still some problems encountered with it. These problems included difficulties in the treatment of groundwater which is naturally high in colour due to organic compounds. There were also difficulties in pumping sufficient water from the system during periods of extended drought.

Additional concerns have been held for the Bribie Island water supply that includes the significant ongoing costs of transferring treated water to the island from the mainland. It has been recognised that there is a need to safeguard continuity of water supply at the end of a very long

(> 40 km) trunk main system. Further to these issues there a projected increased local demand for water from urban extraction on the island placed strain on the water supply system.

In response to these matters the establishment of a new borefield was proposed (Evans et al. 2002) for the central section of the island. The evaluation and planning for the proposed borefield and the operation of the water extraction trench system required prediction of the potential impacts of groundwater extraction and artificial recharge on:

- Seawater intrusion;
- Groundwater levels in wetland areas;
- Depth to groundwater in artificial recharge areas;
- Rates of groundwater movement around artificial recharge areas; and
- Groundwater quality around artificial recharge areas.

METHODS:

A pragmatic approach was developed to utilise an existing numerical (MODFLOW) model (McDonald & Harbaugh, 1988) of the island (Department of Natural Resources, 1996) together with analytical solutions to groundwater flow to achieve the required prediction of groundwater level and seawater intrusion impact.

The MODPATH utility (Pollock,1994) was adopted to examine the travel time of groundwater from the artificial recharge areas. The MT3DMS solute transport model (Zheng & Wang, 1999) was coupled to the MODFLOW model to examine the migration of certain contaminants from the artificial recharge areas.

Seawater intrusion assessment

Because the island is surrounded by seawater, reliable prediction of changes to the position of the seawater interface is crucial to the evaluation of alternative water management strategies on the island.

Within the sand aquifer there is an interface between the seawater and the fresh groundwater. The position of this interface is approximated by the Ghyben-Herzberg relationship. The interface between seawater and freshwater will depend on a range of factors (including local geology). However in aquifers of uniform hydraulic conductivity the shape of the interface is that of a wedge. The toe of the seawater wedge is generally (but not exclusively) located landward of the coast line.

Effectively the location of the seawater wedge depends on:

- The pattern of the heads of fresh water in the aquifer above mean sea level;
- The volume of outflow of freshwater from the aquifer at the coastline; and
- The depth to aquifer basement.

Lower groundwater heads and reduced volumes of freshwater outflow predispose the aquifer to landward incursion of freshwater. Greater depths to aquifer basement potentially allow greater landward incursion of the toe of the seawater wedge.

Reduction in groundwater levels to below mean sea level between the coastline and a groundwater extraction point are typically considered to be the main precondition to induce seawater intrusion. However it is important to note that reductions in the rate of outflow of groundwater from the aquifer to the sea around the coastline will induce a landward shift in the location of the interface. The magnitude of such shifts will be relative to the reduction in outflow.

In order to achieve the required prediction of changes to the position of the seawater interface the following approach was taken:

- Review available hydrogeological, ecological and land use data together with the location of existing and proposed groundwater extraction infrastructure.

- Identify a series of key cross sections for which predictions of the landward movement of the seawater wedge is required.

- Run the one-layer MODFLOW model of the main dune sand aquifer with no groundwater extraction. Confirm the rate of subsurface outflow of groundwater to the coast along the sections of the coast relevant to the selected cross sections.

- Run the one-layer MODFLOW model of the main dune sand aquifer with groundwater extraction as required for each alternative development scenario. Confirm the rate of subsurface outflow of groundwater to the coast along the sections of the coast relevant to the selected cross sections.

- For each cross section use Glover's analytical solution (Glover 1959, and Kashef, 1983) to the Ghyben-Herzberg relationship to predict the change in position of the seawater interface.

Figure 4 provides a diagrammatic section the location of which is indicated in Figure 2. Figure 4 illustrates changes to the position of the seawater interface from a pre-development state to a developed state with pumping from the central island bore-field at 4.32 ML/d.

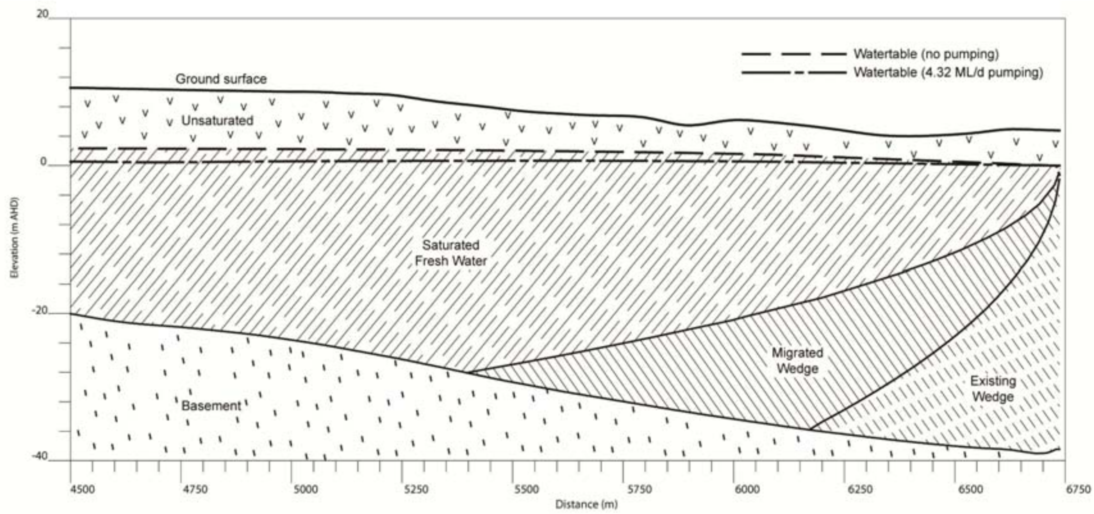

Figure 4 – Schematic model simulated seawater intrusion

Once these calculations were made a process of review was made with existing values of groundwater salinity on the coastal fringe being compared to the predicted position of the seawater wedge.

The advantage if this prediction approach is that it avoids the need to construct complex numerical seawater intrusion models that may entail significant uncertainty.

Assessment of artificial recharge areas

Assessment of the sustainability of the artificial recharge areas involved assessment of:

- Potential waterlogging of the landscape;
- Groundwater travel time from land application areas; and
- Contaminant concentration assessment.

Potential waterlogging of the landscape was assessed using the MODFLOW model to generate groundwater elevation contours for a series of development scenarios. These development scenarios involved differing rates of land application of treated effluent (5 ML/d to 8 ML/d) and groundwater extraction from the water supply trench. The groundwater elevation contours were then compared with a high resolution DEM produced from detailed topographic contours. In this way areas where the groundwater surface could reach the ground surface could be identified. The outcropping of recharge impacted groundwater close to the land application areas is considered to be undesirable. This is because travel time in the groundwater system provides scope for additional pathogen die-off in the already treated water.

The MODPATH utility was used to assess the groundwater velocity in the vicinity of the land application areas. One of the key input parameters required was effective porosity. Effective porosity of 8% was adopted to account for the presence of minor amounts of silt, clay and organic matter in the fine sands.

Cromer et al. (2004) provided a nomograph to assess pathogen die off from travel time and water temperature. This approach to the assessment of pathogen attenuation was applied to the groundwater velocities derived from the MODPATH assessment for a groundwater temperature range of 21ºC and 24ºC.

Bribie Island area has no industrial waste disposal to its sewage system. The key potential groundwater contamination issues for the operation of the effluent management areas are the potential for viable pathogens to migrate into the water supply trench and the concentration of nitrate that develops in the aquifer. For management of treated effluent at Bribie Island by land application, compliance with drinking water standards is not a major issue. This is because the effluent has mean total nitrogen concentrations in the order of only 2.7 mg/L as N. However slightly elevated nitrogen levels in the groundwater exposed in the water extraction trench could potentially exacerbate macrophyte and algal growth in the trench.

To support a preliminary assessment of impact from nutrients, the MT3DMS model (Zheng and Wang, 1999) was applied adopting a uniform longitudinal dispersivity value of 50 m. As there was a relative paucity of local data pertaining to lateral nutrient attenuation processes such as denitrification and phosphorus sorption a conservative approach was taken, assuming no lateral attenuation. Constant input concentrations of 2.7 mg/L total nitrogen and 0.36 mg/L total phosphorus were adopted.

RESULTS AND DISCUSSION:

It was found that in general the available groundwater monitoring data around the coastline was consistent with the position of the seawater wedge as predicted by modelling. This modelling involved analytical assessment of the wedge position in response to changes to coastal subsurface groundwater outflow predicted by the numerical model.

An assessment was made that unacceptable landward migration of the seawater interface would not occur for the following development scenario:

- Implementation of a new borefield in the central section of the island operating at a total 4.32 ML/d from a series of 20 low capacity production bores;

- Continued operation of extraction of 1.08 ML/d from an open extraction trench at the southern end of the island; and

- Continued operation of a 5 ML/d artificial recharge scheme using land applied treated effluent at the southern end of the island.

Appreciable waterlogging from shallow groundwater was predicted for the artificial recharge scheme using the steady-state MODFLOW model for applications exceeding 5 ML/d in concert with groundwater extraction from the main water extraction trench at 1.08 ML/d.
The MODPATH simulation predicted rates of groundwater movement in the order of 0.5 to 0.8 m/d from the land application areas for treated effluent. Applying the nomographs of Cromer et al. (2004) for a lateral groundwater travel time of 365 days from the effluent irrigation area, an eight order of magnitude decline in viral load could be expected. Such a magnitude in decline in viral load, achieves a three order of magnitude reduction to achieve the WHO guidelines. These guidelines recommend for human health (drinking water) protection, no detectable viruses (i.e. one or less) in a cubic metre of water.

Not surprisingly the MT3DMS simulation suggested that groundwater with quality similar to land applied input quality would, in steady-state migrate relatively widely from the land application areas. This largely reflects the simplifying assumption of no lateral attenuation process other than dilution and dispersion in the aquifer.

Although the central island borefield is being operated to provide town water supply, currently extraction from the trench system has been suspended. This is because Seqwater, the bulk water supply agency has discontinued operation of the existing water treatment plant at Woorim in favour of operation of the new central island borefield and treatment plant. The future status of the Woorim treatment plan remains under consideration at this time.

A process of additional reassessment is currently being undertaken to explore options for the potential reintroduction of groundwater pumping from the trench at the southern end of the island.

CONCLUSIONS:

A process of numerical groundwater flow modelling combined with analytical modelling of seawater intrusion has been successfully applied to the investigation of groundwater development impacts on Bribie Island.

Although the predictive modelling approach taken is both robust and defensible, the management of the water resources of the island is being supported by an adaptive management framework. This framework include systematic observation of both groundwater levels and quality and ecological (terrestrial flora and fauna) monitoring.

Beneficial management of treated sewage effluent to create a mound of groundwater between the sea and a groundwater extraction system has proved successful over a very long period of time at Bribie Island. The future of this process will depend on a decision regarding future extraction and treatment of water from the major water extraction trench and assessment work for this remains ongoing.

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Modelling the regional impacts of multiple MAR schemes on the Northern Adelaide Plains

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Abstract Managed aquifer recharge (MAR) utilising both captured stormwater and treated reclaimed wastewater is being increasingly adopted across Australia to supplement traditional water supplies. MAR has been adopted to control abstraction and restore the groundwater balance, prevent seawater intrusion and to take pressure off traditional water supplies. Some local government authorities in South Australia are implementing MAR as part of their integrated water management plans to meet irrigation water demand. Once all of these schemes are operational it is expected that they will capture an estimated 17 gigalitres of stormwater annually for summer reuse. Most of these schemes are located on the Northern Adelaide Plains where market gardens are the principal land use and irrigation water is sourced from the deeper Tertiary aquifer systems.

As a consequence of over 60 years of prolonged use groundwater taking across the Northern Adelaide Plains Prescribed Wells Area is managed under a set of rules outlined in the water allocation plan for the area. As MAR represents a new use, juxtaposed with the traditional groundwater use, there is the potential for one activity to impact on the other if not effectively administered. For example, the new use results in greater summer drawdowns in the aquifer which may require existing users to deepen their wells. During winter, recharge pressures may create artesian conditions requiring existing users to enclose the well headworks to prevent flooding. Additionally, the existing management policies may, in the longer-term, exacerbate the potential problem associated with artesian impacts during the winter recharge period. A regional numerical groundwater model of the Northern Adelaide Plains aquifers was used to test these and other potential impacts the results of which are presented in this summary paper.

Keywords managed aquifer recharge; aquifer storage and recovery, groundwater numerical modelling; Northern Adelaide Plains

INTRODUCTION

The Cities of Salisbury, Playford and Tea Tree Gully received funding from the National Water Initiative (NWI) to construct and operate a series of managed aquifer recharge (MAR) schemes across the Northern Adelaide Plains (NAP) as part of the Waterproofing Northern Adelaide strategy. These schemes involve the capture and passive treatment (via wetlands) of urban stormwater runoff prior to recharge into the deep Tertiary (T1 and T2) aquifers. Once all of the schemes are in operation the total volume of stormwater that will be harvested and recharged on an annual basis will approach 17,000 ML (**Table 1**). The recharged stormwater is recovered during the dryer summer months and used for industrial purposes and irrigation of community assets (parks and recreation ovals) within the council areas. Options are also being explored to deliver some of the harvested stormwater to domestic dwellings via a third pipe.

Table 1 Proposed recharge volumes for the various MAR schemes by target aquifer.

MAR Scheme T1 Aquifer	Proposed Recharge (ML/yr)	MAR Scheme T2 Aquifer	Proposed Recharge (ML/yr)	MAR Scheme Fractured Rock	Proposed Recharge (ML/yr)
Andrews Farm	200	Andrews Farm	1100	Banksia	50
Edinburgh Parks North	100	Andrews Farm South	1200	Harper's Field	40
Greenfields	2500	Bennett Road	800	Kingfisher	40
Kaurna Park	200	Edinburgh Parks North	400	Satsuma	40
Munno Para West	200	Edinburgh Parks South	800	Smart Road	50
Paddocks	160	Kaurna Park	200	Solandra	20
		Lindblom	160	Summer Road	2400
		Montague Road	1500	Tiley Reserve	60
		Munno Para West	1200	Torrens 1	70
		Parafield	1600	Torrens 3	80
		Pooraka Triangle	160	Walkley Heights	160
		Sp Watera/Burton	400	Wynn Vale	90
		Whites Road	800		
TOTAL Volumes	**3360**		**10320**		**3020**

The study area (**Figure 1**) lies partly within the NAP Prescribed Wells Area (PWA) where rules setting out the conditions of groundwater management including, transfer, allocation and fees, are described in the Water Allocation Plan (WAP) for the area. Groundwater use, to support market gardening activities, has been occurring (primarily in the northern part of the area) since the late 1940's. MAR constitutes a relatively new use and may inadvertently impact on the existing users through increased aquifer drawdowns during summer and increased groundwater levels during the winter recharge period. Additionally, the management policies for the region, which currently require 20% of all recharged stormwater to remain in the aquifer as an environmental benefit may, in time, exacerbate the problem of artesian impacts during winter.

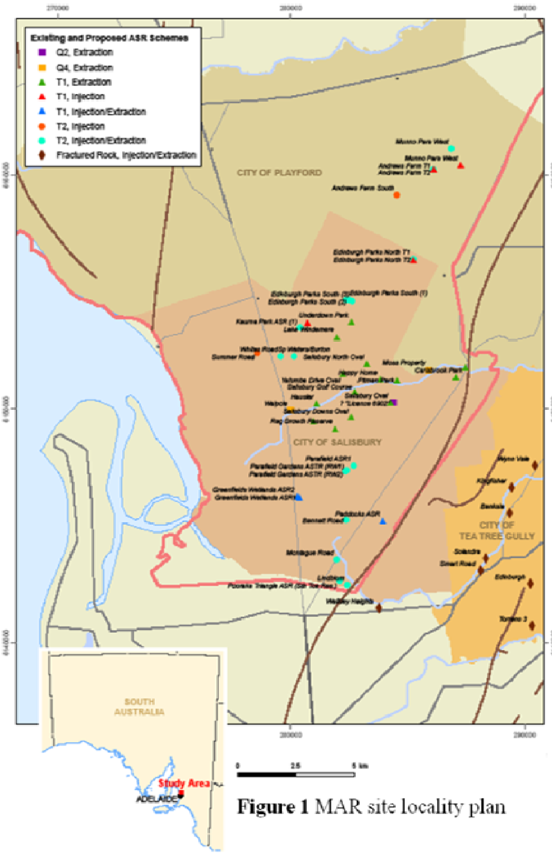

Figure 1 MAR site locality plan

HYDROGEOLOGY

The major geological units within the NAP contain multiple aquifers making the hydrogeology of the region extremely complex (**Figure 2**). Structural controls, such as faulting, further complicate the hydrogeological setting. These structural controls are thought to influence the movement of groundwater in the deeper regional Tertiary aquifer systems.

The upper-most Quaternary sediments can contain up to six unconfined, semi-confined and confined Quaternary aquifers (Q1 to Q6). These Quaternary aquifers are relatively thin and of varying lateral extent and therefore all six aquifers are not always intersected in every drill hole. Aquifers in the Quaternary sediments have typically been developed for abstraction by small-scale users for stock and domestic purposes in areas where yield and underground water quality is favourable (Hodgkin 2004). The upper Quaternary aquifers are not typically influenced by activities associated with the MAR schemes.

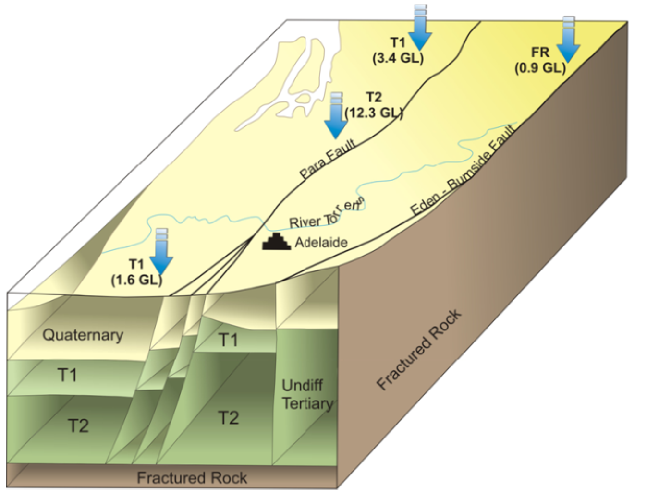

Figure 2 Conceptual model Adelaide sub-basin aquifer system (after Gerges 1999)

Tertiary sediments underlie the Quaternary sediments and contain up to four (T1, T2, T3 and T4) semi-confined and confined regional aquifer systems. The upper T1 and T2 Aquifers are the most widely accessed aquifers on the NAP.

The T1 aquifer is mainly utilised in the southern NAP where groundwater quality is typically less than 1,000 mg/L whilst the T2 aquifer is largely utilised in the northern part of the NAP PWA. Aquifers within the Cambrian and Precambrian basement, collectively referred to as the fractured rock aquifer, lie along the eastern margin and underlie the Tertiary age sediments.

The T1 aquifer is hydraulically connected to the overlying Q4 aquifer in the northern PWA; however, the T1 and Q4 aquifers are hydraulically separated by a leaky confining bed in the southern central part of the NAP PWA around Waterloo Corner. The T1 aquifer is generally intersected at depths of 60 m below ground surface and is typically between 30 and 60 m thick. In the northern part of the NAP PWA the T1 aquifer thins and pinches out. Groundwater salinity within the T1 aquifer is reported to range from 600 mg/L to greater than 5,000 mg/L (Martin, 2005), with the lowest reported groundwater salinities occurring in the southern portion of the NAP PWA.

The T2 aquifer is hydraulically separated from the overlying T1 aquifer by the laterally extensive Munno Para Clay member which is uniformly 10 m thick throughout the NAP. The T2 aquifer is comprised of a well-cemented sandy limestone of the lower Port Willunga Formation and is continuous throughout the NAP, varying between 40 and 120 m thick. Groundwater salinity is reported to range from 700 mg/L to 3,000 mg/L, with lower salinities typically reported in the northern portion of the NAP around Virginia. The T2 aquifer is the primary aquifer used in the northern part of the NAP PWA for both irrigation purposes and also for MAR.

NUMERICAL GROUNDWATER MODEL

A numerical groundwater flow model of the Adelaide Coastal Plains Tertiary aquifers and surrounding fractured rock recharge area of the Western Mount Lofty Ranges was developed using the MODFLOW software package (MacDonald and Harbaugh, 1989) version 4.0. The model incorporates essential features of the hydrogeological system (**Figure 2**) and has been used to predict the affects of varying groundwater abstraction regimes in that system (REM, 2005). Key features of the model include:

- The confined lower Quaternary (Q4), and the main Tertiary aquifers (T1 and T2) of the Adelaide Plains sub-basin and the Tertiary aquifers of the adjoining Golden Grove Embayment;
- Recharge mechanisms via through-flow from the fractured rock aquifers of the Western Mount Lofty Ranges (WMLR); and
- Discharge features comprising leakage under the Gulf of St Vincent and abstraction wells.

The model represents a good approximation of the NAP aquifer system where existing stresses occur. Gaps in knowledge in both the fractured rock and sedimentary aquifers have resulted in some model limitations that include:

- Limited information concerning historical groundwater levels and abstraction means that it is not possible to calibrate the model to observed hydraulic heads across the fractured rock aquifer portion of the model domain.
- Insufficient information on the spatial distribution of groundwater use from the fractured rock aquifer system does not allow pumping demand to be incorporated into the model at this time for the fractured rock portion of the model domain.
- Groundwater abstraction across certain parts of the Adelaide metropolitan area (south of the NAP PWA) is based on estimated use as there is no metered data available.
- As at 2005, when the model was first developed, there was limited information on use or groundwater levels in the T1 and T2 aquifers along the western and northern margins of the model domain resulting in some uncertainties for model calibration in these areas.
- The model is currently constrained in areas where there is limited information on aquifer properties, such as the eastern area of the NAP adjacent to the Adelaide Hills.
- The model encompasses a large area and was initially developed using a grid cell size of 500 by 500 metres to assess regional scale impacts associated with a range of stresses. Preliminary evaluation of the model for this study has indicated that the current grid size of 500 by 500 metres is not adequate for site-scale evaluation of operational issues, such as well interference, without significant modification to the model. Therefore, an analytical model was used to address issues relating to well interference and optimal well spacing for the various MAR schemes.
- The grid cell size of 500 by 500 m results in modelled heads during recharge being slightly under predicted compared to the observed heads. This occurs primarily because the modelled results are integrated over the area of each cell.
- Predictive simulations were run from 2005 through to 2015. The 10 year period is considered sufficient to reliably predict the overall impacts associated with the development of the MAR schemes. Beyond 10 to 15 years uncertainty in patterns of use, climate variability and various other external influences generally make predictions less reliable.
- MAR schemes within the City of Tea Tree Gully (CTTG) area that inject into the fractured rock aquifer (**Figure 1**) have not been included in the modelling predictions because of the limitations associated with the fractured rock region of the model domain.

MODEL PREDICTIONS

Model predictions were run to identify:

- Likely peak pressures generated during recharge to ensure safe operational limits were maintained thereby avoiding potential failure of the confining beds;
- Maximum drawdown during abstraction at the proposed pumping rates; and
- The extent of artesian effects during recharge.

Peak aquifer pressures

Increased or decreased aquifer pressures may have unintended consequences. Pressures (heads) generated during MAR recharge cycles can potentially lead to hydraulic fracturing of the aquifer medium. Therefore, injection pressures need to be monitored carefully to ensure that critical pressures are never exceeded. An assessment of the pressure induced changes was carried out to estimate the potential shear stress of the T1 and T2 aquifers using the approach adopted by Brown *et. al.*, 2005.

The approach adopted applies classical soil mechanics principles such as the "Mohr-Coulomb failure envelope" (Blyth, & de Freitas, 1984). Using this type of evaluation, the total vertical and horizontal effective stresses were computed and then combined to estimate the critical normal stress and shear stress that occurs on a failure plane. In the absence of measured data on the T1 and T2 aquifer matrix empirical values were adopted for rock strength. Brown *et. al.*, 2005, suggests a safety margin of approximately half the calculated failure pressure of the aquifer matrix should be adopted. This implies that the pressure head that could safely be applied to the T1 and T2 aquifers, where failure would be unlikely to occur is, 150 m.

Model predictions indicate that the maximum operating heads during recharge into the T1 aquifer are likely to be in the order of 45 m (441 kPa) or less except at the Andrews Farm/Munno Para site where heads up to 85 m (analytical model) may be observed (**Figure 3**). These pressures are within the estimated safe operating limit (150 m head) for the T1 aquifer. The potential for localised variation in either the aquifer matrix properties or confining bed cannot be entirely eliminated and therefore operating pressures should be constrained to reasonable values.

The model predicts maximum operating heads during recharge into the T2 aquifer of up to 60 m (588 kPa) for the proposed volumes of recharge (**Figure 4**). This compares favourably with observed levels from the operation of existing schemes.

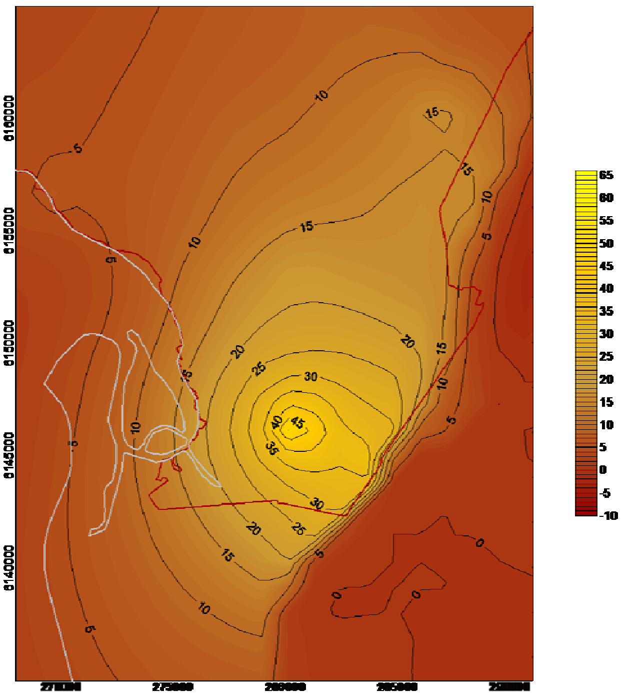

Figure 3 Peak aquifer pressures (m head) in the T1 aquifer during recharge.

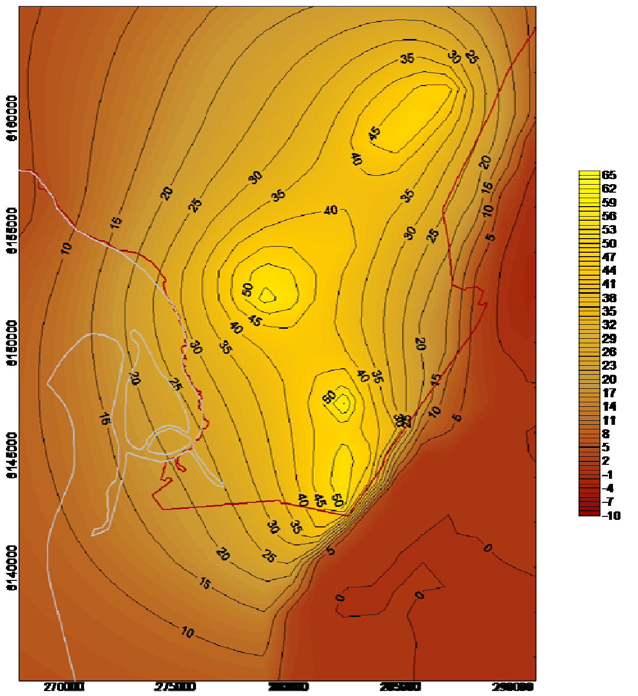

Figure 4 Peak aquifer pressures (m head) in the T2 aquifer during recharge.

The predicted levels are achieved using the minimum number of recharge wells at each site calculated from the expected recharge rates. The predicted operating pressures are well below the estimated safe operating limit (150 m head) and therefore pressure-induced hydraulic fracturing of the T1/T2 aquifer and Munno Para Clay confining bed, or total matrix failure, is unlikely if recharge heads are constrained to less than 70 m maximum operating head.

Maximum drawdown during abstraction

MAR wells located in close proximity can lead to significant hydraulic interference with large head increases or decreases. Understanding the potential maximum drawdown during abstraction is important to ensure that pumps are set at the correct depth thereby reducing the possibility of the pumps running dry near the end of the abstraction cycle when groundwater levels are likely to be at their lowest. Pumping interference between the MAR scheme and neighbouring wells may result in larger drawdowns causing existing users to reduce their pumping demand, set pumps lower in the well, or deepen their wells which results in increased costs.

Modelling of the recharge and subsequent abstraction has shown greater seasonal amplitude in groundwater levels in both the T1 and T2 aquifers occurs in response to the MAR activities (**Figure 5**). Additionally, because the management policies in the WAP that require 20% of the injected volume to remain in the aquifer there is an overall increase predicted in groundwater levels in the T1 aquifer. Overall; the inference from the modelled results is that, at the proposed rates of recharge and recovery for the MAR schemes, existing users will not have to change the pump setting and wells should not have to be deepened. However, pumps should be selected taking in to consideration the increased head in the aquifer at the start of the irrigation season.

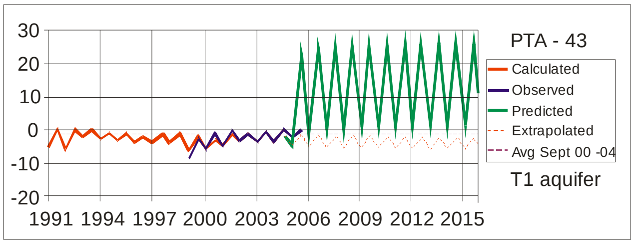

Figure 5 T1 aquifer hydrograph showing predicted seasonal fluctuations in groundwater levels under recharge and abstraction.

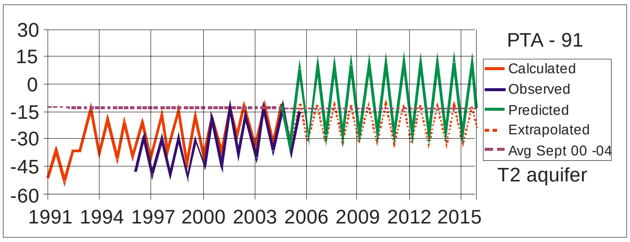

Figure 6 T2 aquifer hydrograph showing predicted seasonal fluctuations in groundwater levels under recharge and abstraction.

Model predictions indicate that in the T2 aquifer, adjacent to MAR schemes, seasonal drawdowns may increase by between 5 to 10 m (**Figure 6**). It is possible therefore that some existing users, depending on their proximity to the MAR schemes, may need to lower their pumps by up to 10 m to avoid pumping interference effects. Alternatively, if Council choose to irrigate at night this may minimise potential well interference associated with pumping between neighbouring users.

The seasonal amplitude in the T2 aquifer is significantly greater than in the T1 aquifer and pump selection will need to account for the additional operating head at the start of the irrigation season. It is possible that electricity costs associated with pumping will be greater from the T2 aquifer because of the increased seasonal drawdown of 5 to 10 m.

Artesian effects

Pressures generated during injection at the various MAR sites are likely to result in artesian conditions in the immediate areas surrounding the injection wells. The numerical groundwater model was used to predict the extent of artesian effects during injection. This information is important to determine the zone of hydrological influence that may extend away from the operating MAR wells.

In areas where groundwater levels are likely to become artesian, existing users may need to modify irrigation well heads to prevent wells flowing during the winter recharge season. Additionally, there is the potential risk that in some areas poorly abandoned or disused wells may be affected and require proper abandonment.

Model predictions indicate that artesian effects are likely to be observed at radial distances from the MAR wells of up to 1 km in the T1 aquifer and approximately 3 km in the T2 aquifer. Groundwater levels in the T1 aquifer are currently approximately 10 m below ground surface across much of the NAP. **Figure 7** illustrates the main area where artesian conditions are likely to occur is in the southern part of the NAP (Greenfields) where this scheme will be recharging up to 2,500 ML/yr. In the T2 aquifer groundwater levels are around 15 m below Australian Height Datum (AHD). On average the ground surface is 10 to 15 m above AHD giving a capacity of approximately 30 m before artesian conditions will occur. The main areas where artesian conditions are likely to occur are around the central NAP (associated with the Edinburgh Parks MAR scheme and to the north (Munno Para MAR scheme) as illustrated on **Figure 8.**

Most of the MAR schemes are located in areas away from existing irrigators, however, there are likely to be one or two irrigators especially in the southern part of the NAP where well heads of existing users may need to be sealed to prevent flowing during the winter recharge period.

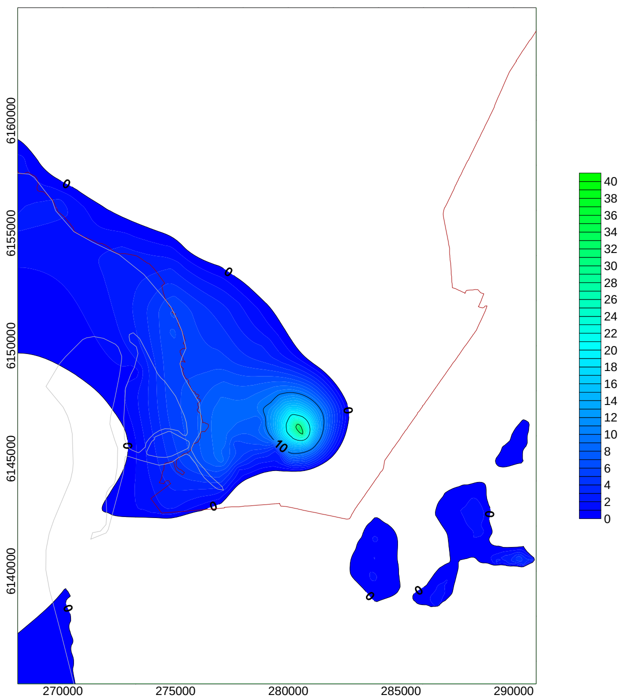

Figure 7 Extent of artesian effects at 2015 in T1 aquifer.

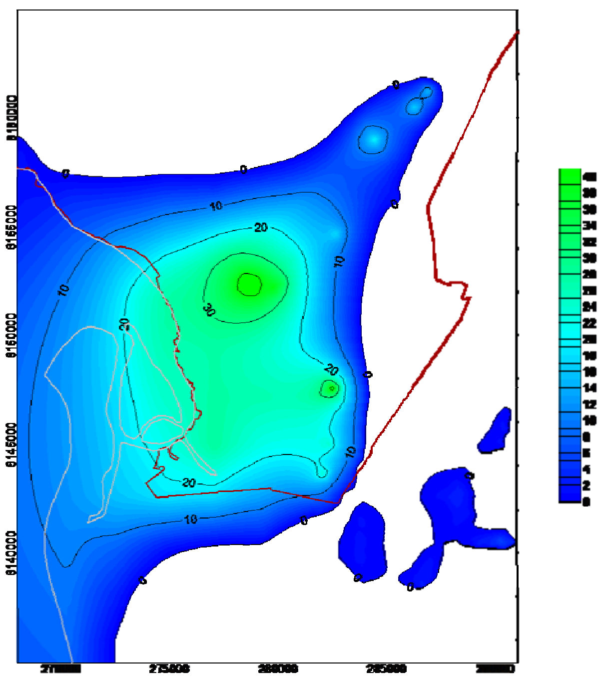

Figure 8 Extent of artesian effects at 2015 in T2 aquifer.

CONCLUSIONS

Overall, the MAR schemes and the existing management policies have a net regional benefit by causing groundwater levels to gradually rise in many areas as illustrated in Figures 5 and 6. Modelling of the cumulative impacts of multiple MAR schemes on the NAP has shown that:

- maximum operating heads during recharge into the T1 aquifer are likely to be in the order of 45 m (441 kPa) or less and 60 m in the T2 aquifer which is approximately half of the estimated safe operating head (150 m) for the two aquifers.

- a greater seasonal amplitude in groundwater levels in both the T1 and T2 aquifers occurs in response to the MAR activities; however, it is unlikely to impact significantly on existing users;
- the headworks to existing wells within a 1 to 1.5 km radius of the MAR schemes targeting the T1 aquifer and up to 3 km radius for MAR schemes that target the T2 aquifer should be inspected and if necessary modified to prevent artesian conditions.

Whilst the schemes will operate at considerably less that the estimated safe operating pressure of 150 m head it is difficult to predict what may happen to the confining bed separating the Q4/T1 and T1/T2 aquifers following repeated cycles of loading (recharge) and unloading (abstraction), in particular where this may be occurring in concert with some dissolution of the aquifer matrix. A possible outcome is that repeated loading and unloading, plus dissolution, may result in failure of the aquifer and/or confining bed. Failure of the confining bed would be catastrophic resulting in upward leakage into the overlying, and in some locations, more saline T1 aquifer.

The potential for failure of the confining bed requires further investigation. Where the confining bed between the T1 and T2 aquifers is known to be thin it is recommended that observation wells are installed in the overlying T1 aquifer. As a further precautionary measure it is recommended that the production wells are completed deeper in the T2 aquifer, ideally in the more competent horizon, which occurs some 10 to 15 m below the Munno Para confining bed separating the T1 and T2 aquifers.

Comparing the model zone budget pre and post MAR development the modelling results show that recharge of storm water into the T1 and T2 aquifers results in a reduced rate of inflow from the adjacent fractured rock aquifers of the Western Mount Lofty Ranges. However, this reduction in lateral inflow is offset by the current policies in the WAP which require 20% of the volume injected to remain in the aquifer.

Groundwater management policies within the potential zone of influence may need to be modified to protect the operators of MAR schemes and safe guard their access to the groundwater and also to protect any existing or future users from any adverse effects such as flowing wells or potential aquifer pollution by a contaminant that may inadvertently be introduced into the aquifer. Management options may include establishing buffers or attenuation zones around the recharge wells and suitable monitoring. Additionally, in order to provide greater operational flexibility for the MAR schemes management options may need to consider allowing the taking of groundwater at locations some three to five kilometeres removed from the main point of recharge.

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Framework for Feasibility Assessment and Performance Analysis of Riverbank Filtration Systems for Water Treatment

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Abstract
Bank filtration (BF) is an attractive, robust and reliable water treatment technology. It has been used in Europe and the USA for a long time; however the experience with this technology so far is site specific. There are no guidelines or tools for transfer of this technology to other locations, specifically to developing countries. A four step methodology was developed to analyze the feasibility, and to predict the performance, of BF for water treatment. The protocol included (i) hydraulic simulation using MODFLOW, (ii) determination of share of bank filtrate (%) using the NASRI bank filtration simulator, (iii) prediction of water quality from a bank filtration system using water quality guidelines developed, and (iv) comparison of the costs of BF and existing conventional systems for water treatment. The methodology was then applied to assess the feasibility of BF in five cities in Africa. It was found that, in most of the cities studied, BF is feasible and attractive option from hydraulic, water quality as well as cost considerations. Considerable O&M costs saving can be achieved and water quality can be further improved by switching from conventional chemical-based surface water treatment to BF, or at least by replacing some of the treatment units with BF.

Keywords
Riverbank filtration, water treatment, performance, framework, design, developing countries,

INTRODUCTION

Bank Filtration (BF), river or lake, is a reliable and proven natural water treatment technology, in which surface water contaminants are removed or degraded as water moves through the soil/aquifer to a recovery well(s). Extraction of water is accomplished by an infiltration gallery or line of wells (horizontal, vertical or at an angle) located at a short to intermediate distance from the bank of a river or lake. Because of its ability to remove even the most persistent contaminants and microbes, BF can support or even replace other treatment processes in a water treatment scheme. River bank filtration (RBF) is a traditional, efficient and well accepted method of surface water treatment in Europe. For more than 100 years, RBF has been used in Europe for public and industrial water supply along Rhine, Elbe, and Danube rivers (Grischek et al., 2002; Irmscher and Teermann, 2002). RBF systems are relatively simple to operate and robust, therefore they have a high potential for application in developing countries (Ray, 2008). However, this attractive technology has not been fully utilized in developing countries. Furthermore, the experience with BF technology so far is site specific and requires extensive site investigations and pilot studies to assess its feasibility based on local conditions. There are no guidelines or tools available for feasibility assessment, performance analysis or design of BF systems for water treatment. Therefore, the technology transfer of this viable and attractive multi-component removal technology is limited. The main objective of this study was to develop a framework or methodology to assess the feasibility and to analyze the performance of BF system for water treatment under given local (site specific) conditions.

DEVELOPMENT OF GUIDELINES FOR WATER QUALITY PREDICTION

The design or performance assessment of BF system requires analysis of hydraulic (water quantity) as well as water quality aspects of the system. There are a number of hydraulic models available for estimation of production capacity and draw down from a well under given hydrogeological conditions. However, there are no guidelines to predict water quality improvement during BF. Therefore, first of all, literature reviews of BF systems were conducted to compile the removal rates of four main water quality parameters, namely: (i) bulk organic matter, (ii) trace organic compounds, (iii) nitrogen species and (iv) microbes. Data from 33 literature sources on BF systems were analysed for removal of organic matter, nitrogen species, trace organics and microbes for different travel (residence) times and travel distances.

The data collected from the literature review on contaminant removal by BF technology at various sites was arranged in a spreadsheet according to classification together with the site conditions. The main parameters considered for removal were the travel time and the travel distance. Scatter plots of removal efficiency against the travel time or travel distance for each contaminant was then drawn for all of the parameters. A typical scatter plot for DOC removal during BF versus residence time is presented in Figure 1.

From the scatter-plots, data were grouped in different *bins* depending on the range of travel time or travel distance versus the removal efficiency. This step was necessary since the data were obtained from various sites and need to be normalized by calculating means and standard deviations. For data falling in each bin, means and standard deviations were calculated so as to achieve lower and the upper limit of the removal efficiency for that particular range of travel distance or travel time. Based on these values and looking at general trend, the guideline for removal efficiency for each contaminant versus residence times and/or travel distance was defined. A typical guideline for DOC removal during BF based on residence time is presented in Table 1.

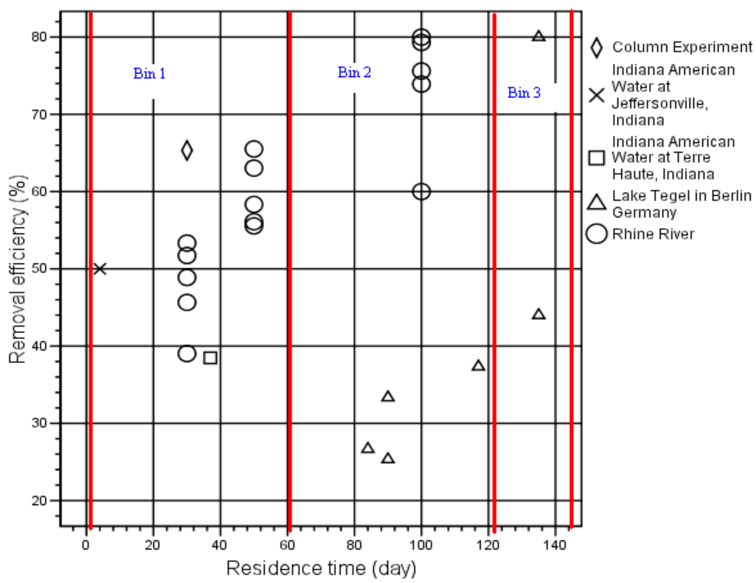

Figure 1 Scatter plot of DOC removal efficiency versus residence time for bank filtration systems

Table 1 Typical DOC removal guideline for bank filtration system

Influent range (mg/l)	Effluent range (mg/l)	Residence time (day)	Predicted removal efficiency (%)
1.0 – 7.5	0.5 – 5.6	1 - 60	44 – 62
		60 - 120	62 - 77
		120 - 145	77 - 87

FOUR STEP METHODOLOGY FOR FEASIBILITY ASSESSMENT OF BANK FILTRATION

The general framework for assessment of the feasibility of BF technology for water treatment at a particular location is presented in Figure 2.

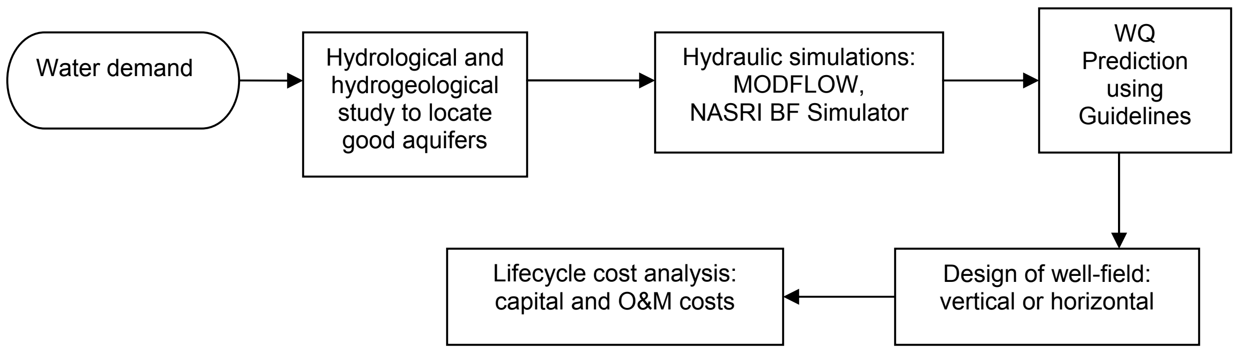

Figure 2 General framework for assessment of feasibility of BF technology at a particular location.

After collecting the data on water demand, water quality and hydrological and hydrogeological aspects of given site, the feasibility of BF could be assessed. The four step methodology developed for feasibility assessment of BF under given conditions consists of the following steps:

(i) Hydraulic simulation using MODFLOW

(ii) Determination of share of bank filtrate (%) and local groundwater using NASRI BF Simulator

(iii) Prediction of the water quality using algorithm/guidelines developed based on literature review

(iv) Comparison of the capital and O&M costs of BF system with that of existing conventional treatment system

Hydraulic Simulation Using MODFLOW

Hydraulic analysis software MODFLOW Pro (PMWIN Pro) was used to determine the pumping yield and drawdown corresponding to given location of extraction wells. MODFLOW requires input of hydrological and hydrogeological data including river flow, groundwater table, ground layers, aquifer depth and type, hydraulic conductivities and porosities. For the given allowable drawdown, the number of wells required for a given production capacity and their spacing and corresponding travel times were determined.

Table 2 shows a typical example of hydraulic analysis using MODFLOW for one of the sites for a well spacing of 50 m centre to centre. It was found that the required production capacity of 38,800 m^3/day could be achieved with different combination of number of wells and distance of wells from water source for different residence times. From this type of analysis, the number of wells required meeting the given criteria of drawdown in the well and minimum residence time can be determined. For example, for the given site, if the maximum allowable drawdown is 1 m and if minimum required residence time is 40 days (from water quality considerations), the appropriate option would be using 10 wells at a distance of 100 m from the source.

Table 2 Effect of number of wells and distance of wells from source on travel time and drawdown (well spacing = 50 m)

No. of Wells	Pump rate (m^3/d)	Distance of well from source = 50 m		Distance of well from source = 100 m		Distance of well from source = 150 m	
		Time (days)	Drawdown (m)	Time (days)	Drawdown (m)	Time (days)	Drawdown (m)
4	9,700	9	2.8	18	2.5	36	2.7
10	3,880	26	0.78	46	0.5	68	0.2
20	1,940	41	0.7	70	0.18	120	0.01

Determination of the Share (%) of Bank Filtrate and Local Groundwater

NASRI Bank Filtration Simulator (version 1.3a) developed during the NASRI (Natural Systems for Recharge and Infiltration) Project of Germany was used to determine the share (%) of bank filtrate and local groundwater for given well arrangements (Holzbecher 2006, Holzbecher et al., 2006). This simulator calculates the share of bank filtrate for a given location of wells based on (i) number of wells, (ii) well spacing, (iii) distance of the wells from river bank, (iv) pumping rates, (v) base flow, and (v) hydraulic conductivities. Figure 3 shows list of input and output parameters for the NASRI BF simulator and Figure 4 shows an example of NASRI BF simulator output display. The clogging parameter is the product of clogging layer thickness and relative impermeability compared with the aquifer. If there is no clogging layer, the value of the clogging parameter is zero.

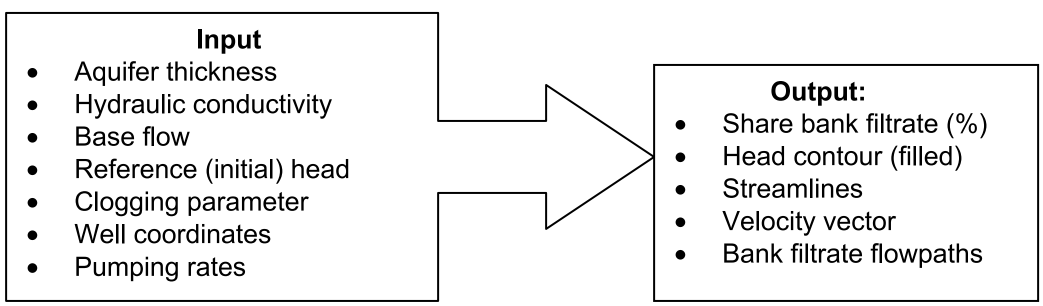

Figure 3 NASRI BF simulator data input and output parameters

The information obtained from this model is used to decide upon the number and location of the wells and for water quality predictions when the quality of surface water, contaminant removal rates and quality of native groundwater are known. Both of the hydraulic models used, MODFLOW and NASRI-BF Simulator can be employed to assess different hydraulic scenarios with respect to number, spacing and location of production wells.

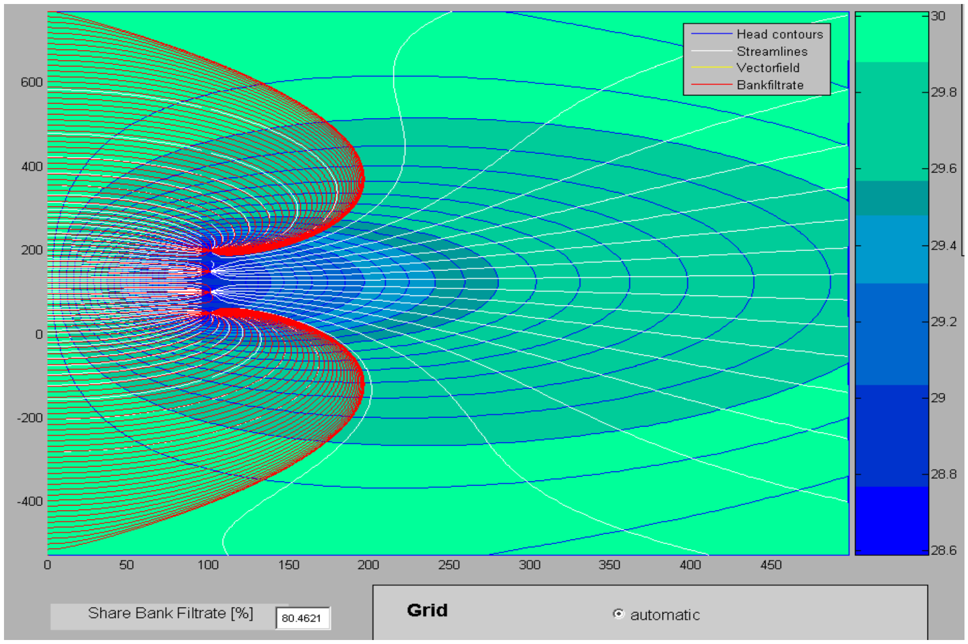

Figure 4 Sample NASRI BF Simulator output display (4 wells, 100 m from bank, 50 m well spacing C/C, hydraulic conductivity 0.006 m/s)

Prediction of Water Quality

After hydraulic simulations with MODFLOW and NASRI BF Simulator and development of guidelines for water quality prediction, the next step was to predict the quality of final bank filtrate. Removal efficiencies of a BF system depends on travel times/travel distance from the bank to the well. Final water quality from the BF system was determined by the combined effect of (i) the removal of given contaminant during soil passage and (ii) dilution effect of local groundwater. For the given well arrangements, the final water quality can be predicted using the mass balance equation below:

$$C = C_{river} * P_{RBF} * (1-R_{RBF}) + C_{GW} * (1-P_{RBF})$$

Where, C_{river} = Concentration of a contaminant in surface water,
R_{BF} = Removal efficiency of BF for the given contaminant (based on the guidelines developed)
P_{RBF} = % share of bank filtrate,
C_{GW} = Concentration of contaminant in native groundwater

Table 3 An example of prediction of water quality from a RBF system using the algorithm developed (Distance of wells from the bank = 50 m; Bank filtrate share = 94.1%, Travel time = 26 - 38 days)

		Groundwater quality			Surface water quality			Average BF Removal based on guidelines (%)	Final quality of bank filtrate (average)
Parameter	Unit	Min.	Avg.	Max	Min	Avg.	Max		
DOC	mg/L	0.9	1	1.2	1	2.7	4.5	53	1.25
NH_4^+	mg/L	0.63	0.63	0.63	0.00	0.00	0.00	74	0.04
Faecal Coliform	/100 mL	0	0	0	10	384	1600	100	0
Herbicides	µg/L	9.00	11.00	13.00	15	22	29	53.2	10.34

A typical example of prediction of water quality from a BF system is presented in Table 3. The bank filtrate quality is predicted corresponding to minimum, average and maximum water quality parameter values to check if the final bank filtrate meets the applicable water quality guidelines and/or standards for all possible cases. The outcome of the prediction of water quality of bank filtrate is used to determine the best location for the wells. This step is carried out for various distances of wells from the bank (50 m to 200 m) with varying percentages of share bank filtrate. Water quality is predicted for all hydraulically feasible options and the option meeting the local water quality guidelines or standards is selected. It should be noted to be that the larger the distance of the wells from the bank, the higher will be the removal efficiency. So the choice is between (i) a few wells near the bank producing relatively lower quality of water, which may require some treatment, or (ii) more wells farther from the bank producing relatively good quality of water.

Cost Calculations and Comparison

After selecting the design which meets both hydraulic and water quality requirements, capital and operation and maintenance costs of the BF system are estimated. Finally, cost comparisons are done between BF technologies and existing conventional water treatment plants in terms of capital cost as well as O&M costs (labour, power, and chemicals). For successful technology transfer to developing countries, cost is important for its sustainability. A decision can then be made regarding selection or further detailed analysis of suitable water treatment methods for given conditions.

The costs for establishing BF systems depend on many factors, including aquifer characteristics, type of well-screen installation, facility design, and distance to the population served. A BF system is valued by both the utility and consumers. The value of BF is not just found in reduced treatment and delivery costs, but also in the invaluable services it provides to the consumer, environment, and future generations (Ray et al., 2002). Furthermore, operation and maintenance needs vary due to size of the BF facility, types of wells employed, continuous or intermittent operation of wells, materials used for well construction, geologic environment and river conditions (Hunt et al., 2002).

APPLICATION OF ASSESSMENT METHODOLOGIES IN MALAWI AND KENYA

The assessment methodology developed was then applied for feasibility assessment of BF in five cities in Africa; three cities in Malawi; Blantyre, Lilongwe, and Mzuzu; and two cities in Kenya; Eldoret and Nakuru. Details of these case studies are presented elsewhere (Chaweza, 2006; Bosuben, 2007). In most of these water supply systems, surface water (river) is the main source of water and conventional surface water treatment (sedimentation, chemical coagulation/flocculation, clarification and chlorination) is applied. Water demand, water quality and treatment as well as hydrogeological data were collected from each of these sites to provide input for the different models. The pumping rates for wells in different cities ranged from 0.024 m^3/s to 0.2 m^3/s depending upon the hydraulic conductivities at the site, water demands and number of wells proposed. The distance of the wells from water source (river or lake) and the spacing between the wells both ranged from 50 to 100 m. Feasibility studies revealed that in all of the five cities, the hydrogeological conditions are favourable for BF and with the proper design of the production wells, existing water demand can be met with the BF systems. Furthermore, it was found that the quality of water produced by the proposed BF systems are comparable to that produced by existing treatment plants, and meet local drinking water quality standards and WHO guidelines.

Table 4 summarizes the salient features of the five water supply systems analysed for BF. Analysis of three water supply systems in Malawi showed that by switching from existing conventional treatment to BF, savings of over 80% on existing annual treatment costs (chemical and energy only) are likely for Blantyre and Lilongwe cities, but for Mzuzu the annual costs of BF and existing conventional water treatment are comparable. In Eldoret and Nakuru cities in Kenya, annual operational costs savings of about 16% and 32%, respectively, could be achieved by switching from conventional surface water treatment to a BF system. This analysis shows that the proposed four step methodology can be used as a preliminary screening tool to assess the feasibility of bank filtration system for water treatment at a given site.

Two immediate benefits of BF are: (i) minimized need for adding chemicals like disinfectants and coagulants to surface water to control pathogens and turbidity, and (ii) decreased costs to community due to reduced risks to human health (Ray et al., 2002). The costs for establishing riverbank filtration or artificial groundwater recharge systems depend on many factors, including aquifer characteristics, type of well-screen installation, facility design, and distance to the population served (Schmidt et al., 2008). The main capital cost components include land acquisition, construction of wells and pipelines from wells to treatment works and post-treatment. In general, BF systems are cost-effective as water quality is improved considerably after soil passage and subsequent treatment steps may be supported and simplified leading to decreased water treatment costs (Kim et al., 2003; Schmidt et al., 2008).

Table 4 Salient features of the water supply systems analysed for feasibility of BF

	Blantyre	Lilongwe	Mzuzu	Eldoret	Nakuru
Population	700,000	670,000	135,000	217,000	285,000
Water sources	Shire river, Mudi dam	Lilongwe river	Lunyangwa river	Ellegirini, Endoroto and Moiben rivers	Lake water (31%); Groundwater (69%)
Existing water treatment plant capacity (m^3/day)	110,000	75,000	15,000	22,000	35,950
Operational costs of existing treatment system (US$/$m^3$)	0.120	0.030	0.009	0.043	0.029
Share of bank filtrate in proposed BF system (%)	83	88	89	95	91
Estimated operational costs of BF system (US$/$m^3$)	0.098	0.014	0.009	0.036	0.020

CONCLUSIONS

- BF is a robust natural water treatment system which has high potential for application in developing countries, as the main treatment step or as the pre-treatment.
- A guideline for estimation of water quality of BF systems for a given travel time and/or travel distance was developed based on literature review. Guidelines for water quality prediction developed in this research can be used as a quick screening tool to make preliminary estimation of water quality. However, detailed design of the wells and pilot studies is required to make more accurate predictions or estimation of water quality for a specific BF site.
- A four-step methodology, using existing hydraulic softwares and the water quality guidelines developed, was proposed for assessing the feasibility of BF for water treatment. This methodology can be used for preliminary design of BF and to compare it with other above-ground conventional surface water treatment systems in terms of water quantity, quality and costs.
- Spacing between the wells and distance of the wells from the bank can be varied to obtain the required water quality and quantity from the wells for a given allowable drawdown and production capacity.
- Application of the methodology developed in selected cities in Malawi and Kenya showed that BF is feasible and attractive in these cities from hydraulic, water quality as well as cost considerations. It was found that considerable O&M costs savings can be achieved by switching from conventional chemical-based surface water treatment to BF.

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Development of a Flow and Transport Model in a highly Interactive Surface Water Groundwater System Near Everglades National Park in South Florida, USA

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Abstract

For an area along the southeastern boundary of the Everglades National Park, a three-dimensional integrated surface water-groundwater flow and transport model was developed and used as an analytical and management tool to investigate the operational effects of levees, canals, pumping stations, and detention basins on the groundwater flow pattern and water quality. The developed model simulates transient surface water and groundwater flow and transport in response to the operations of the system of canals, levees, and pumping stations, as well as climatic conditions. The model covers an area of more than 400 square kilometers and was calibrated to a set of pre-specified calibration criteria. Available data of total phosphorus concentration, potentiometric elevation, canal and detention basin stages, and flow through canals between 2000 and 2007 were used as quantitative calibration targets. Details of model development, calibration and sensitivity analysis results are presented and discussed.

Keywords

Everglades, surface water, groundwater, integrated flow and transport modelling, double porosity model, phosphorus transport.

INTRODUCTION

Everglades National Park (ENP) in south Florida, with the largest subtropical wilderness in the United States, was created to protect a fragile ecosystem instead of safeguarding a geographic feature. In 2000, the U.S. Congress authorized the Comprehensive Everglades Restoration Plan (CERP), the largest environmental restoration effort in history. The CERP provides a framework and guide to restore, protect, and preserve the water resources of central and southern Florida, including the Everglades. Along the southeastern adjoining areas of ENP in the C-111 canal basin, a number of hydraulic structures have been constructed to help regulate the groundwater flow pattern within ENP (see Figure 1). The C-111 canal separates ENP from highly productive agricultural lands to the east. In this area, surface water is highly interactive with groundwater. This area is underlain by Biscayne Aquifer, one of the most transmissive aquifers in the world (Fish and Stewart, 1991). Owing to the extreme permeability of the Biscayne Aquifer in this area, the canals have a direct impact on water levels in adjacent areas. The C-111 South Dade Project is intended to provide both a flood control function and an ecological restoration function. It seeks to minimize drainage of the adjacent wetlands by pumping water from the L-31N canal into detention basins west of the L-31N canal to maintain such water levels that reduce west-to-east gradient draining the Rocky Glades region of the ENP (Figure 1). Phosphorus distribution and transport are also of concern in this area as the Everglades are oligotrophic and phosphorus inputs from canals that drain agricultural areas have resulted in ecosystem changes (Davis, 1994; Noe *et al*., 2003; Gaiser *et al*., 2004)

One of the objectives of this investigation was to quantify the dynamic phosphorus mass balance since the Everglades are fairly sensitive to the phosphorus input from the canals. Phosphorus sampling at the S-332D detention area, at the outflow from the detention area, and in the middle of the detention area showed that the outflow phosphorus concentrations were often greater than inflow concentrations possibly resulting from construction activities unearthing phosphorus in the soil from past agricultural activities (U.S. Army Corps of Engineers [USACE], 2002). Munoz-Carpena and Li (2003) also noted that concentrations of total phosphorus (TP) in the L-31W canal samples were consistently higher than those obtained from the C-111 canal.

Phosphorus was also noted in shallow and deeper wells in the Frog Pond area. Walker (2003) used statistical analyses to identify sites where changes in average flows, concentrations or loads are likely to have occurred following Interim Operational Plan implementation in late 1999. At the study site phosphorus is present in the form of dissolved and organic phosphates which constitute TP. The main influx of TP is through the L-31N canal from which water-borne TP gets pumped into the S-332BN, S-332BW, S-332C, S-332DN, and S-332DS detention basins (Figure 1). The objective of the current study is to develop a model that can be used to simulate integrated surface water-groundwater flow as well as the fate and transport of TP in the S-332 detention basins near ENP within the study area (Figure 1).

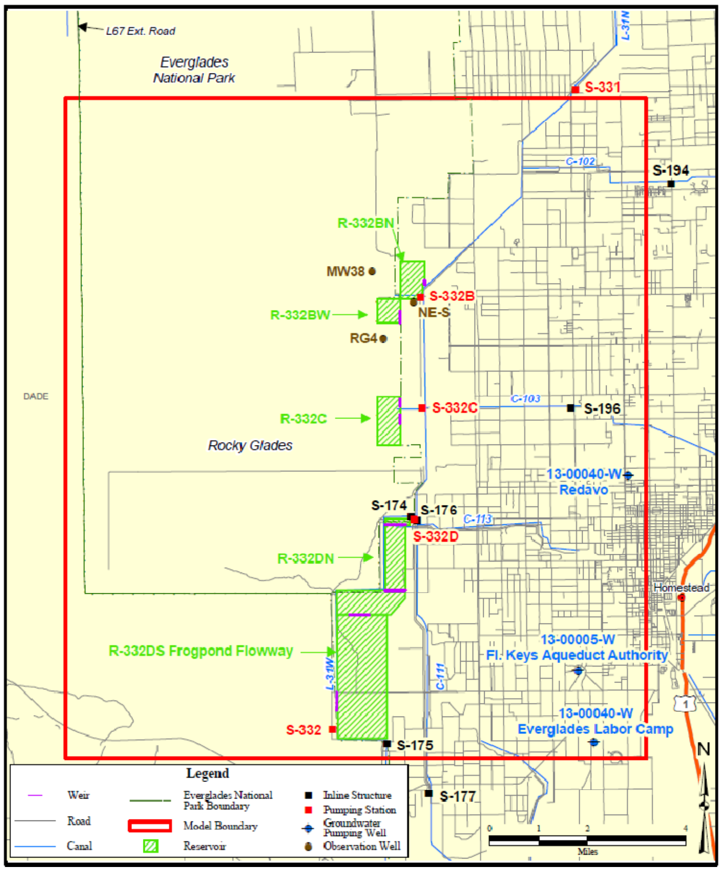

Figure1. Study area showing hydraulic structures, pumping stations, detention basins, and example observation locations (blue solid circles).

CONCEPTUAL MODEL

The model domain is located in south Miami Dade County along the L-31N canal (Figure 1). The model covers approximately 400 sq. km (157 sq mi) of area. The northern boundary of the model is about 335 m (1,100 ft) south of the S-331 pumping station and the southern boundary is about 1,112 m (3,650 ft) north of the S-177 pumping station for a total north-south extent of approximately 21.4 km (13.4 mi). The domain extends approximately 11.4 km (7.1 mi) west and 7.4 km (4.6 mi) east of the L-31N canal.

The model is represented by two hydrologic regimes – subsurface flow and surface flow which includes the overland flow (OLF) and channel flow domains. Part of the subsurface system is drained by the L-31W and C-111 canals. The L-31W canal traverses from the northern boundary to the southern boundary of the S-332D basin. TP from outside the study area is first introduced into the surface system via the S-331 structure and pumped into the S-332BN, S-332BW, S-332C, S-332DN, and S-332DS detention basins.

The hydrostratigraphy of the model area consists of the surface sediments underlain by the Biscayne Aquifer. In the western portion of the model area, the Biscayne aquifer is underlain by the Gray Limestone Confining Unit, and the Gray Limestone Aquifer. To the east beyond the extent of the Gray Limestone, the Biscayne Aquifer is underlain by the Lower Clastics of the Tamiami Formation. The Biscayne aquifer is a karst aquifer and is conceptualized as a double-porosity medium of conduits and matrix porosity (Cunningham *et al*., 2009).

Water in the basins is derived from precipitation and water pumped from the canal. This water infiltrates downward through the bottom of the basins (which can be the top soil if present or exposed limestone) to the karst aquifer below. Carried by the infiltration water is the TP from the canal, from wet atmospheric deposition, and from the leaching of the top soil. Around the detention basins, the water table is usually a few feet below the top soil-limestone contact. When the basins are inundated by water from the canal or heavy precipitation, the groundwater elevation in the limestone immediately below rises well above the top soil-limestone contact, thereby allowing the groundwater to reach chemical equilibrium with the top soil (where present). Once the TP enters the groundwater, part of it is transported back to the canal and reemerges into the canal downstream.

MODEL DEVELOPMENT

A three-dimensional integrated surface water-groundwater flow and transport model was developed to investigate the groundwater flow pattern and water quality in and around the detention areas in response to the operations of the system of canals, levees, and pumping stations, as well as climatic conditions. The model was discretized into 4 subsurface layers with 97 rows and 97 columns. Vertical and horizontal discretization of the groundwater grid of the model was based on the P-D-M model (the USACE Jacksonville District in the Pennsuco-Dade-Monroe model [Evans, 2003]). The stratification information of subsurface layers and the associated hydrogeologic properties for the model were imported from the P-D-M model. The top-most subsurface model layer represents the surface sediments. Subsurface model layer 2 represents the Biscayne Aquifer. The approximate eastern extent of the Gray Limestone Aquifer (Fish and Stewart, 1990) passes through Limestone Aquifer respectively, west of this line. To the east of the approximate eastern extent of the Gray Limestone Aquifer, model layers 3 represents the lower portions of the Biscayne Aquifer, and model layer 4 represents the Lower Clastics of the Tamiami Formation.

Surface-water flow is calculated on the OLF grid which coincides areally with the groundwater grid. Grid resolution varies in the horizontal from 73.15 to 731.5 m (240 to 2,400 ft) and in the vertical from 0.03 to 36.6 m (0.1 to 120 ft). The main canal feature present within the study area is the L-31N canal flowing into the domain from the north. This canal branches out into canals C-102, C-103 and C-113 before it splits into C-111 and L-31W canals. The canal network in the model domain was defined by 325 channel nodes.

MODHMS (HGL 2007), a state-of-the-art MODFLOW-based Hydrologic Modeling System, was used to perform comprehensive simulations of conjunctive surface-water/groundwater flow and TP transport. MODHMS performs integrated hydrologic analysis using a fully coupled solution of the diffusion wave equations governing overland flow and channel flow with the Richards equation governing unsaturated/saturated groundwater flow.

DATA

Hydrostratigraphic and hydraulic information in subsurface layers were obtained from the P-D-M model. The information includes: subsurface layer geometry (thickness and elevation), hydraulic properties (hydraulic conductivity, leakance, and storativity). There are three groundwater pumping wells, east of the C-111 canal, within the model domain of this study (Figure 1).

Manning's coefficient distribution for flow along the OLF domain within the wetland areas was obtained from the Southern Inland and Coastal Systems (SICS) study of Swain *et al.* (2004). For the cultivated soil to the east of the ENP, Manning's coefficient for flow along the OLF domain within the agricultural areas was obtained from the USDA (1986) database. The rill storage height was set according to the land use types (marl prairie and citrus / row crops) as defined by the South Florida Water Management Model (SFWMM) and P-D-M model. The SFWMM developed by the SFWMD (SFWMD, 1997) is a regional-scale hydrologic model used as a planning tool for system-wide evaluations in south Florida. Data for the channel segments, (Manning's coefficient, leakance, and channel geometry) were extracted from the SICS study (Swain *et al.*, 2004), the SFWMM model, and the P-D-M model

Water level information is available on a daily basis from January 1, 2000 through December 31, 2007 at 36 observation locations (wells, staff gauges or combinations of both) within and around the model domain. The water level information was also used to provide initial and boundary conditions on the OLF domain and in the subsurface. The spatially interpolated head distribution of January 1, 2000 was used as initial conditions for a preliminary simulation which involved stabilizing the solution to steady-state conditions from which transient simulations were initiated.

Boundary conditions for the L-31N canal running across the model domain include a flux boundary at the upstream reaches where water enters the domain. Daily flow observations from the S-331 monitoring station (see Figure 1) were used as the inflow boundary condition at L-31N canal for the model. This information was available for the period from January 1, 2000 through December 31, 2007.

The reservoirs or detention areas that exist within the model domain are shown in Figure 1. There are four detention basins within the model domain– S-332BN, S-332B, S-332C and S-332D. The detention basins are typically surrounded by 6 ft high berms which prevent the flow of water across these basins. There are four pumping stations that exist in the L-31N canal within the model area and one (S-331) just north of the domain. The pumping stations (Figure 1) move water to their respective detention basins. There are also six inline structures (gated weirs and spillways) existing within the model area as located in Figure 1. Various structures are present along the berms of the detention basins to let water flow in and out of the basins. These structures are shown in Figure 1 and generally include weirs or culverts cut into the berm walls of the detention basins.

Data from eight rainfall gauging stations was used to apply average daily rainfall over the model area. This information was from January 1, 2000 through December 31, 2007 and was available on an hourly basis. The hourly data were summed to create daily inputs. The distribution of rainfall was estimated by interpolating between the rain gauges.

Evapotranspiration (ET) was simulated according to a parametric model for water removal which describes the ET flux as a function of depth. The maximum ET flux occurs at or above a user defined maximum ET surface elevation, which linearly reduces to zero at or below the extinction surface elevation. The maximum ET surface was set at the shallow root zone and the extinction surface at the deep root zone. The maximum ET flux, which varied on a daily basis, was calculated as an average of the daily values over the simulation time period, at two ENP ET sites located just north and south of the model area.

Groundwater TP concentration data between 2000 and 2004 were collected by the USACE and provided by the NPS (NPS, 2008). There are ten monitoring wells, seven of which are shallow and screened in Layer 2 of the model. The remaining three wells are screened in Layer 3 of the model. These ten wells are located within or around the S-332B basin. Concentration data from these ten wells were used to establish the transport model's initial conditions. Groundwater TP concentration data between 2005 and 2007 were collected and provided by Gaiser *et al.* (2008). There are twenty-one monitoring wells, all of which are shallow and screened in Layer 2 of the model. The depth of these wells varies from 4 to 14 feet. There are eight, three, nine, and one observation wells in and around Basins S-332BN and BW, S-332C, S-332DN, and S-332DS, respectively. Concentration data from these wells were used in the calibration of the transport model.

For some periods between 2000 and 2007 water with water-borne phosphorous was pumped from the L-31N canal to the four detention basins. Daily TP concentration and pumping data related to all S-332 basins were provided by the NPS in 2008 (NPS, 2008). The daily TP concentration and pumping rate data were used as input conditions for the model to ensure that the estimation of the TP flux through the basins is as accurate as possible. The background TP concentration in the canal is typically low, on the order of 8-10 µg/L. Atmospheric wet TP deposition is based on the data across Florida compiled by Pollman *et al*. (2001). At the ENP study site, the volumetric weighted mean TP concentration for wet deposition was found to be 8 µg/L. This level of concentration is comparable to the typical TP concentration in the canal.

Data relating to top soil thickness, TP distributions in soil and limestone, and physico-chemical properties of the soils and limestone were obtained from several sources. For additional details, the reader is referred to HGL (2006, 2010).

CALIBRATION STRATEGY AND RESULTS

The calibration strategy was designed to produce a model that is capable of simulating the general surface water and groundwater flow patterns and regional trends in TP concentration. The calibration metrics used in the calibration were: observed groundwater and surface water elevations, observed flow rates through inline structures, and observed TP in surface water and groundwater. The purpose of the calibration effort was to minimize the residual errors (=simulated - observed) at target locations. For this model, the calibration criteria for mean residual errors (MEs) and root mean square errors (RMSEs) for the flow component were 5 and10 percent of the respective differences between the observed maximum and minimum. For the transport component, the corresponding criteria were 10 and 20 percent, respectively.

Model simulations for calibration were conducted as follows. A steady-state flow condition was first simulated by the model to represent January 1, 2000 conditions, which provides a reproducible starting state from which transients of the model were simulated. The transient simulation was performed for a period of eight years from January 1, 2000 through December 31, 2007. Data between 2000 and 2003 were used for calibration and data from the remaining period were used for verification. Once the flow model had been calibrated, the transport component was calibrated. The model was calibrated using an expert interactive (manual) calibration approach. Initial calibration simulations were first conducted to probe the sensitivity of the model to critical parameters, so as to note cause-and-effect for determining alterations from the original dataset that were necessary to achieve calibration. Following this, the model parameters were altered systematically such that calibration goals were realized.

For the flow component of the model, excellent agreement between observed and simulated water elevations was obtained. Comparisons between observed and simulated groundwater elevations at well RG4 and observed and simulated surface water stage at the S-174 canal inline structure are shown in Figures 2 and 3, respectively. For the transport component of the model, the model is able to capture the variation of TP concentration between 2005 and 2007 with time reasonably well around the detention basins. An example for well MW38 is shown in Figure 4. Note that the peak TP concentrations predicted by the model correspond to peak precipitation but the observed high concentrations tend to lag behind the peak precipitation. However, some of the spikes do not correspond to peak precipitations, suggesting relatively complex transport processes of sediments and floc at the surface as well as complex preferential flowpaths in the aquifer below. In Figure 5, it can be seen that at well NE-S the model is able to simulate the decay of the elevated TP concentration mound that appeared prior to 2000 and the peaks that appear in 2002 and the end of 2003 reasonably well. A representative TP concentration in canal versus time curve near the S-332B pumping station is shown in Figure 6 that shows the TP concentration in the channel as predicted by the model agrees reasonably well with the observed data. All these above-mentioned observation locations are shown in Figure 1. The predetermined calibration criteria were met satisfactorily.

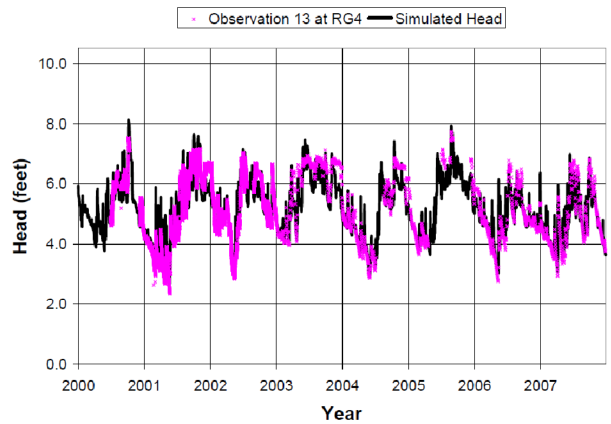

Figure 2. Groundwater head at well RG4 vs. time

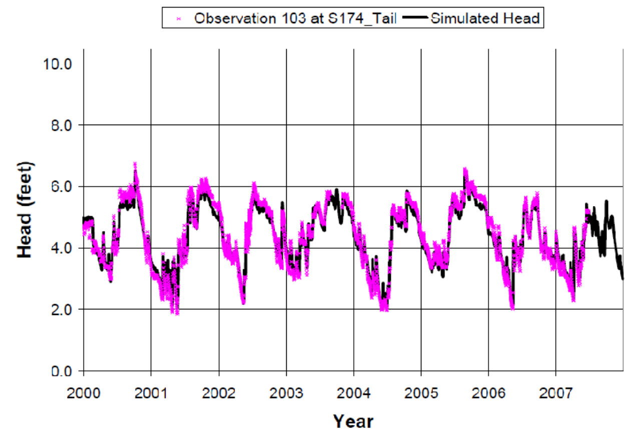

Figure 3. Stage at inline structure S-174 vs. time

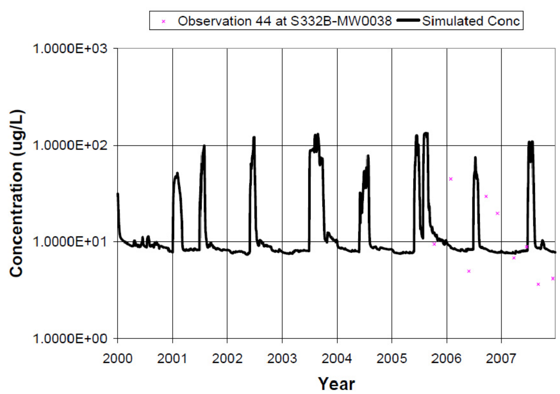

Figure 4. TP concentration at well MW38 vs. time

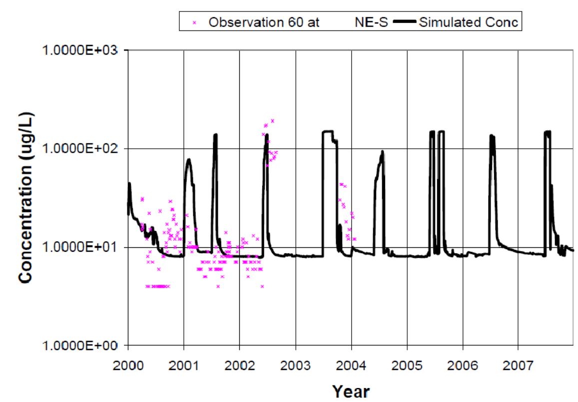

Figure 5. TP concentration at well NE-S vs. time

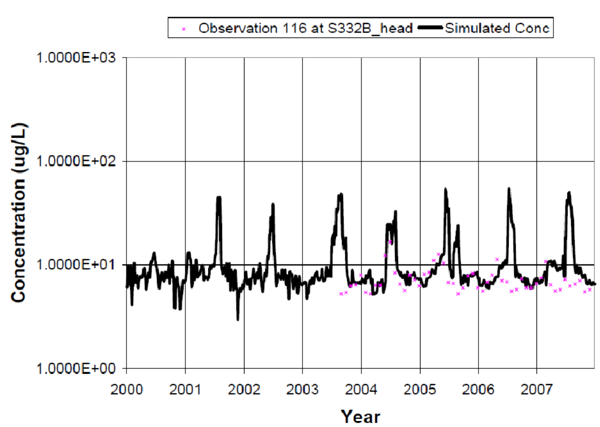

Figure 6. TP concentration in the L-31N canal at Basin B vs. time

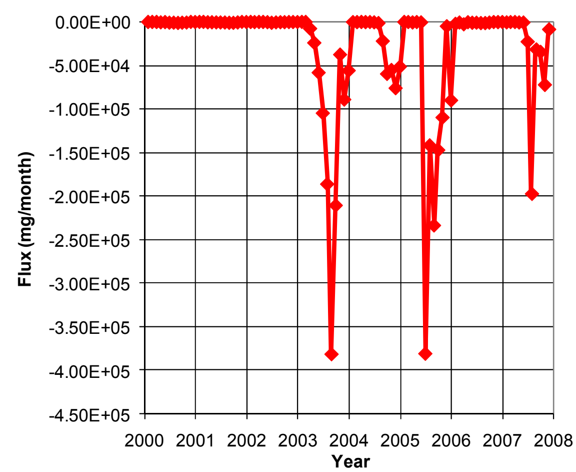

Figure 7. Estimated TP flux (mg/month) through the S-332BN basin vs. time

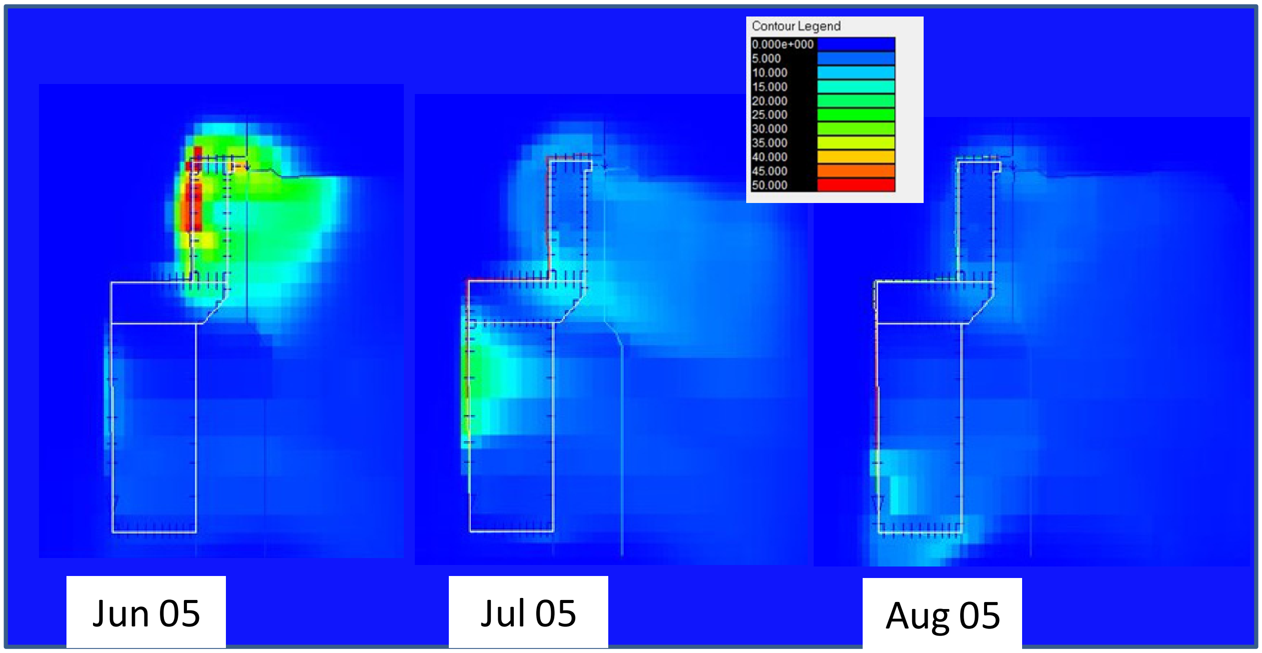

Figure 8. Tracer Distribution below the S-322D basin in the Biscayne Aquifer (concentration values are in mg/L)

SENSITIVITY ANALYSIS

A number of sensitivity analysis runs were performed for both the flow and transport components of the model. For the flow component, the model (in terms of head and flux) was found sensitive to pumping rate at pump stations, stage rating curves for structures, aquifer hydraulic conductivity, and canal leakance. For the transport component, the model was found sensitive to distribution coefficient and dual-porosity mass transfer rate. Details are provided in HydroGeoLogic (2006, 2010).

MODEL APPLICATIONS

The developed model has been used as an analytical and management tool to investigate the operations and the impacts due to the construction of levees, canals, pumping stations, and detention basins on the groundwater flow pattern and water quality as part of the Mod Waters C-111 project. Examples include determination of water and TP budgets for the detention basins (see Figure 7 for an example for the S-332BN basin) and determination of the movement pumped-in water through tracer (simulated) in the S-332D basin. Figure 8 shows, in this simulation, that very little tracer (along with water) from Basin D moves into the ENP area. This simulation showed that all the tracer and water from Basin D that migrated into the ENP area eventually flushed out to the east. Details are provided in HydroGeoLogic (2010).

CONCLUSIONS

A three-dimensional integrated surface water groundwater flow and transport model has been developed for an area along the southeastern boundary of the ENP. Available data of TP concentration, potentiometric elevation, canal and detention basin stages, and flow through canals between 2000 and 2007 were used as quantitative calibration targets. Sensitivity analysis of the model indicated sensitivity to pumping rate at pumping stations, distribution coefficient for TP, and dual porosity mass transfer rate. The developed model is currently used as an analytical and management tool.

ACKNOWLEDGMENTS

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Recovery Efficiency Assessment of an ASR Well Using Groundwater Models

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Abstract

Aquifer Storage and Recovery (ASR) has become an effective means of water resources management in many places. ASR is particularly effective where water availability varies seasonally. A measure of the performance of an ASR system is recovery efficiency. The recovery efficiencies of ASR systems depend upon a number of factors such as aquifer transmissivity, vertical permeability of confining units, native water quality, regional groundwater flow gradient, etc.

The success of an ASR system depends upon site-specific hydrogeological conditions. However, a theoretical modeling assessment of the controlling variables can provide useful insights for ASR site selection and system design. A model with variable density groundwater flow and solute transport was developed using the USGS computer code SEAWAT to evaluate how the physical characteristics of an aquifer system impact ASR recovery efficiency. Typical aquifer parameters from some existing and proposed ASR sites were used in the model. Simulations were performed for typical operational cycles for water utility-scale systems. The modeling results indicate that aquifer vertical anisotropy, heterogeneity, native water quality and regional gradients are the most significant factors for the recovery efficiency of an ASR system.

Key words: ASR recovery efficiency, factors, modeling assessment

INTRODUCTION

Aquifer Storage and Recovery (ASR) is increasingly being recognized as tool for more effective water resource management. The storage provided by ASR systems can be used to capture water that would otherwise not be put to beneficial use and to increase the efficiency of operation of water supply infrastructure. As a storage technology, the performance of ASR system necessarily is evaluated based on the amount of additional water that the system provides when needed.

The performance of an ASR system is usually measured in terms of recovery efficiency. For ASR systems that store freshwater in aquifers that contain poor quality (e.g., brackish or saline) water, recovery efficiency is quantified in terms of the percentage of the injected water that can be recovered at a usable quality (Kimbler *et al*., 1975). Drinking water standards are usually used for the recovery thresholds of potable water system. There are two different definitions of recovery efficiency: the cycle (or operational) recovery efficiency and the project (or system) recovery efficiency. The cycle recovery efficiency is defined as the ratio of total amount of recovered water to the amount of volume of injected during an operational cycle, which is typically on an annual basis. The project recovery efficiency is defined as the total amount of water recovered to the total amount of water injected for entire project, including the amount of the water pre-injected. In this study, the cycle recovery efficiency is applied.

The recovery efficiency of an ASR system is controlled by a number of factors, including aquifer transmissivity, vertical anisotropy, heterogeneity, dispersivity, native water quality and regional groundwater flow gradients. These factors will determine the fate of injected water, or the shape of the injected bubbles, and have been the focus of a number of recent studies (Missimer et al., 2002; Vacher et al., 2005; Maliva et al., 2005; Lowry and Anderson, 2006; Sedighi et al., 2007; Ward, et al., 2007).

A generalized 3-dimensional SEAWAT model was developed based on hydraulic data commonly encountered in existing or proposed ASR systems in Florida. The objective of the modeling was to evaluate the impacts of the various hydraulic parameters on recovery efficiency. Such information could be used to develop more accurate screening tools for future ASR systems and for the optimization of system design.

MODEL SETUP

The hypothetic model, as shown in Figure 1, has ten flat layers with a uniform layer thickness of 3.48 m. No-flow boundary conditions were set at the top and the bottom. Only half of the aquifer area was modeled to take advantage of the model symmetry and reduce computational times. The model has 50 rows and 101 columns. A uniform grid size of 17.4 m was applied in both row and the column directions. The ASR well was located at the first row at the middle of the model. The well is open to the model layer 6. Other hydraulic parameters are shown in Table 1.

One complete cycle of aquifer storage and recovery was considered (Table 2). The entire duration of the cycle was assumed to be 120 days for continuous injection, 120 days for storage and 125 days for recovery. Therefore, in each run, the total volume of injected water is 218,400 m^3. Recovery efficiency is calculated as the ratio of the amount of water recovered before the chloride concentration reaches 250 mg/l and the total amount of water injected.

Simulation of the only one operational cycle will result in modeled low recovery efficiencies because of the absence of an existing buffer or mixing zone. Recovery efficiency in most ASR systems improves over time as a buffer zone is developed around the ASR zone. Nevertheless, simulations of single operational cycles can provide insights into the relative importance and magnitude of the effects of hydraulic and water quality variable on system recovery efficiency.

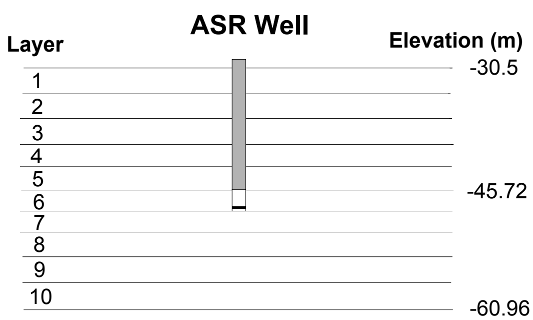

Figure 1. Schematic of Model Structure.

Table 1. Hydraulic parameters used in the base run.

Horizontal hydraulic conductivity	**30.5 m/day**
Vertical hydraulic conductivity	**30.5 m/day**
Longitudinal dispersivity	**15.2 m**
Transverse and vertical dispersivity	**1.52 m**
Regional groundwater gradient	**0.0**
Effective porosity	**(-)**
Inject chloride concentration	**50 mg/l**
Ambient chloride concentration	**500 mg/l**

Table 2. Simulation Time Set up

Stress Period	Duration(days)	Well Operation	Injection/Pumping Rates (m3/day)	Injection Concentration (mg/l)
1	120	Injection	1820	50
2	120	Storage	0	n/a
3	125	Recovery	-1820	n/a

SEAWAT, widely used throughout the world (Guo and Bennett, 1998; Guo and Langevin 2002) was used to construct the model. SEAWAT is a computer program that couples two popular codes, MODFLOW (McDonald and Harbaugh 1988) and MT3DMS (Zheng and Wang 1998) for flow with variable density. This would allow consideration of the density effect in the case of the ambient water in the storage aquifer is brackish. SEAWAT solves two coupled partial differential equations (Guo and Langevin 2002). The governing equation for the flow in terms of freshwater head is:

$$\nabla \cdot \rho K_f \left(\nabla h_f + \frac{(\rho - \rho_f)}{\rho_f} \nabla z \right) = \rho S_f \frac{\partial h_f}{\partial t} + n \frac{\partial \rho}{\partial C} \frac{\partial C}{\partial t} - \rho_s q_s$$

where h_f is the equivalent freshwater head [L], K_f the hydraulic conductivity [LT^{-1}]; ρ the fluid density [ML^{-3}]; ρ_f the freshwater density [ML^{-3}]; S_f the storage coefficient in terms of freshwater head; ρ_s [ML^{-3}] q_s represents the volumetric flow rate per unit volume of aquifer representing source and/or sink terms [T^{-1}]; C the salt concentration [ML^{-3}], and t represents time [T].

The governing equation for solute transport in porous media is:

$$\frac{\partial C}{\partial t} = -\nabla \cdot (\vec{v} C) + \nabla \cdot (D \cdot \nabla C) - \frac{q_s}{\theta} C_s + \sum_{k=1}^{N} R_k$$

where D is the hydrodynamic dispersion coefficient tensor [L^2T^{-1}]; v the flow velocity [LT^{-1}]; Cs the source concentration and θ the effective porosity.

The fluid density is defined as a linear function of salt concentration:

$$\rho = \rho_f + \frac{\partial \rho}{\partial C}(C - C_o)$$

where Co is the salt concentration for freshwater [ML^{-3}]. Practically Co is equal to zero.

ASSESSMENT OF RECOVERY EFFICIENCY

Base Run

The major hydraulic parameters used in the base run are shown in Table 1. The initial head was set as 0.0 m above the sea level. The model was run for a year as a full cycle. The recovery efficiency of the base rum simulation was approximately 71% based on recovery to a chloride concentration of 250 mg/L. The head at the injection zone at the end of injection is about 5.3 m about the sea level. At the end of storage period, the head at the well drops to 2.1 m. Figure 2 shows the chloride concentration at the well versus time. Chloride concentration in the model cells containing the ASR well reduces quickly to the concentration of injected water. The water quality has little change during the storage period and starts to increase when the recovery process begins. The chloride concentration in the well reaches 250 mg/l after approximately 85 days of recovery.

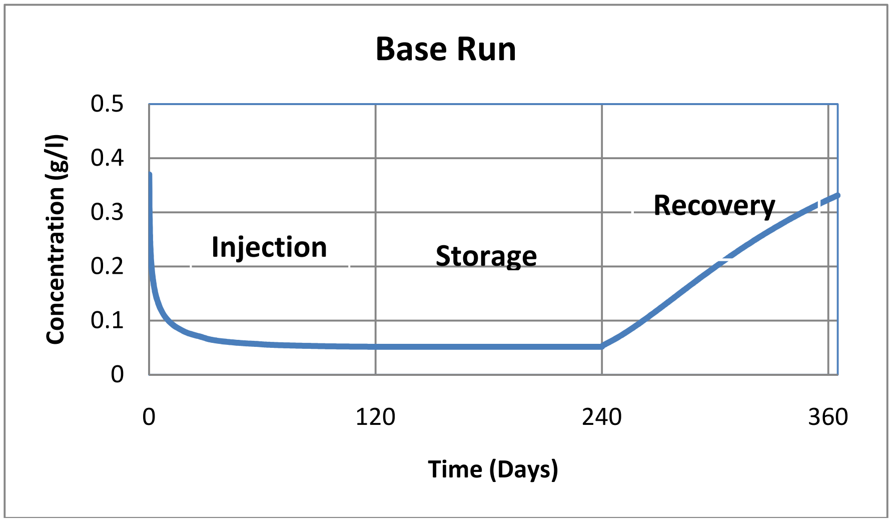

Figure 2. Calculated chloride concentration change with time at the ASR well.

Permeability of Confining Units

Two confining units, represented by model layers 4 and 8, respectively, were simulated by reducing their horizontal and vertical hydraulic conductivities relative to those of the storage aquifer. The horizontal and vertical hydraulic conductivities of all cells in each layers are assumed to be the same. The simulation results indicate the recovery efficiency is not sensitive to the permeability of the confining units (Figure 3). It should be noted that this observation is based on the geometry and specifications of this hypothetic model and it may not be applied to all the field conditions. For example, where the salinity of the ambient groundwater is high, the injected freshwater volume will tend to move upward and the existence of a confining unit above the injection zone would help keep the injected water from moving away.

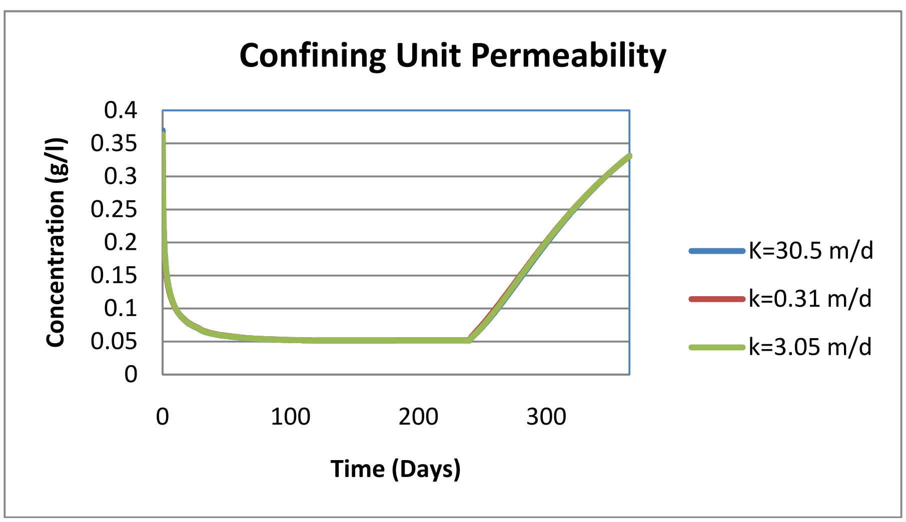

Figure 3. Chloride concentration (g/l) changes versus time (days).

Vertical Anisotropy

Vertical anisotropic ratio may play an important role in determine the shape of injected bubble (Missimer et al., 2002; Maliva et al., 2005). In the base run, an isotropic condition was assumed. In this group of tests, different anisotropic ratios were applied, while all other variables were unchanged. The simulation results indicate that recovery efficiency is not sensitive to the vertical anisotropic ratio (Figure 4). However, the head distribution is quite different in each case. The lower the vertical hydraulic conductivity, the higher the head at the well point. At the end of injection period, the head is 5.33 m while the head is 13.71 m when the vertical anisotropic ratio is 100 to 1. This is to say higher pressure has to be applied to maintain specific injection rate.

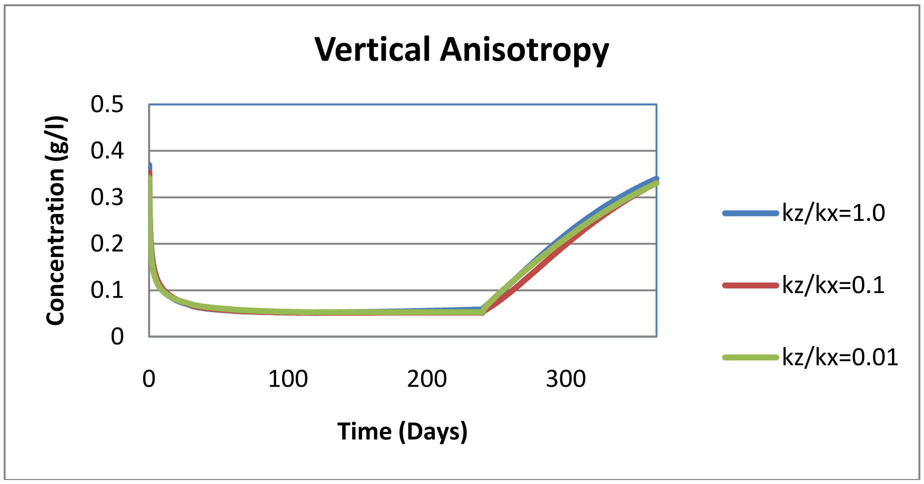

Figure 4. Chloride concentration (g/l) changes versus time (days).

Ambient Salinity

The water quality of the ambient water may influence the recovery efficiency by two processes: (1) the fluid density and mixing between injected water and native water. In general, the higher the ambient salinity, the lower the recovery efficiency for an ASR system (Kimbler et al., 1975; Ward et al., 2007). The density differential will push the injected bubble upward from the injection point and while mixing process will definitely increase the salinity of stored water. The results of this group of simulations indicate the recovery efficiency of an ASR well is very sensitive to the salinity of ambient water in the storage aquifer. Figure 3 shows the concentration changes at the well when the ambient salinity is 500 mg/l (the base run case), 1000 mg/l and 5000 mg/l, respectively. The recovery efficiency is reduced from 71% in the base case to 33% and 6.7%, respectively.

Transmissivity

The transmissivity of the storage aquifer plays an important role in controlling the lateral expansion of injected bubbles, as well as the injection pressure applied (Missimer et al., 2002). If the storage aquifer is limited in the vertical direction, and assuming sufficient pressure is applied and the density differential is not significant, the injected freshwater volume will have the shape of a cylinder that progressively expands laterally. Under these conditions, recovery efficiency may not be sensitive to the aquifer transmissivity, but the head distributions could be very different.

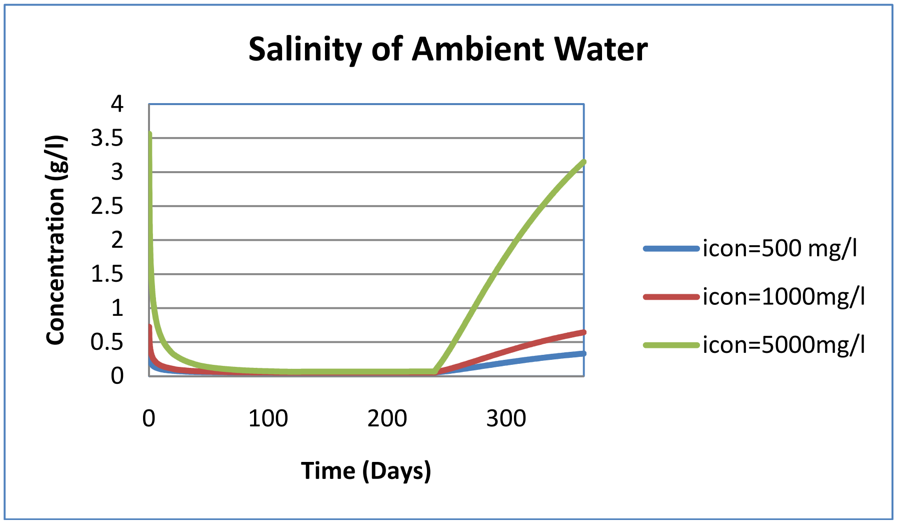

Figure 5. Chloride concentration (g/l) changes versus time (days).

Simulations were performed in which all hydrogeologic conditions were kept the same except for the hydraulic conductivity of the storage aquifer (Figure 6). The vertical hydraulic conductivity is assumed to be the same as the horizontal hydraulic conductivity in all of these runs. The simulation using a value of hydraulic conductivity of 3.05 m/d has a higher recovery rate (85%) but the head at the end of injection period reaches 35.38 m. The simulation using a value of hydraulic conductivity of 30.5 m/d had a similar recovery efficiency (67.5%) as the base case and the head at the end of injection period in this case is 2.45 m.

Regional Groundwater Gradients

Regional groundwater flow gradient is another key factor in determination of the recovery efficiency of an ASR system. Horizontal groundwater flow can cause the stored water to move away from the ASR well and poor-quality native groundwater to move towards the ASR well. The base case simulation did not include a regional hydraulic gradient (i.e., gradient was zero). Simulations were performed in which regional gradients of 2 per thousand (2‰) and 5 per thousand (5‰) were applied. Gradients of these magnitudes are quite often seen in the field. Figure 7 shows the results of chloride concentration changes at the well under different regional groundwater gradients. It is clearly that the recovery efficiency is very sensitive to this parameter. For the case of 5 per thousand gradient, the model calculated recovery efficiency is zero. The chloride concentration reaches 250 mg/l at the end of the storage period. For the case with a regional gradient of two per thousand, the recovery efficiency is reduced to about 44%, significant lower than that in the base case.

CONCLUSIONS

Aquifer Storage and Recovery (ASR) has increasingly being implemented as a tool for water resource management. System performance is dependent upon site-specific hydrogeological conditions, which at some sites may not be favorable for high recovery efficiencies. Theoretical modeling can provide insights into the importance of each of the hydrogeological variables on recovery efficiency, which can be used as a screening tool for potential ASR system sites. Aquifer vertical anisotropy, heterogeneity, native water quality and regional gradients are the most significant factors for the recovery efficiency of an ASR system. Results indicate the recovery efficiency of an ASR well is very sensitive to the salinity of ambient water and regional groundwater flow gradient, in addition to the factors such as vertical anisotropy, aquifer transmissivity and heterogeneity (dispersivity).

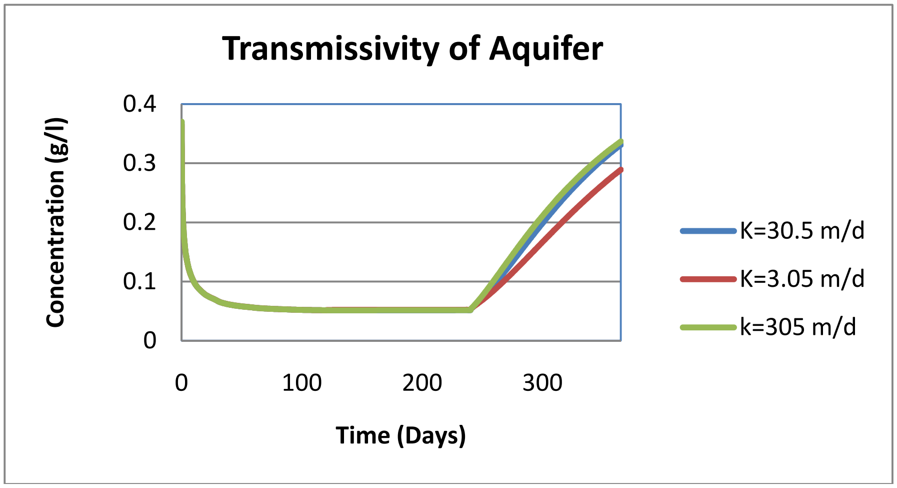

Figure 6. Chloride concentration (g/l) changes versus time (days).

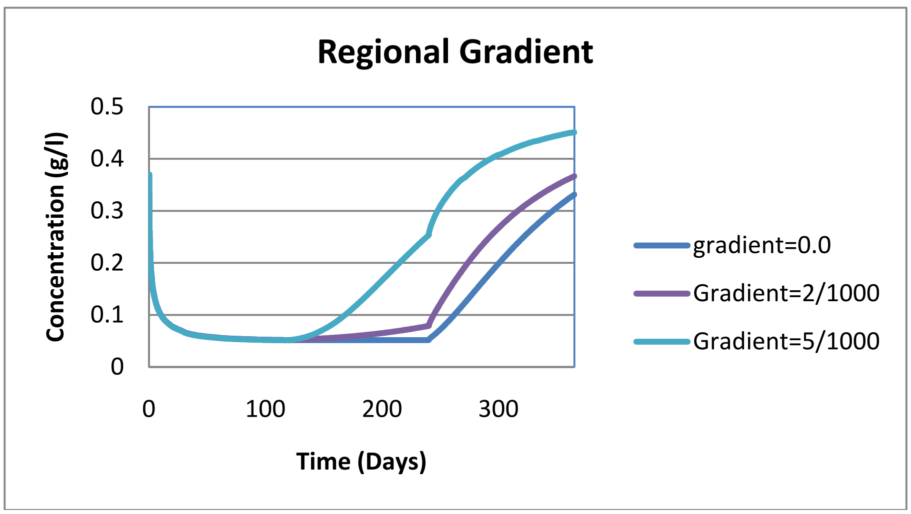

Figure 7. Chloride concentration (g/l) changes versus time (days).

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Groundwater Aquifer Recharge with Treated Wastewater in Egypt: Technical, Environmental, Economical and Regulatory Considerations

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Abstract
The economic development in Egypt and the rapid growth rate in various development sectors are dependent on the availability of water resources. Surface water is used to supply approximately 82% of the Egyptian water demand, while the groundwater is used to supply about 12% and the remaining which is about 6% is coming from the reuse of agriculture drainage water and treated wastewater. Increasingly, Egypt has turned to use the groundwater to satisfy the growing demand, at the expense of exceeding the safe yield and overexploiting the aquifer systems in some areas such as western Nile Delta and along the desert fringes in the Nile Valley. As a part from the Environmental Management of Groundwater Resources, Egypt launched a program to examine the feasibility of artificial recharge for the augmentation of groundwater supply. Through this program a detailed study has been carried out for the investigation and potential site selection for carrying out an artificial recharge experiment using treated wastewater. Through this pilot project detailed hydrogeological investigation and engineering design were carried out. Many scenarios for the aquifer storage and recovery methods have been tested to evaluate the technical environmental, economical and regulatory consideration. Results indicated that artificial recharge for groundwater aquifer using treated wastewater is promising whoever it needs more detailed study to assess the aquifer feature influences the mechanism of recharge with treated wastewater. The health risk due to changes in the physical and chemical conditions prevailing in the aquifer or due to limited adsorption capacities as well as the microorganisms survive and toxic pollutants degradation.

Key words: groundwater, groundwater management, infiltration, recharge, artificial recharge, unsaturated flow, Egypt.

1. General Background
Where soil and groundwater conditions are favorable for artificial recharge of groundwater through infiltration basins, a high degree of upgrading can be achieved by allowing partially-treated sewage effluent to infiltrate into the soil and move down to the groundwater. The unsaturated or "vadose" zone then acts as a natural filter and can remove essentially all suspended solids, biodegradable materials, bacteria, viruses, and other microorganisms. Significant reductions in nitrogen, phosphorus, and heavy metals concentrations can also be achieved.

Recharging aquifers with reclaimed water appears to be a viable option. Conventional surface disposal of wastewater can cause significant river water quality deterioration. Direct disposal of reclaimed water into rivers and drains often leads to eutrophication due to the presence of contaminants, such as nitrate and phosphate *(Horn and Goldman, 1994)*. A common indicator of eutrophication is increased phytoplankton density resulting in green turbid and foulsmelling water. After such a bloom, the algal biomass is broken down both chemically and biologically, resulting in greater chemical and biological oxygen demands (COD, BOD). Alleviating such a situation requires a reduction in water-borne nitrate, phosphate and pathogenic bacteria in the river or drain water.

Egypt is an example of hyper arid country, which has limited water resources, and less than 3 percent of the country area is cultivated. Water resources in Egypt are limited to the river water which is fixed at 55.5 billion m^3/year by agreement with Sudan in 1959, groundwater from both renewable and nonrenewable aquifer systems which is about 6.20 billion m^3/year and the reuse of both treated wastewater and agriculture drainage water which is about 5.50 billion m^3/year as shown in Figure 1 *(El Assuiti, 2003)*. The other non-conventional water resources such as desalination of seawater and brackish groundwater have been given low priority as a source of water. That is because the costs of treating seawater and brackish groundwater are high compared with other sources and its use is limit to water supply for some resorts and tourist areas. Egypt is now fully utilizing its share of the Nile River flow, shallow groundwater and is effectively reusing drainage water and treated wastewater. Continued population and economic growth will exert further pressure on the existing water resources. Major water quality problems are still limited to a number of black spots, but proper attention will have to be given to the effects of expected socio-economic developments on water quality. The threat of the available water resources being insufficient to meet the demands from all socio-economic sectors requires an adequate and integrated water resources management *(Abu Zeid, 1985)*. The present situation, with limited possibilities for extension of conventional water supply and growing demands, provides new challenges for non-conventional water resources and new planning. Within the framework of the Environmental Management of Groundwater Resources project, Egypt launched a program to examine the feasibility of aquifer storage and recovery using the surface water and treated wastewater for the augmentation of groundwater supply *(RIGW, 2002)*.

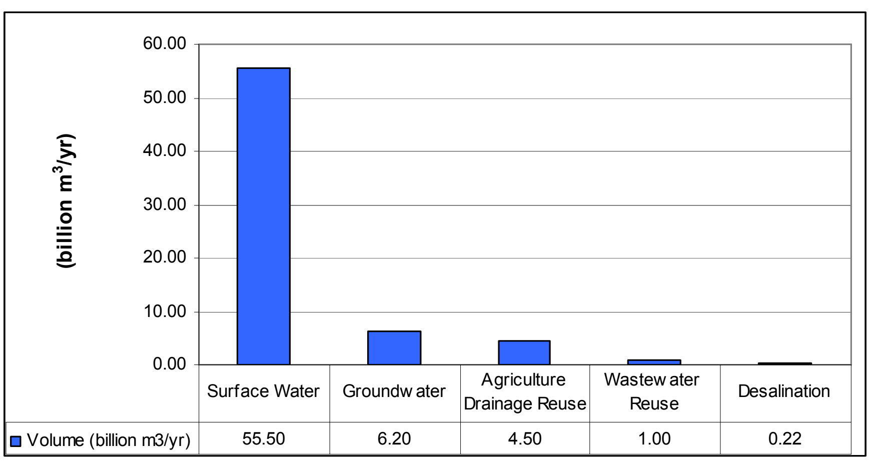

Figure 1. Water resources in Egypt.

Groundwater is considered as one of the main sources for both rural and agriculture water supplies in many areas in Egypt especially in the desert regions. Over recent years, increasing abstraction to meet rising demand for domestic supplies and expansion in reclamation in desert fringes has raised concerns for the sustainability of the groundwater resource and the livelihoods it supports. To address these concerns, considerable emphasis is being given to the augmentation of natural recharge by both traditional and modern techniques. Some of these techniques have been employed for centuries ranging from simple check bunds in gullies to complex diversion and infiltration structures as well as injection wells *(Bouwer, 1996)*. Recently there have been considerable effort and investment to maintain and restore such traditional facilities as well as investigating new technologies for aquifer storage and recovery. However there has been little systematic assessment of the effectiveness of these schemes, neither technical nor socio-economic.

2. Wastewater in Egypt

2.1 Production

Historical in Egypt reuse of sewage, after primary treatment, in agriculture has been in practice since 1911 (Gabal Al Asfar farm: 3000 feddans). Yet, experience of large scale, planned and regulated reuse project is still Limited. At presen there some large scale pilot projects (167 thousands feddans) namely East Cairo, Abu Rawash, Sadat City, Luxor, and Ismailia. In the mean time most of the sewage water drained to the agricultural drains is actually reused in one way or another. The total wastewater production from the main cities in Egypt is about 3.93 in 2009 as shown in Table 1.

Table 1: Wastewater Production at Major Cities.

City	**Treated Wastewater Production (Billion m3/y)**		
	2000	**2009**	**2017**
Cairo	1.40	1.50	1.70
Alexandria	0.60	0.63	0.75
Other areas	1.60	1.80	2.6
Total	**3.60**	**3.93**	**5.05**

2.2 Wastewater Quality

In Egypt, the domestic wastewater in the rural areas is concentrated with a COD (chemical oxygen demand) as high as 1100 mg/l, which is almost two times of that in the urban areas *(Elmitwalli et. al., 2002)*. *El-sherbiney et al. (2001)* determined the maximum aerobic biodegradability of the Egyptian domestic wastewater. They found that the minimum aerobic effluent COD concentration of rural areas was almost similar to the Egyptian effluent standards for COD, while the minimum aerobic effluent COD concentration of urban areas was significantly lower than that of the Egyptian effluent standards for COD. *Ibrahim (1995)* evaluated the applied different technologies for domestic wastewater treatment in rural areas of Egypt. He also found that the effluent of these systems did not comply with Egyptian effluent standards for COD. In this paper the effluent standards for treated domestic wastewater in Egypt and in other developed and developing countries will be compared and evaluated. Moreover, the state of the art for the criteria for effluent standards for developing countries will be discussed. Finally, applicable recommendations for the Egyptian effluent standards for the treated domestic wastewater and recommended treatment systems will be illustrated.

The effluent standards in many developing countries are copied from that of developed countries, with the assumption to achieve these standards too quickly, without considering the economical and technological capacities *(Sperling et al., 2002)*. Some standards in developing countries are excessively stringent, which leads to increase the distance between desirable and achievable, between law and reality. This result in illegal discharge of wastewater, even without any treatment, to the surface water (like illegal discharge of raw sewage to the Egyptian canals and drains. For decreasing this gap between standards and achievement in developing countries, the effluent standards should be placed to be achieved in a short period and to minimize the pollution. This can be done by implementation of standards in stepwise, in phases, to achieve the final target value for the effluent standards in the last phase. The time required for each phase should be parallel and represent the economical, institutional and technological development in the developing countries. Moreover, the effluent standards for treated domestic wastewater should be divided in classes. The effluent standards for the treated sewage in a large city, like Cairo, should not be similar to that of small communities with less than 100 capita.

Most of developed countries have the effluent standards for treated municipal wastewater in Classes. Egypt had only one effluent standard for the treated domestic wastewater.

The Egyptian effluent standards for COD are even lower than some classes in developed countries. Moreover, this value is higher than that in most of developing countries. Based on the effluent standards in many developed and developing countries. The Egyptian effluent standards for treated domestic sewage are recommended to be divided in the four classes.

- First class: for the effluent of the wastewater treatment plants in the cities. The domestic wastewater of the Egyptian cities, which represents the major part of the Egyptian domestic wastewater, is less concentrated as compared to that of villages and, therefore, it is possible to achieve a high quality effluent, if these wastewater treatment plants are operated properly.
- Second class: for the effluent from wastewater treatment plants in villages and this class can have a lower effluent quality as compared to that in class 1.
- Third class: for the effluent of on-site treatment systems for remote houses or communities, which will be installed in any area without any sanitation? This means that any new houses or communities should have pre-treatment facilities.
- Fourth class: for the existing houses in the rural areas without any sanitation.

Table 2: Recommended maximum permissible-concentration for COD, BOD5 and pathogen.

Parameter	Class 1	Class 2	Class 3	Class 4
COD (mg/L)	100	200	350	350
BOD5 (mg/L)	40	80	200	200
Pathogen (FC/100 mL)	1000	5000	10000	10000
Recommended treatment system	Pre-treatment (sedimentation or Anaerobic treatment) followed by aerobic treatment + disinfection	Pre-treatment (sedimentation or anaerobic treatment) followed by aerobic treatment + disinfection	Sedimentation tank, septic tank or anaerobic treatment + filter, wetland or pond	Sedimentation tank, septic tank or anaerobic treatment + filter, wetland or pond

These mentioned-classes have to upgraded and modified stepwise, in each phase, until achieving the targeted effluent standards. The adaptation and implementation of such classes and recommendations in this paper will result in a control and a reduction of the pollution from domestic wastewater in a short period and will reduce the illegal discharge of untreated wastewater to the canals and drains in Egypt.

Using treated wastewater in groundwater aquifer recharge can play an important role in managing these resources and enhancing the groundwater quality and quantity. Applying of such projects for recharging aquifer with treated wastewater is restricted by the regulation of the Egyptian Environmental EEAA, Authority of tourist, development and irrigation laws. Applying this system will be a solution in wastewater sanitation in the small new communities if it is environmentally safe. The project will provide Ministry of Water Resources and Irrigation with a set of guidelines in applying the obtained results of the controlled artificial recharge technique.

3. Artificial Recharge Technologies

The increasing demand for water has increased awareness towards the use of artificial recharge for augmentation of groundwater supplies all over the world. Stated simply, artificial recharge is a process by which excess surface water is directed into the ground to replenish as aquifer either by spreading on the surface in basins, by using recharge wells, or by altering natural conditions to increase infiltration *(Asano, 1985)*. It refers to the movement of water through man-made systems from the surface of the earth to underground water-bearing strata where it may be stored for future use. Artificial recharge (sometimes called planned recharge) is a way to store water underground in times of water surplus to meet demand in times of shortage.

Artificial recharge of groundwater is achieved by putting surface water in basins, furrows, ditches, or other facilities where it infiltrates into the soil and moves downward to recharge aquifers. Artificial recharge is increasingly used for short- or long-term underground storage, where it has several advantages over surface storage, and in water reuse *(Pyne, 1994)*. Artificial recharge using surface basins requires permeable surface soils. Where these are not available, trenches or shafts in the unsaturated zone can be used, or water can be directly injected into aquifers through wells. To design a system for artificial recharge of groundwater, infiltration rates of the soil must be determined and the unsaturated zone between land surface and the aquifer must be checked for adequate permeability and absence of polluted areas *(Oaksford, 1985)*. The aquifer should be sufficiently transmissive to avoid excessive buildup of groundwater mounds.

Knowledge of these conditions requires field investigations and, if no fatal flaws are detected, test basins to predict system performance. Water-quality issues must be evaluated, especially with respect to formation of clogging layers on basin bottoms or other infiltration surfaces, and to geochemical reactions in the aquifer *(Johnson and Finlayson, 1990)*. Clogging layers are managed by desilting or other pretreatment of the water and by remedial techniques in the infiltration system, such as drying, scraping, disking, ripping, or other tillage. Recharge wells should be pumped periodically to backwash clogging layers. Table 3 summaries the major characteristics for various technologies used for artificial recharge.

Recharging basins are still the most common method of recharge and provide excellent versatility for water resources planning. However, the high cost of land in some urban areas has provided the motivation for the development of vadose zone injection wells. The problem of vadose zone injection wells is that once they are clogged, they are very difficult to redevelop since there is no technique to backwash the well or to rapidly dry them. A combination of low technologies can be used to accomplish groundwater recharge with reclaimed water or other poor quality water sources.

Table 3: Major characteristics of aquifer recharge methodologies

Parameter	Recharge Basin	Vadose Zone Injection Wells	Direct Injection Wells
Aquifer Type	Unconfined	Unconfined	Confined/Unconfined
Pre-Treatment Requirements	Low Technology	Removal of Solids	High Technology
Capacity	1000-20,000 m^3/ha/day	1000-3000 m^3/well/day	2000-6000 m^3/well/day
Maintenance Requirements	Drying and Scraping	Drying and Disinfection	Disinfection and Flow Recovery
Soil-Aquifer Treatment	Vadose Zone and Saturated Zone	Vadose Zone and Saturated Zone	Saturated Zone

Artificial recharge of groundwater can be expected to increase worldwide as populations rise, demands for water increase, water resources are finite, dams for surface storage are increasingly difficult to build. Also, dams can have significant evaporation losses. Seasonal or long-term underground storage (water banking), where possible, is often preferred. Artificial recharge also plays an important role in water reuse because it gives water quality improvement (soil-aquifer treatment) and storage opportunities to absorb differences between supply and demand for reclaimed sewage effluent *(Nightingale, 1990)*. Where sewage effluent is to be used for potable purposes, recharge and recovery breaks the toilet-to-tap connection of water reuse and enables blending with natural groundwater. Combined with soil-aquifer treatment, these aspects enhance the aesthetics and public acceptance of potable water reuse. Water reuse and storage of surplus water for use in times of water shortage also must be increasingly relied upon to cope with uncertainties in future climates and their effect on surface and groundwater supplies. Design and management of artificial recharge systems involves geological, geochemical, hydrological, and engineering aspects. Since soils and underground formations are inherently heterogeneous, planning, design and construction of groundwater recharge schemes must be piecemeal, first testing for fatal flaws and general feasibility, and then proceeding with pilot and small scale systems until the complete system can be designed and constructed. Beneficiaries are water resources planners and managers, consultants, municipalities, government agencies, environmentalists, and the public at large.

4. Selection Criteria and Preliminary Investigation

To delineate the suitable areas for applying the artificial recharge technique over Egypt, a regional study for the factors affecting the feasibility of the artificial recharge has been carried out. The analysis was carried out for the entire Egypt using the Geological Information System (GIS) as well as extensive field data. A GIS based method is found to be very useful in suitability analysis for artificial recharge sites *(Saraf and Choudhury, 1998)*. For such analysis the first task was to identify the factors affecting the feasibility of the artificial recharge in Egypt. The factors affecting the feasibility of the artificial recharge in Egypt have been classified as shown in Figure 2. Based on the availability of water resources in terms of quantity and quality, physical and hydrogeological settings and a preliminary cost analysis, a set of rules has been designed to demarcate the most suitable sites for artificial recharge. The thematic information layers used in this suitability analysis and weighted indexed overlay model are: 1) Geomorphology, 2) Aquifer system and extent, 3) Soil Classification, 4) Land use, 5) Depth to groundwater table, 6) Hydraulic properties of the aquifer system and 7) Underground storage capacity

Weighted overlay analysis is a simple and straightforward method for a combined analysis of multi-class maps. The efficacy of this method lies in that human judgment can be incorporated in the analysis. A weight represents the relative importance of a parameter. Weighted index overlay method takes into consideration the relative importance of the parameters and the classes belonging to each parameter. There is no standard scale for a simple weighted overlay method. For this purpose, criteria for the analysis should be defined and each parameter should be assigned importance *(Saraf and Choudhury, 1997)*. Determination of weighting of each class is the most crucial in integrated analysis, as the output is largely dependent on the assignment of appropriate weighting. Consideration of relative importance leads to a better representation of the actual ground situation *(Choudhury, 1999)*. In weighted index overlay, the individual thematic layers and also their classes are given a specified weight as shown in Table 4 on the basis of their relative affect on the aquifer storage and recovery.

Table 4: Weighting of different parameters for the selection of artificial recharge sites

No.	Criteria	Classes	Weights
1	Water	Water is available	1
		Water is not available	0
2	Geology	Alluvial deposits (sand, gravel)	3
		Alluvial deposits (shale, clay)	1
		Sandstone	1
		Limestone	0
		Hard Rock	0
3	Geomorphology	Lower alluvial plain	3
		Flood plains and alluvial fill	3
		Upper undulating alluvial plain	3
		Gently to moderately sloping land interspersed with mounds and valleys	2
		Moderate to strongly sloping land interspersed with isolated hills	1
		Rock outcrops	1
4	Soil	Gravel	4
		Sand	3
		Silt	2
		Clay	1
5	Depth to Groundwater	0 – 5 m	1
		5 – 10 m	2
		10 – 20 m	3
		>20 m	4
6	Recharge	25 – 35 mm/day	3
		25 – 15 mm/day	2
		0 – 15 mm/day	1

Using these weighting factors, two sites have been selected for investigating the aquifer storage and recovery. Figure 2 shows the schematic diagram for the selection process for the pilot areas. The first is at El Bustan area in the western Nile Delta using the surface water and the other is at Abu Rawash area west of Cairo City using the treated wastewater as shown in Figure 3.

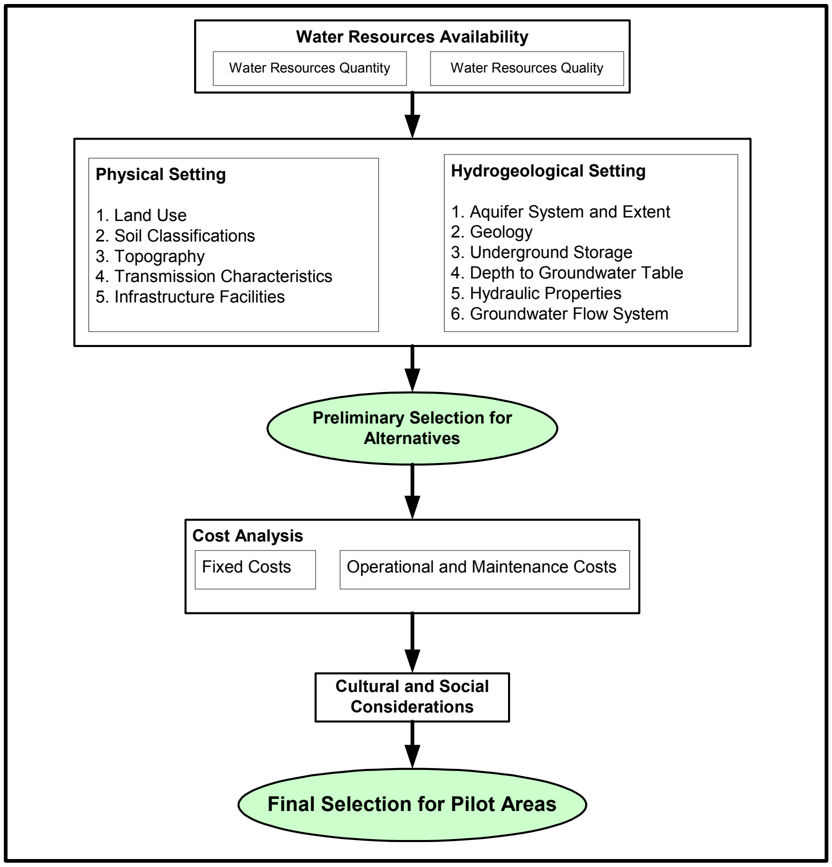

Figure 2. Schematic diagram of the selection process.

5. Abu Rawash Case Study

In Egypt, the raw wastewater from greater Cairo is collected to Abu Rawash wastewater treatment plant at the desert fringes northwest of Cairo. Then the treated wastewater is disposed to a line canal then to Al Rahawy Drain. To investigate and quantify the aquifer recharge by treated wastewater under different hydraulic load conditions using recharge basin or injection well, a square basin with length of about 50 m was constructed.

5.1 Physical and Hydrogeological Settings

The recharge site is located 30 km northwest of Cairo City and east of Cairo-Alexandria desert road. The surface is almost flat and the ground elevation is about 15.0 m (amsl). Detailed hydrogeological investigation for the site has been carried out including topographical surveying, soil classification, and geo-electrical resistivity survey. It was assessed that it is one layer aquifer system which comprises of coarse sand with fine gravel and intercalation of silt and clay thin layers which promised good prospects for the groundwater recharge. The depth to groundwater ranges from 4 m to 7 m below the ground surface. The pumping test data analysis indicating that the saturated hydraulic conductivity for the study area ranges from 15 to 30 m/day. The average effective porosity ranges from 15% to 25%.

5.2 Design of Recharge Basin

The water is pumped from the lined canal to a lined basin. In this lined basin the wastewater is allowed for fine suspended solids to settlements and also for aeration using air jets. This pretreatment process leads to decrease the chemical and biological load. The Biological Oxygen Demand (BOD) and Suspended Solids (SS) were measured before and after the pretreatment process to assess its efficiency as shown in Table 5. The efficiency of suspended solid removal is about 41% while it is about 25% for decreasing the bacteriological load. The pretreated wastewater is then pumped to a square recharge basin with a length of about 30 m and a depth of about 1.25 m from the ground surface. Also an injection well with a depth of 25 m was drilled in the center of the basin. A monitoring system has been design to monitor the effect of the recharge process on the groundwater in terms of quantity and quality. Figure 3 shows the general layout for the recharge basin in Abu Rawash.

Table 5. Measured BOD and SS for wastewater before and after the pretreatment process.

Parameter	Average Concentration (mg/l)	
	Raw Wastewater	Pretreated Wastewater
SS	2800 – 2000	1500 – 1300
BOD	400 – 300	300 – 200

5.3 Recharging Process and Results

The basin was operated under a variable hydraulic load. Variable hydraulic load means start filling the basin to certain depth and stop adding wastewater to the basin until the total volume of wastewater in the basin is fully infiltrated. The time needed for this process is called round. Also the basin was operated under constant hydraulic load keeping the water level in the basin constant at head of 0.65 m. Table 6 summaries the operation scenarios for recharging process. During the recharging process the effect of basin hydraulic load on the recharge rate as an indicator for basin clogging rate was evaluated. It was concluded that increasing the basin hydraulic load resulted in significant increase in the infiltration rate. The estimated recharge rate during the operation of the basin with constant hydraulic load ranges from 0.15 to 0.25 m/day.

The observed height of groundwater mound at the center of the basin was 0.60 m after four recharge rounds with variable hydraulic load. After the fifth recharge round with constant hydraulic load the height of the mound at the center of the basin was 1.40 m *(El Shewy, 2002) and (RIGW, 2009)*.

5.4 Identification of Optimum Recharge Cycle

To identify the optimum recharge cycle, the Accumulative Average Recharge Cycle Method was used. The method is used to calculate for each periodic measurement of recharge rate, a value representing the average total recharged volume over the entire to date recharge cycle including a required drying period using the following equation:

$$V_t = \sum_{n=1}^{t} \left[\frac{Q_n x \Delta t}{t + d} \right] \qquad (4)$$

where V_t represents the total average daily recharge at the start of draying cycle at time t, Q_n is the periodic measured recharge rate, Δt is the duration of measurement period and d is the length of drying period. The optimum drying point occurs at maximum V_t. Figure 10 shows the plot of infiltration rate and AARC with time. From this graphs it is concluded that the optimal operating cycle for enhancing the daily recharge rate and restoring the infiltration rate after drying period is 6 days wet and 7 days dry.

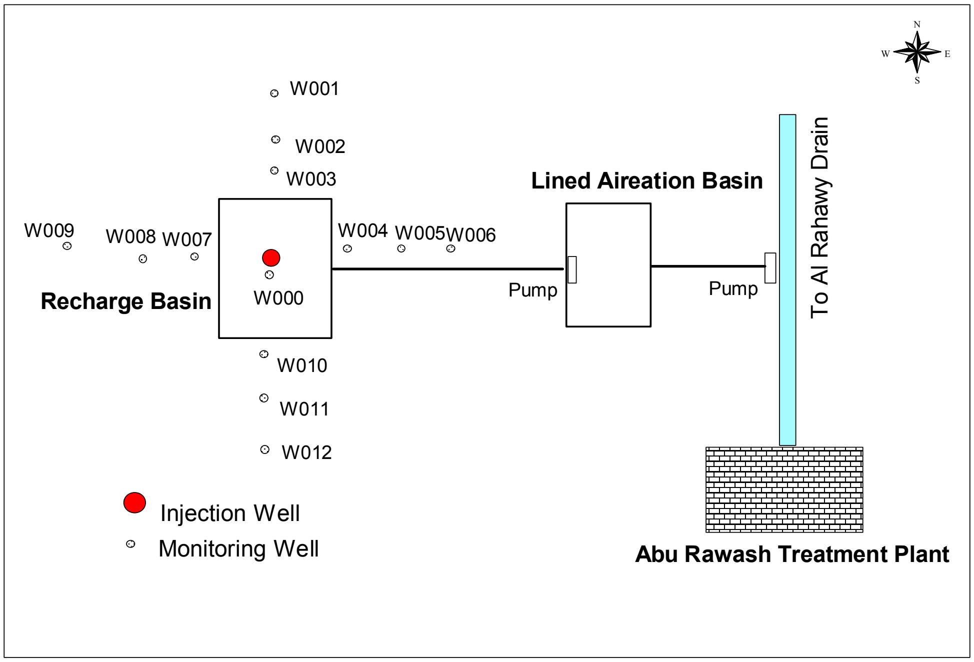

Figure 3: General layout for artificial recharge basin in Abu Rawash.

Table 6: Operation scenarios for the recharging of treated wastewater in Abu Rawash

Round	Scenario	No. Rounds	Duration (days)	
			on	off
1	Recharging the aquifer with raw wastewater under variable hydraulic head (from 0.75 m to 0 m)	2	5	7
2	Recharging the aquifer with treated wastewater under variable hydraulic load (from 0.60 m to 0 m)	2	2.7	7
3	Recharging the aquifer by treated wastewater under variable hydraulic load (from 0.75 m to 0 m)	5	8.7	9
4	Recharging the aquifer by treated wastewater under variable hydraulic load (from 0.75 m to 0 m)	5	7.3	9
5	Recharging the aquifer by treated wastewater under constant hydraulic load (head = 0.65m)	1	13.5	the end

5.5 Groundwater Quality Evaluation

To evaluate the effect of using pretreated wastewater for recharging the aquifer on the groundwater quality, the chemical and bacteriological constitutes of the wastewater and groundwater was tested. Total Nitrogen, Phosphorus, BOD and Fecal Coliform were tested as indicators for soil aquifer treatment efficiency.

The removal efficiency of total Nitrogen at the depth of 1 m in the unsaturated zone was about 10% and it was about 35% at the end of unsaturated zone. The increase of in total Nitrogen removal rate with depth and time is mainly due to the high content of soil Oxygen in the unsaturated zone. The average removal rate for Phosphorus within the experimental project was about 5% from the initial concentration of recharged wastewater. Also, at a depth of 2 m in the unsaturated zone the average bacterial removal rate was about 65% (Dawoud, 2001). Table 7 summaries the results of the quantitative assessment for the recharging process with pretreated wastewater.

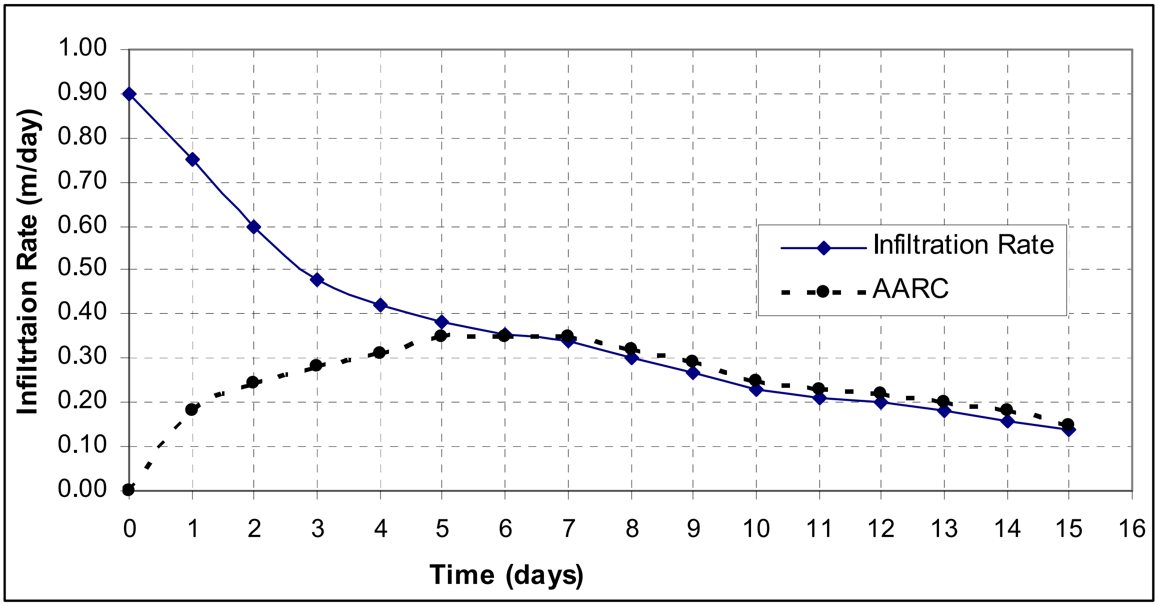

Figure 4: Calculated infiltration rate and AARC.

Table 7: Change in groundwater during the recharge process

Round		Total Nitrogen (mg/l)	PO4-P (mg/l)	Fecal Coliform (FUM/100mm)
Initial Condition		1.5	3.9	0
1	Wet	2.5	3.7	2000
	Dry	1.8	4.5	1500
2	Wet	2.7	3.7	1600
	Dry	2.3	4.2	2350
3	Wet	3.5	3.1	2100
	Dry	1.7	3.9	2500
4	Wet	3.0	2.9	2000
	Dry	1.8	3.5	2400
5	Wet	1.7	3.0	2500
	Dry	1.6	3.4	2300

6. Summery and Conclusions

Increasing demand for water in Egypt particularly in the areas suffering from shortage of surface water has shown that the extended groundwater reservoirs formed by aquifers are invaluable for water supply and storage. Natural replenishment of this vast supply of groundwater is very slow. Artificial recharge as a means to boost the natural supply of groundwater aquifers is becoming increasingly important in groundwater management particularly in non-potable water reuse.

The objective of this study is to evaluate the hydrogeological suitability of artificial recharge by pretreated wastewater in Egypt for augmentation of groundwater supply and mitigation the negative environmental impact of direct illegal reuse of raw wastewater. Results indicated that artificial recharge for groundwater is promising field needs more detailed study specially in the field of remediation of clogging problems under intermittent as well as continuous recharge. The results indicated that main factors affecting the success of recharge process are, water availability in terms of quantity and quality, method of aquifer recharge (mainly basins and injection wells), clogging, aquifer extent and boundary, aquifer hydraulic parameters, hydraulic recharge load, and economical and social issues.

Recharging the aquifer with constant hydraulic load increased the recharge rate by about 40%. In case of using the pretreated wastewater, the soil efficiency in removing the pollutants is decreasing due to the changes in its chemical balance and physical properties. It is recommended to carry out detailed study about the effect of leaching the soil by fresh water or adding some chemicals in the recharging basin to activate pollutants removal.

The retrieval of the stored groundwater should be considered quite important as the storage of water. Various scenarios for the storage recovery have been developed and tested and the results indicating that the rate of recovery will increase with time. A well designed active recovery system for the recharged pretreated wastewater can be considered as a protection system that prevent the contaminants from migration and extend. Also a good designed monitoring system is a must to evaluate the effect of recharge process on the groundwater in terms of quantity and quality.

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A History of Success: Artificial Recharge in Las Vegas, Nevada

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Success of a large scale Aquifer Storage and Recovery (ASR) project depends on appropriate goals, favorable hydrogeologic setting, careful science and dedication to the program. The Las Vegas Valley Water District (LVVWD) ASR program is the largest direct injection ASR system in the world with a total of 78 permitted dual-use production wells and dedicated artificial recharge wells. The LVVWD system is a physical-storage ASR system in the sense that injection results in a persistent rise in aquifer heads. The system also involves regulatory storage because the injection gives the LVVWD a right to later recover the injected water.

The principal aquifer was in overdraft and water levels in the aquifer system declined by more than 100 m (300 feet) in parts of the valley between 1912 and 1980. Artificial recharge was started with two wells in 1987 and over 430 x 10^6 m^3 (350,000 acre-feet) of recoverable water has been stored since program inception. The LVVWD system is unique in that it has achieved a long-term, large-scale increase in aquifer water levels, which have risen by over 30 m (100 ft) in some wells. A key feature of the LVVWD system is that it is located within a closed basin, which prevents horizontal migration, and thus loss, of stored water. Management of well clogging is the main operational challenge that is addressed through adaptive management. The LVVWD ASR system may serve as a prototype for other physical storage ASR systems in intermontane basins.

Key Words: Aquifer Storage and Recovery, Artificial Recharge, Injection, Water Resource Management

Introduction

The Las Vegas Valley Water District (LVVWD) artificial recharge system is the largest direct injection Aquifer Storage and Recovery (ASR) system in the world with a total of 78 permitted dual-use production wells and dedicated artificial recharge wells (Las Vegas Valley Water District, 2009). The system stores treated (public water supply) Colorado River water, which is injected in the winter and stored for later recovery. Operation of the ASR system allows the LVVWD to capture surface water rights that would otherwise be lost.

The artificial recharge program is operated under permits issued by the Nevada Division of Water Resources (NDWR) and the Nevada Department of Environmental Protection (NDEP). These regulatory agencies work in cooperation with the LVVWD to insure that the system is operated in a safe and responsible manner. Detailed accounting of volumes of water recharged and recovered as well as the analysis and reporting of water quality are stipulated within the permits. Annual reporting of information required by the permits is provided by the LVVWD through the Southern Nevada Water Authority (SNWA). Much of the information for this paper was taken from the 2009 Annual Artificial Recharge Report.

Additional benefits of the ASR Program include replenishment of the aquifer and maintenance of groundwater levels. ASR is accomplished by injection of water into dual-use wells (wells that are capable of both groundwater production and recharge) and dedicated artificial recharge (AR) wells (wells that are capable of recharge only). The depth of the wells is typically less than 360 m (1,200 ft) bls. The wells included in the program are located in the central and northwestern portions of the Las Vegas Valley as shown in Figure 1.

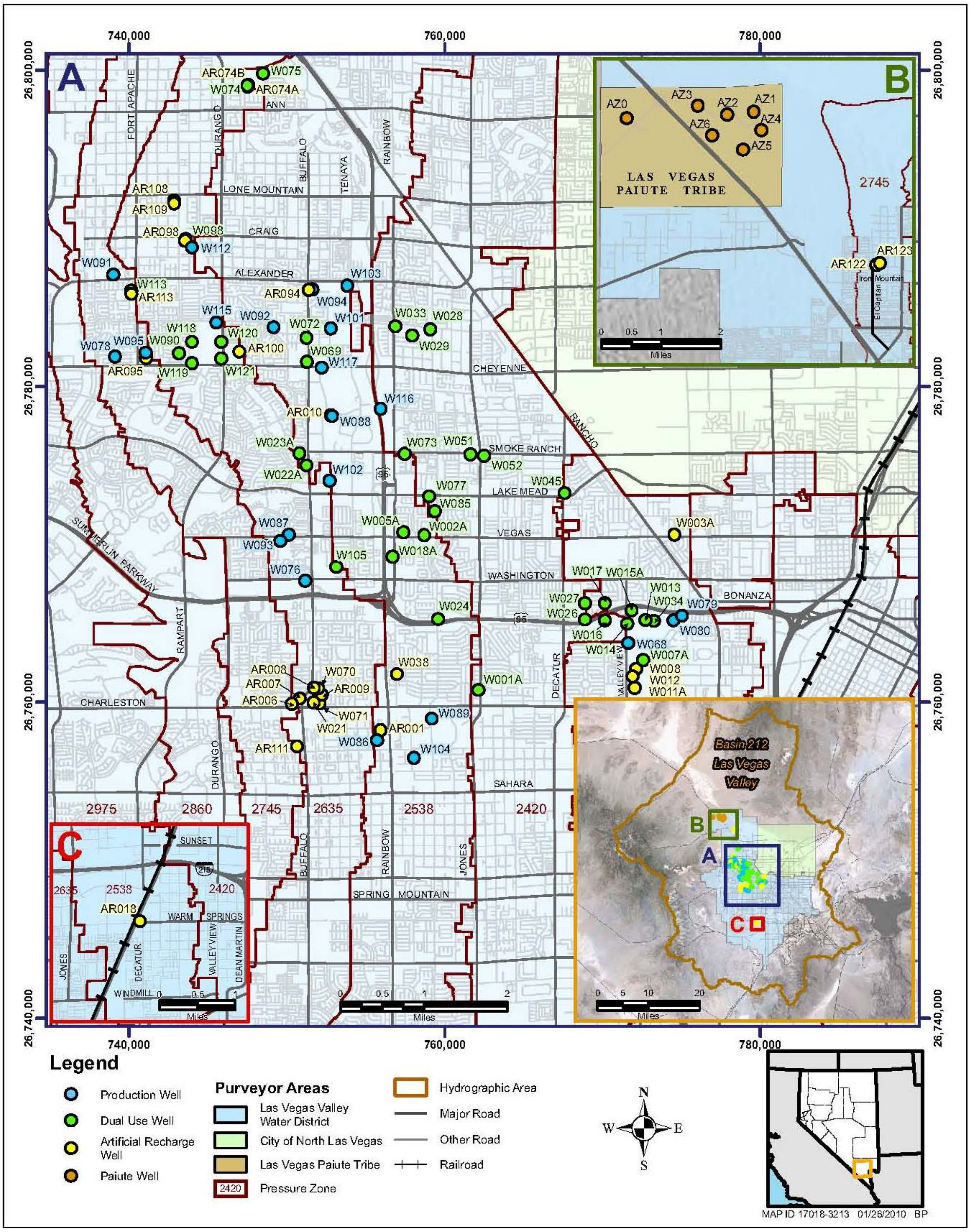

Figure 1 Location of Production, Dual Use and AR Wells, Las Vegas Valley Water District System (SNWA 2009)

Operational History

Pilot testing of the system began in 1987 with 1,987 m^3 (525,000 gallons) of treated Colorado River Water (CRW) injected into Well No. 21. In 1988 two wells were utilized for injection of 1.4 x 10^6 m^3 (1,153 acre-feet) of CRW and analysis of chemical compatibility of injectate with formation water was conducted. The program expanded by retrofitting existing production wells into dual use wells and drilling dedicated injection wells. The system had 78 permitted recharge wells in 2009. The maximum number of wells in which injection was performed at one time was 53 with a corresponding peak day injection of 389,140 m^3 (316 acre-feet) in late 2003. Figure 2 includes monthly and annual recharge totals from 1987 through 2009. The total volume of water recharged through 2009 is 430.7 x 10^6 m^3 (353,004 acre-feet).

In-lieu recharge is another option available since NDWR issued In-Lieu Recharge Order (No. 1176) in December 2004. Order No. 1176 allows LVVWD to get credit for a portion of water not pumped from their annual permitted groundwater rights of 50 x 10^6 m^3 (40,629 acre-feet). Unpumped water is considered recharged to the system and 85% is recoverable. The balance of 15% is left in the aquifer as a benefit to the system. Table 1 includes amounts of water recharged through artificial recharge and in-lieu recharge strategies as well as groundwater pumpage and recovery of recharge water.

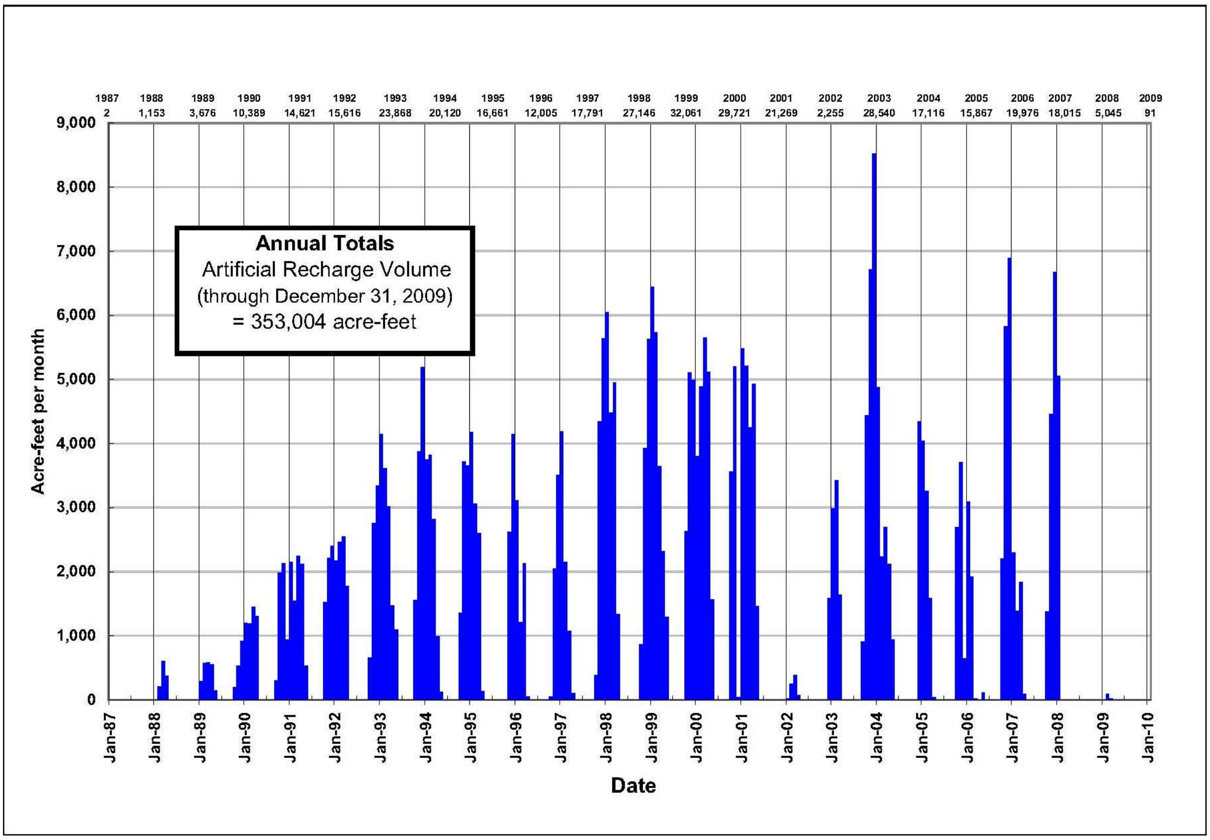

Figure 2 : Monthly Recharge Volumes and Annual Totals 1987 to 2009 (LVVWD 2009)

Availability of CRW and LVVWD customer water-use demands influenced AR operations in 2009. During the 2009 calendar year, 112,250 m^3 (91 acre-feet) of treated CRW were injected in 1 permitted LVVWD AR well. This recharge occurred during February and early March at Well AR123, which is located in the northwest portion of the Las Vegas Valley.

Recovery is conducted through LVVWD production wells and production wells owned and operated by the Las Vegas Paiute Tribe (LVPT) which are indicated in Figure 1. The LVPT wells are located on the Las Vegas Paiute Snow Mountain Reservation, northwest of Las Vegas. These production wells are permitted by the NDWR as AR recovery wells. During the 2009 calendar year, the LVVWD production wells recovered 13,569 m^3 (11 acre-feet) of recharged CRW. The Las Vegas Paiute Tribe (LVPT) production wells, per agreement with LVVWD and SNWA, recovered 757,369 m^3 (614 acre-feet) of water in 2009.

Year	RECHARGE				PRODUCTION			
	LVVWD Colorado River Water Recharge	LVVWD In-Lieu			LVVWD			LVPT Groundwater Recovery
		Recharge Recoverable	Recharge Unrecoverable	Recharge Total	Well Production	Groundwater Rights	Groundwater Recovery	
1987	2	0	0	0	37,145	39,682	0	0
1988	1,153	0	0	0	37,096	39,772	0	0
1989	3,676	0	0	0	34,025	39,890	0	0
1990	10,389	0	0	0	33,925	39,920	0	0
1991	14,621	0	0	0	36,653	40,314	0	0
1992	15,616	0	0	0	39,937	40,314	0	0
1993	23,868	0	0	0	35,647	40,314	0	0
1994	20,120	0	0	0	37,907	40,314	0	0
1995	16,661	0	0	0	42,720	40,247	2,473	0
1996	12,005	0	0	0	41,543	39,947	1,596	0
1997	17,791	0	0	0	40,316	40,152	164	0
1998	27,146	0	0	0	39,857	40,126	0	0
1999	32,061	0	0	0	39,028	40,126	0	0
2000	29,721	0	0	0	38,255	40,126	0	0
2001	21,269	0	0	0	40,620	40,126	494	1,205
2002	2,255	0	0	0	41,218	40,126	1,092	1,178
2003	28,540	0	0	0	40,127	40,126	1	985
2004	17,116	0	0	0	40,877	40,612	265	664
2005	15,867	7,621	1,345	8,966	31,661	40,626	0	572
2006	19,976	4,064	717	4,781	35,845	40,626	0	815
2007	18,015	0	0	0	40,932	40,629	303	923
2008	5,045	0	0	0	40,671	40,629	42	809
2009	91	0	0	0	40,640	40,629	11	614
Total Artificial Recharge[a]:								
Total	**353,004**	**11,685**	**2,062**	**13,747**	**886,645**		**6,441**	**7,765**
Net Recoverable AR Storage[b]: 350,483								

Notes:
All volumes in acre-feet (af), ±1 af, due to rounding.
Yearly groundwater rights reflect revisions per Las Vegas Basin Adjudication (1999).
Las Vegas Paiute Tribe's recovery of LVVWD-recharged water began in 2001.
[a]LVVWD Total Colorado River water (CRW) artificial recharge.
[b]Net Recoverable AR Storage = Total LVVWD CRW Recharged + Total LVVWD In-Lieu Recharge Recoverable - LVVWD Recovery - LVPT Recovery.

Table 1: Annual Groundwater Production, Recharge and Recovery 1987 to 2009 (LVVWD 2009)

As shown in Table 1 there has been some recovery of recharged water since about 1995 and to a greater extent since 2001 as a result of the initiation of recovery from the LVPT wells at that time. Net recoverable aquifer recharge (AR) storage is defined by the Las Vegas Valley Water District (2009) as:

Net recoverable AR storage = injected CRW + recoverable in-lieu recharge – recovery

where,

CRW = Colorado River water,
Recoverable in-lieu recharge = 85% of the non-pumped groundwater allocation.

The net recoverable AR storage through December 31, 2009, was 428 x 10^6 m^3 (350,483 acre-feet).

A cost-benefit analysis of the LVVWD artificial recharge system demonstrated that the overall benefits of the artificial recharge system are greater than the costs. In addition to water resource management considerations, operation of the system benefits all aquifer users by lower energy costs, decreasing the need to deepen wells, lessening impacts from land subsidence, and providing additional water for the aquifer system (Katzer et al., 1998; Donovan et al., 2002).

Hydrogeology

The success of the LVVWD ASR program is largely a result of exceptionally suitable local hydrogeology. The Las Vegas Valley has hydrogeologic conditions particularly favorable for a physical-storage ASR system. It is a closed basin, so stored water will not leak out of the basin to a significant degree. Las Vegas originally obtained its water supply from groundwater produced locally within the Las Vegas Valley Basin. The Las Vegas Valley Basin has an area of about 4,140 km^2 (1,600 mi^2) and is located in a desert environment that receives an average annual rainfall of only 10 cm (4 in). Local aquifer recharge is derived largely from snowmelt in the surrounding mountains.

The Las Vegas Valley Basin formed primarily by a middle Miocene extensional event and is filled with up to 5,000 ft (1,520 m) of mostly siliciclastic deposits that range in age from Miocene to Holocene. The bedrock of the basin consists of (1) Precambrian metamorphic rock, (2) Precambrian and Paleozoic carbonate rock, (3) Permian to Jurassic siliciclastic rock, and (4) Miocene igneous rock (Plume, 1989). The carbonate rock probably transmits most of the groundwater to the valley fill from the recharge areas.

The upper 300 to 360 m (1,000 to 1,200 ft) of the valley-fill deposits consist predominantly of coarse-grained (gravel and sand), fine-grained (silt and clay) and mixed siliciclastic alluvial deposits that originated from erosion of the nearby mountains (Plume, 1989). The hydrogeology of the valley-fill deposits is complex because the boundaries between the aquifers and confining strata are indistinct and difficult to map. Low quality water is locally present near the top of the valley-fill deposits. The underlying confined aquifers can be subdivided into three general zones with depth intervals of approximately 0 to 61 m (0 to 200 ft), 61 to 213 m (200 to 700 ft), and 213 to 305 m (700 to 1,000 ft) bls (Maxey and Jameson, 1948; Malmberg, 1965; Plume, 1989). The deeper two intervals are referred to as the “principal aquifer system.” The middle and lower zones are used as the storage zones for the LVVWD artificial recharge system.

History of Groundwater Resources

The first flowing artesian well in the valley was constructed in 1907 by the Vegas Artesian Water Syndicate (Wood, 2000). The number of wells in the valley subsequently increased to about 100 in 1910 then to about 9,700 wells in 1998 (Katzer et al., 1998). Groundwater withdrawals have exceeded natural recharge since the mid-1940’s, resulting overdraft conditions and declining water levels in the aquifer (Wood, 2000). Water-budget estimates indicate that as early as 1955 the total overdraft of the aquifer was just under twice that of average annual recharge (Malmberg, 1965). The aquifer was in an overdraft condition and water levels in the principal aquifer system declined by more than 100 m (300 feet) in parts of the valley between 1912 and 1980 (Wood, 2000). The declining water levels necessitated a shift to surface-water supplies. The Southern Nevada Water Authority (SNWA), which is a cooperative agency formed in 1991 to address water needs on a regional basis, now obtains only about 10% of its water from the Las Vegas Valley Basin, with the remainder coming from the Colorado River.

The use of confined storage zones provides protection of the stored water from surficial contamination and the overdrafted condition provides storage space for artificial recharge. Long-term hydrographs show increases in water levels since the start of recharge of as much as 30 m (100 ft) in some wells (LVVWD, 2009). Historically, the Las Vegas Valley was quite wet for a desert setting with springs common in several areas of the valley. Rising water levels in areas where land development occurred after aquifer water levels had declined is a concern, as historic springs may be reactivated in urbanized areas.

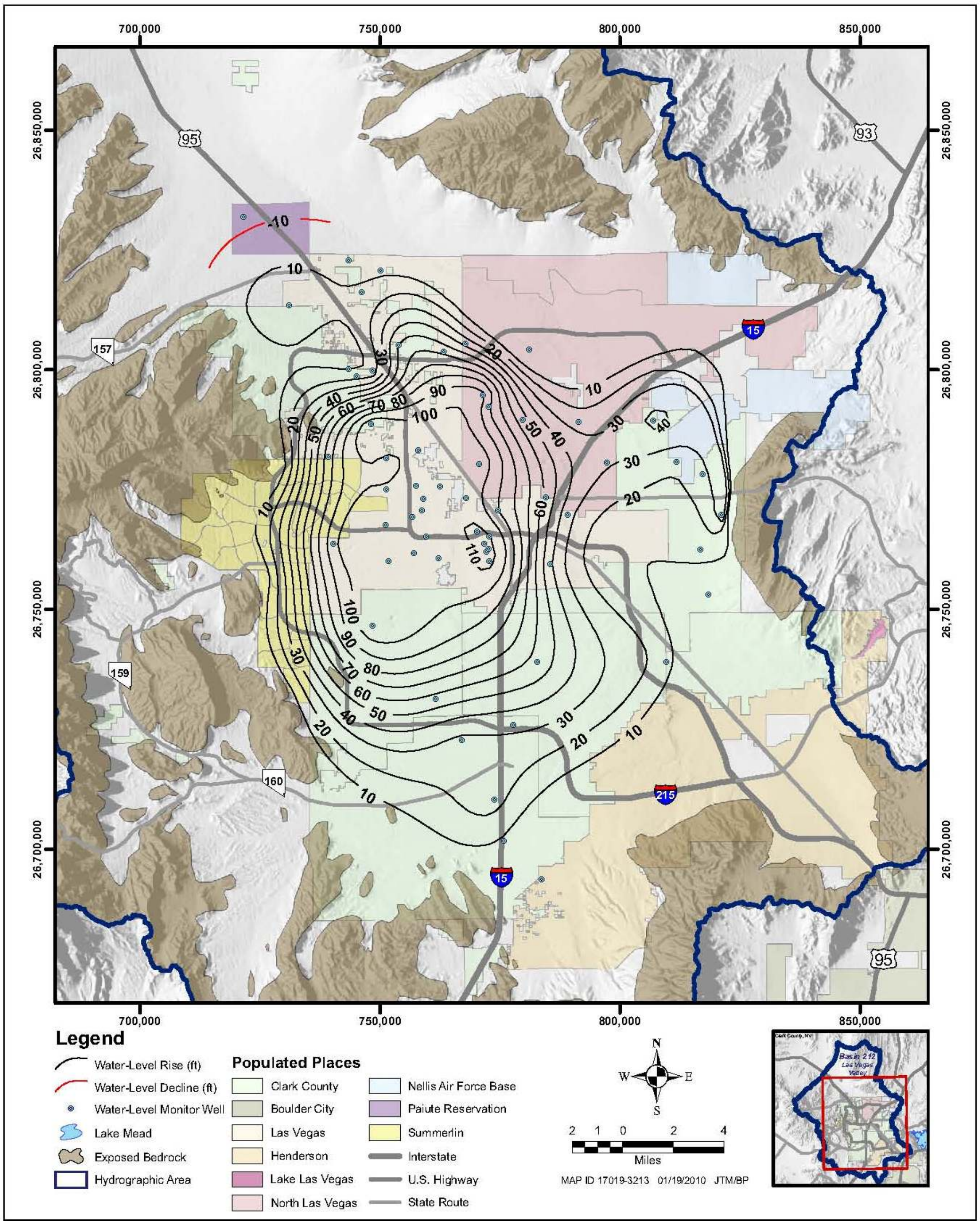

Figure 3: Change in Potentiometric Surface of the Las Vegas Principal Aquifer Fall 1990 – Fall 2009 (LVVWD 2009)

Well Construction and Operations

The LVVWD artificial recharge system employs a variety of well construction and completion types including the use of previously constructed production wells and dedicated well construction specifically for the artificial recharge system. Wells were constructed using cable tool, mud-rotary, and reverse-circulation methods. Older cable tool wells have better performance (specific capacity) and clog less quickly (Bloetscher et al., 2005), which may be due to a combination of extensive development from long-term operation, minimal drilling fluid introduction into the formation, and the impacts of the drill bit tending to open up the formation near the borehole. Most of the wells are completed using wire-wrapped screens with louvered screens used in a lesser number of wells. Both screen types are suitable for the LVVWD artificial recharge systems.

The current preferred well drilling and construction methods include the use of the reverse-circulation, air-rotary method in order to avoid the introduction of drilling mud, which can be very difficult to completely remove during development. Thorough development is also important. Louvered screens with a gravel pack are preferred for strength in deeper wells. Both vertical turbine and, less commonly, submersible pumps are used. Vertical turbine pumps have been the preferred option, with submersible pumps being used where the wells are located in residential areas, where noise is an issue. The wells equipped with vertical turbine pumps inject through the pump column with a ratchet device used to prevent backspin. Injection is also performed through the pump column in wells equipped with submersible pumps, with or without (depending on the well and pump) a downhole flow control valve. Downhole flow control valves are preferred because they allow for a better control over changing injection capacity with time and are also effective in limiting air entrainment at the start of injection.

Operational Considerations

Injection rates generally range from about 0.4 m^3/min (100 gpm) to over 14 m^3/min (3,600 gpm) depending on the wells and local hydrogeology (LVVWD, 2008). The artificial recharge wells require periodic well rehabilitation due to bacterial clogging and encrustation of the screens. The rehabilitation work typically involves brushing, bailing, super-chlorination (including chemicals designed to eradicate biological agents and dissolve encrustations), and redevelopment pumping.

The fate of disinfection byproducts in the LVVWD artificial recharge system has received considerable study. Of particular concern is the concentration of total trihalomethanes (THMs), which needs to meet drinking water standards both at the time of injection and in the recovered water after redisinfection. Experimental and laboratory studies indicate that the concentrations of THMs decrease after injection by a combination of dilution and biotransformation. The highest THM concentrations during recovery occur in the initially recovered turbid water, which is pumped to waste. The THM concentration behavior of individual wells has been monitored, and data are thus available as to when after the initiation of pumping the recovered water is suitable to be sent to distribution system. The recovered water from the production wells is sampled for THMs at the start of recovery prior to turning the water from waste to the distribution system.

Conclusions

An important factor influencing the success of the LVVWD artificial recharge system is that it has been designed to take advantage of a favorable hydrogeological setting which allows program goals to be met. The LVVWD artificial recharge system is an uncommon, successful example of an operational physical-storage ASR system because it was constructed in a closed groundwater basin and the recharge rates are sufficiently high to significantly impact basin-wide water levels (Maliva, Missimer 2010).

The LVVWD artificial recharge system also illustrates the value of experimentation and adaptive management in the development of ASR systems, especially large-scale, multiple-well systems using screen wells. Multiple-well construction, completion, and well rehabilitation options exist and should be investigated and selected based on local conditions. System performance can be improved over time with experimentation and an openness to learn from operational lessons. The LVVWD is an exceptionally well-managed system and has always taken a science-based approach to design and operation.

The LVVWD artificial recharge system demonstrates the importance of cooperation between operators and regulators in the design and evolution of ASR systems. The rational and open exchange between operator and regulator allows development and adaptability of an efficient system that meets the requirements of public health, safety and benefit. The system has developed into a valuable water resource management tool that has put into storage significant quantities of water that would have otherwise been lost. All aquifer users in the groundwater basin benefit from the artificial recharge whether or not they contribute to the recharge.

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DINA-MAR. Project for management of aquifer recharge in the context of sustainable development. Lines of action and results

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Abstract
The main objective is to determine the best horizon for MAR in Spain (new “MAR zones” with different sources of water and for different uses), taking into account over 80 criteria (the most notable being the aquifers’ storage capacity and water quality). It is also worth mentioning the environmental side of MAR from different points of view.

This article describes the main action lines of DINA-MAR i&R+D project and their most remarkable results. The main subjects are related to hydrogeology, a new GIS methodology based on map algebra operations, environmental planning, hydrology, agro-hydrology, forestry engineering, and urban hydrogeology.

On the whole, in all the lines of action and the disciplines taken on, it has been seen that the advantages of the MAR technique outweigh the inconveniences, the innovation side must receive more emphasis, and that there are still many gaps in the state of this art.

Key words
Artificial recharge, DINA-MAR, Managed Aquifer Recharge, MAR, Water management.

INTRODUCTION

This article aims to disseminate the results of the two multi-disciplinary i&R+D projects, DINA-MAR and GIAE, carried out over several years in Grupo Tragsa, both centred on the role of MAR within the management as a whole.

DINA-MAR *(Depth Investigation of New Areas for Managed Aquifer Recharge)* has been undertaken by a multidisciplinary team (14 researchers with eight different specialisms). In its four years of operation, a significant amount of light has been cast on the state of the art; especially with regards to aspects related to manage aquifer recharge within the disciplines most indirectly related to hydrology. GIAE (Spanish acronym, in English: Integral Management of Water in Building Construction) is more focused on urban hydrology.

Both are sequential and consider five blocks of objectives. The main one is to determine what zones of Spain are apt for carrying out new operations (MAR zones) with different sources of origin of the water (especially river flows and wastewater treatment plants) and for different uses (especially irrigation, storage and environmental uses), taking into account over 80 criteria; the most notable being the aquifers’ storage capacity. As well as MAR, it is worth mentioning the environmental side, the drawing up of specific indicators, the application of the MAR as a mitigating water management measure by means of recharge in forestry areas, the development of a methodology for determining the ecological flow in the feed flow watercourses, the application of techniques for specific Soil and Aquifer Treatment (SAT) in different aquifers, recharge in urban areas and new architectonic designs.

The article describes some action lines and their most remarkable results, specifically:

- In hydrogeology, the "classic" facilities for artificial recharge were studied, an inventory was made of 15 traditional schemes and eight new ones were defined.
- In GIS a methodology was drawn up that is based on map algebra operations and the adoption of environmental indicators to determine what zones of Spain are apt for MAR operations. Approximately 16% of the Spanish mainland's territory is suitable for successful operations.
- In environmental planning, a methodology has been adopted that is based on six groups of environmental criteria for the application of MAR techniques: Sources of pollution, risks, conditioning factors, demands, tendencies and advantages. The determination of the environmental volume flow in supplying rivers was studied by hydrological, hydraulic, and holistic methods, likewise habitat simulations.
- In agro-hydrology, the existing MAR facilities for irrigation are being improved by means of Soil and aquifer treatment techniques (SATs) and new designs.
- In forestry engineering, mediating and palliative techniques have been quantified in terms of how they increase aquifer recharge, highlighting the positive influence of forest masses on the basin headwaters. Civil engineering work is proposed at the headwater, which has been tested in real cases.
- As for re-utilisation S.S., MAR facilities are being incorporated into urban areas within the framework of complete management of water in building work.

OBJECTIVES

The aims of the article are oriented towards supporting a general view and to present certain conclusions about the different lines of action, bringing together different points of view and a focus on researchers' hydric management. Therefore, the traditional scheme of scientific publications is somewhat modified and presents the reader with a line of action to the documentation generated by the research team, given the difficulty of synthesising several years of activity in one article.

METHODS

Given the nature of the article, information and unification from the different disciplines, it is complicated to define the methods and materials used in each line of activity, although they are all generally accepted.

The "MAR zones" have been defined by algebraic operations on maps and thematic attributes, using a GIS (Dinamap) application that is sufficiently robust to treat up to 83 layers and thematic attributes throughout Spain.

RESULTS AND DISCUSSION

Inventory of existing generic devices and proposal of other "new" devices

The starting point was an inventory of devices available at a global level to create a catalogue of practical experiences. These were grouped according to the Gale, 2005 classification. To these 15 classifications, eight more were added and defined (table 1), which are generally based on variations of irrigation systems that increase return to aquifers and the implementation of MAR techniques in urban zones.

SYSTEM	TYPE OF DEVICE
DISPERSION	INFILTRATION PONDS
	INFILTRATION CHANNELS
	SOIL/AQUIFER TREATMENT TECHNIQUES
	INFILTRATION FIELDS
	RECHARGING BY IRRIGATION CHANNELS
CHANNELS	RETAINING DYKES AND RESERVOIRS
	PERMEABLE DYKES
	DIVERSIONS

	BED SCARIFICATION
	SUB-SURFACE/SUBTERRANEAN DYKES
	PERFORATED DYKES
WELL	QANATS (SUBTERRANEAN GALLERIES)
	OPEN INFILTRATION WELLS
	DEEP WELLS AND MINI-PROBES
	PROBES
	DOLINES, COLLAPSES, etc.
	ASR/ASTR
FILTRATION	FILTRATION BANKS IN RIVERBEDS (RBF)
	INTER-DUNE FILTRATION
	SUBTERRANEAN IRRIGATION***
RAIN	UNPRODUCTIVE RAINWATER CAPTURE
SUDS	ACCIDENTAL CONDUCTION AND SEWERAGE RECHARGE
	SUSTAINABLE URBAN DRAINAGE SYSTEMS

Table 1. MAR schemes inventory grouped by typologies (modified from Gale, 2005).

Study to determine "MAR zones" in Spain and attribution of the most ideal devices

A complete process based on GIS has been undertaken to determine susceptible areas in Spain for the application of managed aquifer recharge techniques, which have been designated "MAR zones", with fluvial and purifying plant origins.

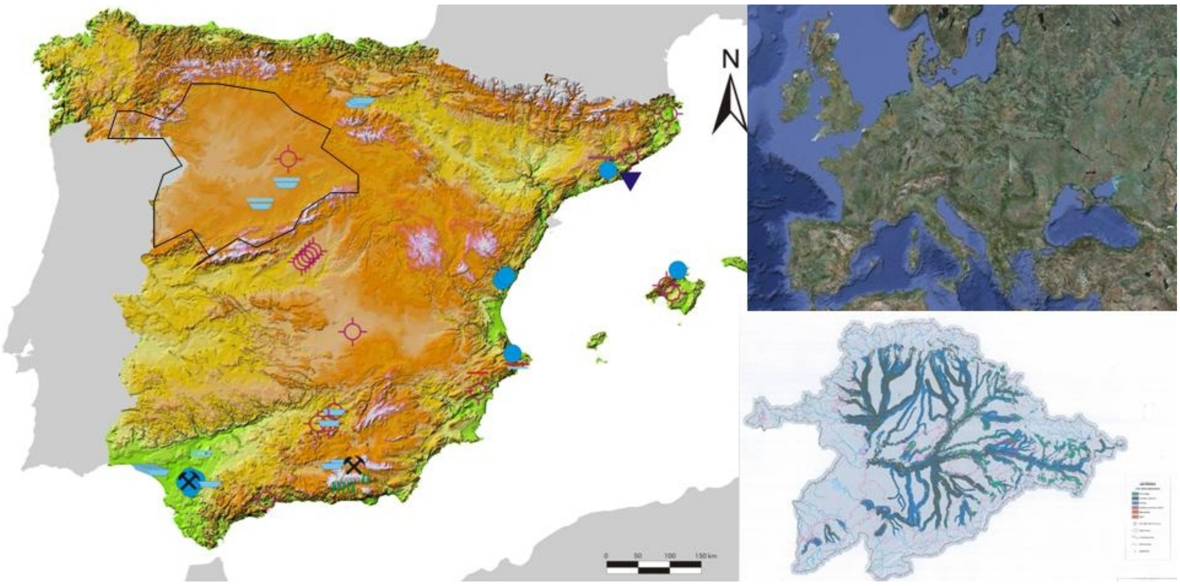
The process has been repetitive, testing different algebraic map options on reductive maps with up to 83 layers and GIS coverages. Permeable outcrop layers, lithology, aquifers, water level, fluvial riverbeds, purifying plants, data collection stations with superfluous measurements, inclines, altitude, distance to the coast, etc. must all be taken into consideration. The main R&D component is based on studying the deductive sequence leading to similar results in existing inventories. The "MAR zones" in Spain have been defined after several trials. Their grouping by hydrographic basins appears in table 2.

ID	Major Basin	MAR zones areas within basin (km^2)	Total basin areas (km^2)	% MAR zones/Basin	% total
1	NORTE	1952.98	53780.90	3.63	2.92
2	DUERO	21565.45	78955.69	27.31	32.26
3	TAJO	10186.19	55814.90	18.25	15.24
4	GUADIANA	5183.57	60125.19	8.62	7.75
5	GUADALQUIVIR	4878.02	63298.10	7.71	7.3
6	SUR	1457.55	18408.22	7.92	2.18
7	SEGURA	2282.97	18833.04	12.12	3.41
8	JUCAR	7891.79	42682.26	18.49	11.8
9	EBRO	8686.32	85936.39	10.11	12.99
10	PIRINEO	1746	16555.28	10.55	2.61
11	BALEARES	1023.07	5038.33	20.31	1.53
	TOTAL	**66853.9**	**499428.31**	**13.39**	**100**

Table 2. Results relating to "MAR zones" by hydrographic major basins. Columns: Basin surface and the MAR zone contained in it and the percentage represented with respect to each basin and the total.

Approximately 16% (67,000km^2) of the Spanish peninsular and Balearic Islands territory is suitable for recharge management. The most ideal basins are Duero and Balearics and the least ideal are those in the north and the Guadalquivir.

To facilitate identification of the MAR zones, 11 chloropeth maps by hydrographic basins have been created. An example of the results for the most ideal basin is shown in figure 1. The entire cartography is available at www.dina-mar.es.



Figures 1. Location map of the MAR sites and distribution of “MAR zones” in Duero basin.

Search criteria to associate devices with each “MAR zone”

With the physical elements well defined and knowing the specifications of the 23 inventoried AR devices, a grades/weights system has been designed and automated in such a way that each device receives a weight according to its suitability and is adjusted to the physical characteristics and the other indicators with GIS support.

The main association criteria considered, supported in layers and thematic coverage, are based on a grades/weights system.

The grades established are the distribution of permeabilities, lithologies, nitrate contaminations, irrigable areas and irrigation origin, proximity to forests, purifying plants (with their treatment types), dams (with their associated capacities), wetlands, rivers (with their average associated flows), to the coast and major aqueducts; incline, height, flood risk, water level, water quality, meteorological stations with surplus hydrics and mainly urban areas. The weights range between zero (inadequate) and three (very favourable).

Creating a relational structure between physical factors and indicators with GIS support and MAR devices, an association matrix that supplies the *Hidrogeoportal DINA-MAR* (table 3) has been designed and automated. The result is a large scale cartography ranking the most to the least recommended devices (figure 2).

Potential for the MAR technique in Spain

Based on the premise defended by DINA-MAR that the future on the matter of water depends on the capacity to store it, a calculation has been made of the storage potential in currently unsaturated Spanish aquifers against the storage capacity of dams.

Based on the storage in dams in Spain in January 2005, which reached 533,193 m^3, and the definition of the MAR zones, a calculation has been made with GIS support based on the water level depth, aquifer permeability and storage coefficients. The result is that Spanish subsoil (excluding the Canary Islands) has a space of approximately 2, 0 hm^3/km^2 in the MAR zones. In other words, the stored volume in the dams could be stored in aquifers in 260%, safeguarding quality with full viability, also enabling surface occupation of the land.

TECNICAS Y DISPOSITIVOS DE MAR				1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
				DISPERSIÓN					CANALES						POZOS							FILTRACIÓN			LLUVIA	SUDS	
ZONAS MAR	CASUÍSTICAS 1	CÓDIGO	PESO	BALSAS DE INFILTRACIÓN / HUMEDALES	CANALES DE INFILTRACIÓN	CABALLONES/TÉCNICAS DE TRATAMIENTO SUELO-ACUÍFERO	CAMPOS DE INFILTRACIÓN (INUNDACIÓN Y DIFUSIÓN CONTROLADA)	RECARGA ACCIDENTAL POR RETORNOS DE RIEGO	DIQUES DE RETENCIÓN Y REPRESAS	DIQUES PERMEABLES	SERPENTEOS, LEVEES	ESCARIFICACIÓN LECHO	DIQUES SUBSUPERFICIALES/SUBTERRÁNEOS	DIQUES PERFORADOS	QANATS (GALERÍAS SUBTERRÁNEAS)	POZOS ABIERTOS DE INFILTRACIÓN	POZOS PROFUNDOS Y MINISONDEOS	SONDEOS	DOLINAS, COLAPSOS...	ASR	ASTR	BANCOS FILTRANTES EN LECHOS DE RÍOS (RBF)	FILTRACIÓN INTERDUNAR	RIEGO SUBTERRÁNEO	CAPTACIÓN DE AGUA DE LLUVIA EN IMPRODUCTIVO	RECARGA ACCIDENTAL CONDUCCIONES Y ALCANTARILLADO	SISTEMAS URBANOS DE DRENAJE SOSTENIBLE
Afloramientos permeables MMA 2006		MUY ALTA	3	3	3	2	3	2	3	2	2	3	1	2	1	3	3	1	3	1	1	1	3	1	1	1	1
		ALTA	2	3	3	1	3	2	2	2	3	2	1	2	1	3	3	1	1	1	1	1	3	1	1	1	1
		MEDIA	1	2	2	1	1	2	2	2	1	1	1	2	1	2	2	1	1	3	3	1	2	1	1	1	1
Geología de España a escala 1:200.000, MMA 2006		ALUVIAL	1	3	3	2	3	1	3	3	3	3	3	3	1	3	3	3	0	3	3	3	0	1	1	1	1
		DETRÍTICO	2	3	3	2	1	1	2	2	1	0	3	3	3	2	2	3	0	3	3	0	3	1	1	1	1
		KARSTICO	3	2	2	2	2	1	3	3	2	0	1	2	3	3	3	2	3	2	2	0	0	1	1	1	1
		METAMÓRFICO	4	0.2	2	2	0.5	1	2	2	0	0	1	3	1	1	1	1	0	1	1	0	0	1	1	1	1
		VOLCÁNICO	5	0.2	2	2	0.5	1	3	3	0	0	2	2	3	1	1	1	0	1	1	0	0	1	1	1	1
		INTRUSIVO	6	0.2	2	2	0.5	1	2	2	0	0	2	2	1	2	1	1	0	1	1	0	0	1	1	1	1
		EVAPORÍTICO	7	0	0	0	0	0	0.5	0	0	0	1	0.5	0.5	2	1	1	0	1	1	0	0	0	1	1	1
Red de control de nitratos en las aguas subterráneas	CONTENIDO EN NITRATOS	<50	1	1	1	1	1	3	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
		>=50	2	1	1	1	2	2	1	1	1	1	1	1	1	1	1	1	0	0	0	1	1	1	1	1	1
Zonas declaradas vulnerables 2005	1:ZONAS VULNERABLES 0:ZONAS NO VULNERABLES		1	1	1	1	2	2	1	1	1	1	1	1	1	1	1	1	0.5	0	0	1	1	1	1	1	1
Origen del agua		SUPERFICIALES	4	3	3	3	3	3	3	3	3	3	1	3	1	3	3	1	3	1	3	1	1	2	1	1	1
		SUBTERRÁNEAS	2	0.5	0.5	1	0.5	1	0.5	0.5	0	0	1	1	0.5	0.5	0.5	0	0	1.5	1.5	1	0	2	0	0	0
		RETORNOS	4	0	2.5	1	2	3	1	1	0	0	1	1	0	1	1	0.5	0	0	0	0	0	1	0	0	0
		DEPURADORAS	6	1	1	1	1	1	1	1	0	1	2	1	0	1	2	1	0.5	1	1	0	1	2	0	0	1
		DESALINIZADORAS	4	1.5	1	0	1	0	0	0	0	0	1.5	0	0	1.5	2	2	0.5	1.5	1	0	1	2	0	0	0
Áreas distantes hasta 2 km desde embalse	1:ZONA 2 km EMBALSES 0:ZONA A DISTANCIA SUPERIOR	<=2 km	1	2	2	2	0	1	2	0	2	2	1	1	1	1	1	3	3	3	3	3	0	2	0	0	0.5
Polígonos concéntricos distantes de 1 a 5 km de los ríos con caudal medio	0 - 0,45	<=1 km	3	3	1	3	3	1	2	0	3	0	1	2	1	1	1	3	2	3	3	3	0.5	1	0	0	0
	> 0,45 - 1,65	>1 y <=2	3	1	2	1	1	1	1	0.5	0	0	1	0	2	1	1	2.5	1	2.5	3	0	1	1	0	0	0
	> 1,65 - 7,26	>2 y <=3	4	1	2	1	0.5	1	1	0	0	0	1	0	2	2	2	1	1	2	3	0	0	1	0	0	0
	> 7,26 - 27,5	>3 y <=4	5	1	2	0	0.5	1	1	0	0	0	0.5	0	2	2	2	1	1	1	2	0	0	1	0	0	0
	> 27,5	>4 y <=5	6	1	3	0	0.5	1	0.5	0	0	0	0.5	0	2	3	3	3	1	1	1	0	0	1	0	0	0
Riesgo de inundación		Sin riesgo	4	3	2	1	0	3	0	0	0	0	1	0	1	3	3	3	1	3	3	0	1	1	1	1	0.5
		Máximo	1	0	0	2	3	0	3	1.5	3	3	1	1.5	0	0	0	0	1	0	0	2	1	0	0	0	1
		Medio	2	0.5	0.5	2	3	0.5	2	2	1	0	1	3	0.5	0.5	0	0	1	0	0	1	1	0.5	0	0	1
		Mínimo	3	2	0.5	1	2	1	0.5	0.5	0	0	1	1.5	1	3	2.5	1	1	1	0.5	0	1	1	1	1	1
Representación de áreas que se encuentran dentro de un intervalo de pendiente preestablecido		0-10	2	3	3	1	3	1	1	0.5	3	3	3	1	1	3	3	3	1	3	3	3	1	1	1	1	1
		10-20	2	2	2	1	0	1	2	1	0.5	0.5	2	1	1	3	2	1	1	2	2	0.5	0	1	1	1	1
		20-30	3	1	1	2	0	0.5	2	2	0	0	1	3	2	2	1	1	1	1	1	0	0	1	1	1	1
		30-40	4	0	0	2	0	0	3	2	0	0	0	2	2	0.5	1	0.5	1	0.5	0.5	0	0	0	0	0	0
		40-50	5	0	0	2	0	0	3	0	0	0	0	2	1	0	0	0	0	0	0	0	0	0	0	0	0
Áreas distantes hasta 1 km de los límites de los humedales	1:ZONAS INFLUENCIA DE HUMEDALES / 0:ZONAS NO INFLUENCIA	<=1 km	0.5	1	2	0	3	1	2	2	0	1	2	1	1	2	3	1	1	0	0	1	1	1.6	0	0	0
Áreas distantes hasta 1 km del Acueducto Tajo-Segura	1:ZONAS INFLUENCIA TAJO-SEGURA / 0:ZONAS NO INFLUENCIA	<=1 km	2	2	2	1	1	1	0	0	2	1	0	0	1	2	2	2	2	2	2	0	0	1	0	0	0
Calidad de las aguas. Conductividad >2500	1: ZONAS CONDUCT. < 2500	< 2500	1	2	2	2	2	2	2	2	2	2	3	2	2	2	2	2	2	2	2	2	3	2	2	1	1
	2: ZONAS CONDUCT. >2500	> 2500	1	0	0	1	0	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	2	0	0	0	0
Minas en acuíferos. Buffer 2 km	1:ZONAS INFLUENCIA MINAS / 0:ZONAS NO INFLUENCIA	<=2 km	2	2	1.5	1	1	0	1	0	0	0	1	0.5	3	3	1	0.5	0	1	2	0	0	0.5	0	0	0
Usos de suelo, obtenida del CORINE Land Cover	FORESTAL		1	0	2	3	0	0	3	3	0	0	3	3	2	1	0	0	3	0	0	0	1	0	0	0	0
	SUBDESERTICO		1	0	0	0	1	0	1	0	2	0	3	3	3	2	2	0	1	1	1	0	0	0	0	0	0
	PRADOS Y PASTOS		4	1	2	2	2.5	2	0.5	0.5	1	2	0.5	0	0	2	2	1	2	0	0	0	0	1	2	0	0
Peso basado en la naturalidad (se da preferencia a las actuaciones en áreas menos sensibles y por tanto más humanizadas)	AGRARIO		4	3	3	2	3	3	1	2	2	3	1	3	0	2.5	2.5	3	0	3	3	1	0	3	2	0	0
	SIN VEGETACIÓN		2	1	0	0	0	0	1	1	1	0	2	1	1.5	1.5	1.5	1	3	1	1	0	3	0	0	0	0
	GLACIARES Y NIEVES PERMANTES		1	0	0	3	0	0	3	3	0	0	1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	HUMEDALES		3	2	2	0	3	1	2	2	2	2.5	1	0	0	0	0	0	0	0	0	2	1	0	0	0	0
	INFRAESTRUCT. HIDRAÚLICAS		4	3	3	0	0	0	2	2	1	0	3	3	1	0	0	0	0	0	0	0	0	0	0	0	0
	INFRAESTRUCT. TRANSPORTE		5	0	0	0	0	0	0	1.5	0	0	1.5	0	0	0	0	0	0	0	0	0	0	0	3	2	1
	URBANO		5	2	1	0	0.5	1	0	0	0	0	0.5	0	0	3	3	1.5	0	2	2	3	0	2	3	3	3
	INDUSTRIAL		5	0	0	0	0	0	0	0	0	0	0	0	0	2	2	3	0	2	2	2.5	0	0	3	2	2
Buffer 1 o 5 km Áreas urbanas	1 km	nº habitantes <20.000	1.5	1	1	1	1	1	1	1	1	1	1	1	1	2	2	1	1	1	1	1	1	1	3	2	2
	5 km	nº habitantes >=20000	2	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	3	3	1	1	1	3	3	3
Piezometría superficial 2008	ISOPIEZAS MORADAS	<25	1	3	3	1	3	3	3	1	3	3	3	1	3	3	1	0	1	0	0	3	1	3	1	1	3
		>25 y <=50	2	2	2	1	1	2	0.5	1	1	2	3	0.5	2	2	3	1	1	0.5	0.5	0.5	1	2	1	1	2
		>50 y <=150	3	0.5	0.5	0	0	0.5	0	0	0	0	0	0	0	0	2	2.5	0.5	3	2	0	0	0	1	1	2
		>150	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	0.5	3	3	0	0	0	0	0	2
Piezometría profunda 2008	ISOPIEZAS ROSAS	p>200 m	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	0	3	3	0	0	0	0	0	0
Masas Forestales a escala 1:50.000	BOSQUES		3	1	1	2	1	1	3	3	1	1	3	1	1	1	1	1	1	1	1	1	1	1	1	1	3
Unidades hidrogeológicas susceptibles de recarga según el IGME			3	2	2	1	1	1	2	2	1	1	2	1	1	2	1	1	1	1	1	2	1	1	1	1	1
Áreas que distan hasta 1 km de las depuradoras	BUFFER DE 1 KM Y DATOS DE HABITANTES EQUIVALENTES	<20.000	1	1	1	1	1	1	2	1	1	1	1	1	1	3	1	1	3	1	1	1	1	1	1	1	1
		>=20000 y <200.000	2	1	1	1	1	1	2	1	1	1	1	1	1	1	3	3	1	2	2	2	1	1	1	1	2
		>=200.000	3	2	2	1	1	1	2	1	1	1	1	1	1	1	1	2	1	3	3	2	1	1	1	1	3
Depuradoras por lagunaje	BUFFER DE 1 KM		2	3	1	1	2	1	1	1	1	1	1	1	1	3	1	1	1	1	1	1	1	1	1	1	1
Áreas que distan hasta 5 km de la red de control de la intrusión marina	BUFFER DE 5 KM PTOS INTRUSIÓN	5 km	3	0	0	0	0	0	0	0	0	0	0	0	0	0	2	3	0	3	3	0	0	0	0	0	1
Rango altitudinal		> 0 y <20	1	2	0	2	2	2	0	1	2	1	1	1	1	2	2	2	1	2	2	1	2	1	1	1	1
		>20 y <1500	2	2	2	2	2	2	2	2	2	1	1	1	1	2	2	2	1	2	2	1	1	1	1	1	1
		>1500	1	0.5	1	0.5	0.5	0	3	2	0	0	0	0	0.5	1	0	0	1	0	0	1	0	1	1	1	1
Zonas áridas	PRECIPITACIÓN	>400 mm	3	2	1	0	2	2	2	1	2	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
		<=400 mm	1	1	0	0	0.5	0.5	3	0	0	0	3	2	3	1	1	1	1	1	1	0.6	3	2	2	1	1
Est. meteo en zonas con excedentes hídricos	SUCUENCAS EXCEDENTE HÍDRICO		2	2	1	1	2	2	2	2	2	1	1	2	1	3	1	1	2	1	1	2	2	1	2	2	3
Distancia a la costa		< 2 km	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	3	1	1	1	1
		> 2 y < 5 km	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	0	1	1	1	1
		>=5 km	1	1	1	1	1	1	1	1	1	1	1	1	2	2	1	1	1	1	1	3	1	1	1	1	1
Sistemas dunares en capa geología (Corine)			1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	3	1	1	1	0

Table 3. Aspect of the table that relates physical factors and indicators (based on GIS support) with the different MAR devices.

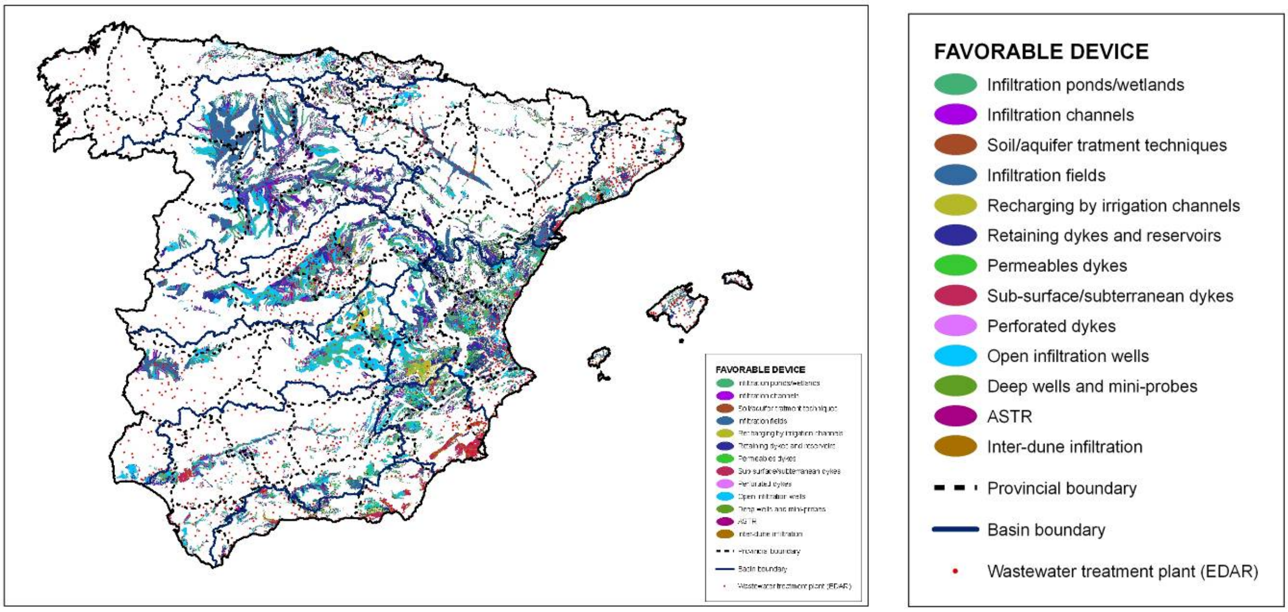


Figure 2. Provisional cartography with attribution of the most ideal MAR devices for each “MAR zone”.

This system has enabled some highly ideal MAR zones to be identified. For example, the Bajo Guadalhorce aquifer (Malaga), considering water coming from the river and a waste water treatment plant, up to 11 MAR devices are concentrated in this area (figure 3).

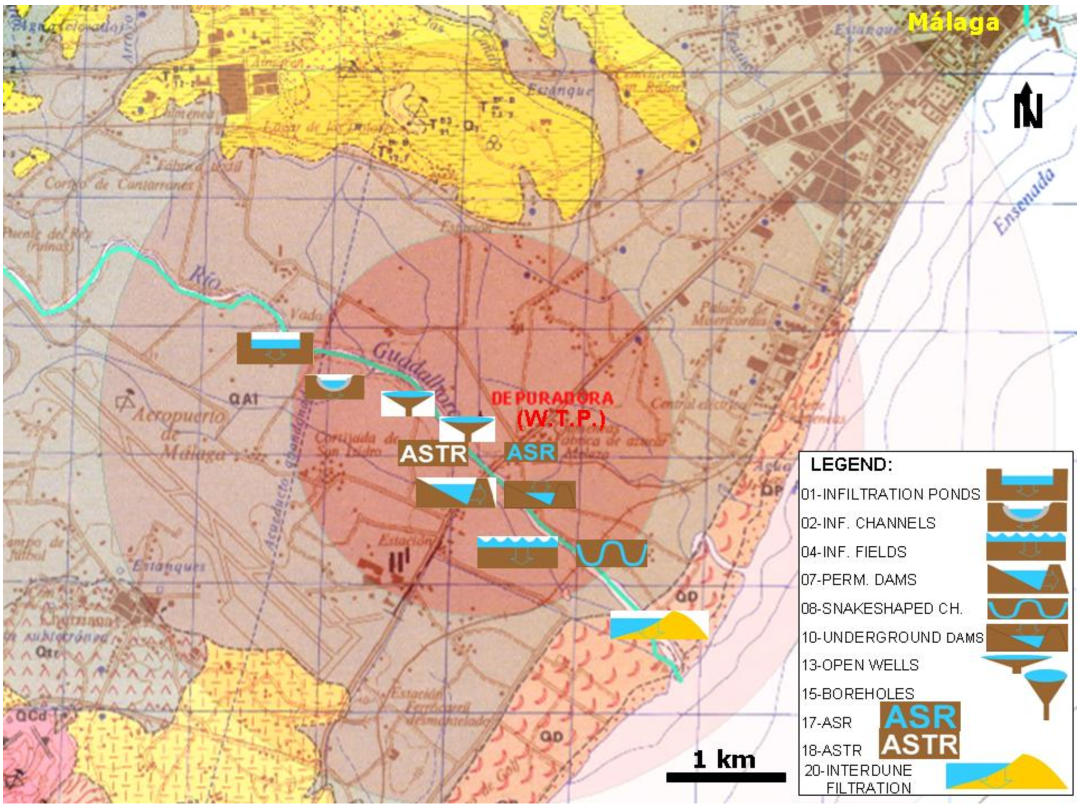


Figure 3. Example for the Bajo Guadalhorce aquifer. Proposal for the location of MAR devices, obtained with the grades/weights system designated “DINA-MAR-hidrogeoportal”.

Environmental aspects

Methodology to determine environmental flows in intake basins

The intake flows from fluvial basins and purifying plants must take environmental flows into consideration, even though a large amount of the water derived from artificial recharge (hereinafter, AR) forms part of the environmental flow, as it is retained in space and time. A methodology has been designed based on the climatic, seasonal, spatial and subsoil considerations in each basin. The main aspects to be considered in each individual study are:

- The river sections must be classified, as well as deciding what classifications are to be considered as priorities.
- The level of detail of the study must be precisely defined in addition to the main species in the different types of riverbeds.
- It must be decided whether specific sections or basins as a whole are to be evaluated.
- In all the areas to be evaluated, it is necessary to know all the water extraction concessions from the riverbeds, dams, and mini-centres.
- Flows must be determined jointly, and in addition to the criteria mentioned, geomorphological, riparian, water quality, wild fauna of the fluvial and/or littoral ecosystem (if applicable), quantity and quality of subterranean waters, landscape values, public use, etc. objectives.

Environmental planning

A methodology has been adopted in environmental planning based on six groups of basic environmental criteria in order to question the application of MAR techniques: Sources of contamination, risks, conditioning factors, demand, trends and advantages. With these, PER type environmental indicators have been created (Friends & Raport, 1979), also applying a system of grades/weights. How these criteria have been designed:

- Contamination sources: Specific established uses cause specific contamination risks, taking the dispersion method (diffuse is the most difficult to control but the often the most serious in its immediate effects) and its origin into account: Each one implies the existence of risks to the quality of the water to be recharged. Nitrates, other synthesis chemicals or solids washed up require different treatments before using input from an agro-farming, industrial or natural run-off origin for recharging. Urban, rural, farming, industrial and suspended solids contamination has been differentiated.

- Risks: The localisation of specific risk terrain that may endanger the viability of recharging or the requirement for this action. The interception of flows (aquifers and run-offs), accidental spills, the presence of biological endemisms, saline or marine intrusions and the effects on health are proposed.

- Conditioning factors: There are characteristics inherent to the use or to association that require one or another types of MAR devices or that simply determine that the recharge does not surpass certain limitations, such as high inclines, high run-off, high elevation, free nappe of continuous or temporary water, high phreatic and the existence of dry periods.

- Requirements: The requirements vary not only in terms of the quality required but also in spatial and seasonal distribution. Drinking, recreational (swimming), ecological, refrigeration and irrigation water and hydroelectric energy are proposed.

- Trends: With the aim of viability and profitability of the devices, it is essential to evaluate both current uses and anticipated trends in these uses. Intensification, sensitivity to climate change, potential irrigation demand and preferential restoration areas are proposed.

- Advantages. Generation of returns, green filter, location of recharge zones, slow discharge and the source of desalination and purifying plants are proposed.

The crossover of environmental planning and the MAR zones and their use of actual land (Corine) have enabled a matrix to be defined (table 3)., which enables the capacity and conditionality of each type of use/coverage to be evaluated with respect to possible MAR activity and the evaluation of risks. The relation between rows and columns for each descriptor has been marked with an X in this table. The number of crosses has been counted for each group of environmental considerations, generating an indicator, whose evolution in time enables the environmental potential to be characterised for each MAR zone and each new action.

The purpose of this is to obtain "uses capacity maps" for the different factors that determine the media where a new MAR device can be implemented.

		POLLUTION							HAZARDS					CONDITIONING FACTORS							WATER DEMANDS						TRENDS				ADVANTAGES							
		Dispersion type		Source																																		
CODE_00_5	DESCRIPTION	Diffuse source	Point source	Urban pollution	Agriculture pollution	Livestock pollution	Industrial Pollution	Suspended Solids Pollution	Water flow interception	Accidental spills	Endemic Biota	Saltwater intrusion	Unhealthy effects	High slope	High runoff	High altitude	Permanent free water surface	Seasonal free water surface	High water table	Drought occurrence	Domestic	Recreation	Ecology	Industry	Irrigation	Energy	Intensification	Climate Change sensitivity	Potential use for irrigation	Preferential restoration spots	Water return flow generation	Vegetative strips	Recharging point location	Slow discharge	Water source for desalination	Source for Sewage treatment	Minimal infiltration	TOTAL
12110	Industrial Areas		X				X			X		X	X								X			X							X							8
13100	Mining Areas		X				X	X	X	X		X	X											X						X			X					10
13200	Rubble landfills & waste dumps		X				X		X				X														X			X								6

Table 4. Example of the cross matrix with three classes of soil use and the environmental conditioning factors for each MAR activity. The total of the crosses is an indicator of environmental potential. The original table considered 85 possibilities.

Finally, an **economic study** has been developed based on the investment ratio or the cost of the device in relation to the water it will enable to be managed. The ratios for superficial MAR devices are about 1/5 of the ratio of the dams, while the ratio for depth probes/ASR is similar.

Soil and Aquifer Treatment techniques (SATs) and improved designs applied to agro-hydrology

From the description and analysis of the different typologies of negative impacts and problems encountered in the MAR devices, problem/solution binomials of an applied nature and based on SATs are proposed, such as:

- It is essential to minimise the decreasing trend of the infiltration rate by regulating flow and reducing the fines and the air in the AR water. To achieve this requires not agitating the water and to recharge slowly.
- The furrows on the bases of the ponds and channels increase infiltration by up to 25%.
- The communicating reservoir systems in channels and the valves on the well equipment reduce the dissolution of air in the water by around 2ppm.
- The most effective SAT measure is recharged water pre-treatment, accompanied by good maintenance, which minimises sedimentation (Bouwer, 2002).
- Given the high complexity of these operations, it is essential to have pilot plants to trial new technologies, devices, etc.

The application of the most recommended activities usually entails interaction of environmental impacts with a negative sign. Therefore, the most recommended alternative is the creation of an integrated system in which the balance is a positive sign, has an integral nature and high resilience.

The study areas are also the object of research into design and establishing control and maintenance parameters, which facilitate their operation and raise their effectiveness. The prototypes proposed at DINA-MAR include engineering developments to achieve minimal losses (evaporation, leaks, etc.), facilitate de-sedimentation, reduce transport, storage and pumping costs, enable operation at the optimal recharge point (including in situations where there is frozen or flooded soil) and to have a sufficiently high useful life to be profitable. Construction and maintenance costs must also be low.

Forestry engineering and palliative water management techniques

The hydric management palliative techniques based on recharging forestry and basin headwater areas studied to date are providing very good results in terms of making available a significant volume of subterranean waters in the “headwaters” of the aquifers, at the same time as helping to reduce the devastating effect of floods, etc.

The most suitable devices are dykes, which considerably increase the reserves in aquifers, as has been stated in studies and real data developed on the eastern part of Spain, where, from studies with climatic data and five years of infiltration data in two specific areas, the forests have enabled the infiltration of a volume of water greater than the 20% on the subsoils of the de-forested areas (Copano *et al*, 2010).

The creation of serialised infrastructures on the headwater basins and along the riverbeds, as well as re-forestation in recharge areas and appropriate management, involve an increase in the recharge of the aquifers and in their hydric availability. Equally, this forestry management favours higher quality waters and the management or appearance of ecosystems with a higher environmental quality.

Urban hydrogeology

As for re-utilisation s.s., MAR facilities are being incorporated into urban areas within the framework of complete management of water in building work, especially by means of Sostenible Urban Drainage Systems (SUDS) and the Integral Urban Water Management (GIAE).

The introduction of buildings and urban development entail a negative effect on the territory. The progressive impermeabilisation of the terrain causes great hydrological changes and means large investment in infrastructures to channel and treat the water. It´s absolutely necessary redesigns the water travelling in urban environment.

The proposal is a new approach to the rainwater management, including:

1 REGULATIONS	Existing regulation analysis, deficiency and improvement.
2 STATISTICS	Data compilation, analysis and conclusions.
3 TREATMENT	Existing systems, variant and improvement.
4 COLLECTED WATER	Existing systems, variant and improvement.
5 ENERGY	Energy inclusión in urban water cycle.
6 UPDATE	New systems.
7 INFORMATION	Awareness / conscience increasing.

Aimed at achieving good practices in cities:

- Minimising surface runoff in cities.
- Draining towards green areas instead of diverting the water to the sewers.
- Collecting rainwater for later use: irrigation, cisterns, washing machines…
- Keeping the city clean regularly.
- Creating awareness about sources of pollution: workshops, hospitals, etc.
- Minimising the use of herbicides and fungicides in gardens and parks.
- Education about the agents involved in designing and maintaining Cities.

The final objective is the rainwater runoff fracture, recovering the original infiltration capacity and breaking the “Heat Island” effect in cities.

CONCLUSIONS

- The future of special techniques must be based on improving “MAR zones” maps and environmental potential, with greater consideration for deep artificial recharge in multi-layer aquifers.

- It is essential to deepen the economical aspects of the MAR technique as well as to make a contingent evaluation on environmental and social aspects, taking the opportunity costs of the resource into consideration.
- The new designs must encompass low cost devices.
- GIAE opens an appropriate research line in the urban hydrogeology framework. Even though the SUDS concept is limited in some aspects that must be encompassed in projects for greater management of water in building construction, and is in the integral management.
- On the whole, in all the lines of action and the disciplines taken on, it has been seen that the advantages of the MAR technique outweigh the inconveniences, the innovation side must receive more emphasis, and that there are still many gaps in the state of this art, especially in those specialities apart from hydrogeology.
- The need to prolong these types of research projects must be highlighted in order to respond to the new threats of the 21st century, such as water contamination with emerging substances, climate change, etc. Such high complexity requires multi-disciplinary teams.
- It is only to be expected that many of the technological breakthroughs in the future will make an effective contribution to water management, not only for surface water but also for groundwater. Therefore, it is necessary to have access to the information and to improve hydrological information for the users, thereby making it easier for them to take part.

ADKNOWLEDGEMENTS

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Title: Application of the drop pipe hydraulic and aquifer hydraulic equations in design and operation of artificial recharge wells

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Abstract. In the State of Arizona similar to other arid regions in the United States and the World, artificial recharge wells such as aquifer storage and recovery (ASR) and vadose zone recharge (VZR) wells are becoming essential instruments to achieve more efficient use of scarce water resources. Two of the main purposes of these wells are to recycle water otherwise lost through evaporation and runoff, and to store high quality water for future uses. Due to their early stage of development, these wells have not received enough attention in terms of performance evaluation, design, and predictive maintenance based on industry standards. This paper evaluates the drop pipe hydraulic and aquifer hydraulic equations used to establish a relationship between an aquifer and recharge wells. This evaluation yields two important outcomes: (a) generation and application of orifice plate-injection rates curves which proved to be useful for design and replacement of orifice plates used to regulate the VZR well's injection rates, and (b) gaining a better understanding of the hydraulic relationship associated with the use of the two cited equations.

Keywords: ASR well, vadose zone recharge well, drop pipe hydraulic equation, aquifer hydraulic equation

Introduction

Artificial groundwater recharge is becoming an increasingly necessary component to enhance aquifer storage and guarantee future water supplies to a never ending growing population. Stream (perennial or intermittent) water, storm runoff, irrigation water, treated sewage water (reclaimed water), and drinking water (treatment plants) are used for artificial recharge (Bouwer, 2002). Spreading basins, infiltration galleries, and recharge wells (aquifer storage and recovery (ASR) and vadose zone recharge (VZR) wells) are the most common infrastructure used to store water in suitable aquifers (Sheng, 2005). Two of the main purposes of these systems are to recycle water otherwise lost through evaporation and runoff, and to store high quality water for future uses. In the United States, the number of groundwater recharge projects have increased dramatically over the last three decades: in 1983, there were only 3 operating ASR systems; by 1994, the number increased to 22 recharge systems; and by late 2005, there were 72 systems in operation with 100 more in development stage (Pyne, 2005).

Different publications have been released providing guidelines to implement artificial recharge groundwater systems. For instance, American Society of Civil Engineers (ASCE) (2001) and Pyne (2005) are two well known bibliographical references in this field. While the first one contains directions for the instrumentation of artificial groundwater recharge through different methods, the latter focuses only in the analysis of ASR wells. The two of them comprehend an artificial recharge holistic approach by covering concepts such as: planning, field investigations and testing, design and construction, regulatory and environmental aspects, operation and maintenance, etc. Due to their early stage of development, ASR and VZR wells still have not received enough attention in terms of performance evaluation, design, and predictive maintenance based on industry standards. In 1999, a new analytical method was devised by Morris and Quinn (1999) to represent the hydraulic relation between the aquifer and recharge wells through a combination of the drop pipe hydraulic and aquifer hydraulic equations. They successfully implemented that procedure for different purposes: to design a new drop pipe orifice at a desired rate and pressures, to determine the flow rate during meter failures, to estimate well performance during any given day of the recharge season, to forecast rate of individual well's clogging, etc (Morris, 2001; Morris and Quinn, 1999).

One of the main challenges when designing recharge wells is choosing the adequate orifice plate diameter, which is the device that controls the injection rate delivered to the well. The adequate selection of the orifice plate diameter regulates the optimum water level reached in the recharge well. The focus of this study is to apply and evaluate the effectiveness of the drop pipe hydraulic and aquifer hydraulic equations, and identify a systematic procedure for calculating the orifice plate diameter for artificial recharge wells. At the same time, this study is aimed to obtain a better understanding of the hydraulic relationship by using those equations in assessment of well performance.

Methods

In this paper, the analysis is based on the application of the following equation developed by Morris and Quinn (1999). This equation is composed of two terms: Q_o, the drop pipe hydraulic equation (Equation 2); and Q_a, the aquifer hydraulic equation (Equation 3). Both terms are defined using the orifice equation.

$$Q_o = Q_a \tag{1}$$

$$Q_o = C_{d1}\ A_o \sqrt{64.4\,\frac{\left((2.31\,P_d) + H_o - H_l\right)}{1-(\frac{A_o\,C_{c1}}{A_{i1}})^2}} \tag{2}$$

$$Q_a = C_{d2}\ A_a \sqrt{\frac{64.4\,H_a}{1-(\frac{A_a\,C_{c2}}{A_{i2}})^2}} \tag{3}$$

Q_o is injection flow rate (gallons per minute (gpm)); H_o is head in the drop pipe (feet (ft)); H_l is head loss in the drop pipe losses, which are in function of injection flow rates; A_o is hydraulic area of the drop pipe hydraulic restriction (square feet (ft^2)); P_d is wellhead pressure (psi); A_{i1} is cross sectional area of the drop pipe (ft^2); C_{d1} is coefficient of discharge for the drop pipe hydraulic restriction (unitless); C_{c1} is coefficient of contraction for the drop pipe hydraulic restriction (unitless); Q_a is flow rate through the aquifer hydraulic restriction (gpm); H_a is head on the aquifer (ft); A_{i2} is cross sectional area of the well casing (ft^2); C_{d2} is coefficient of discharge for the aquifer hydraulic restriction (held constant); C_{c2} is coefficient of contraction for the aquifer hydraulic restriction (held constant).

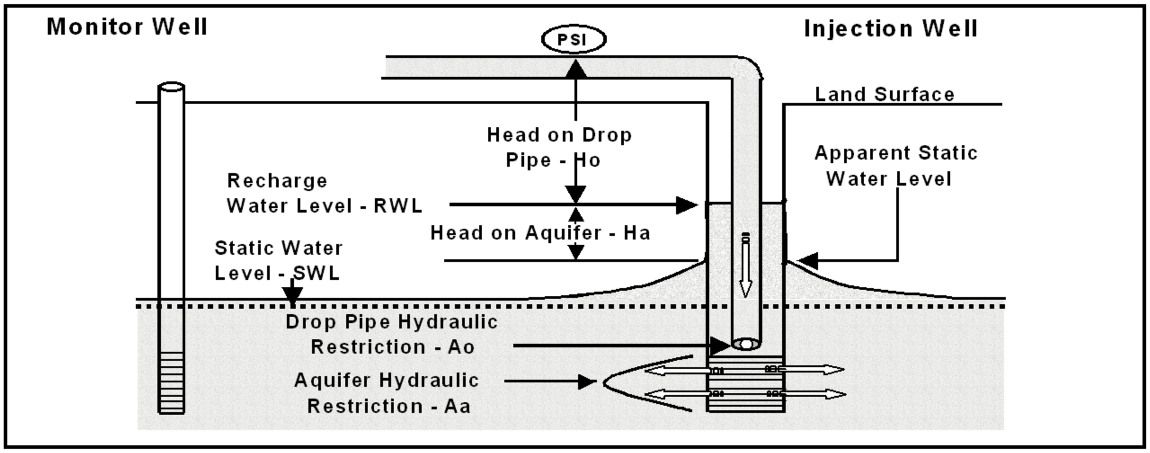

Figure 1. Representation of hydraulic relationship between aquifer and injection well
Note: From Morris and Quinn (1999)

A typical design of VZR wells consist of the following components: fabric sleeve to prevent migration of fine material when shutting down the VZR well; casing made of PVC section (with a usual length of 180 ft) perforated in the lower portion and with a 10 ft blank reservoir section at the bottom; gravel pack that serves to increase the permeability of the well; injection line and orifice plate to control the amount of supplied water to the well while minimizing air entrainment; two gravel pack injection lines, made of PVC used to supply water to the well during rehabilitation and chemical treatment; and an underground vault secured with lockable doors to protect and allow easy access to the well, piping, and valving (HydroSystems, 2009).

The selection process of obtaining a proper diameter of orifice plate has been normally chosen by trial and error. This process has been applied in both new wells and old wells until reaching a satisfactory operating water level in the VZR well. This process can be time consuming and does not provide knowledge about the hydraulic factors controlling flow and defining the operating water levels. Morris and Quinn (1999) used Equation 2 to estimate orifice plate diameter for a given injection flow rate, total head, and drop pipe properties. Because of the previous cited parameters are evolving over time due to decreased performance of wells and fluctuations/modifications to the water supply pipeline, a graphical tool has been created using an excel spreadsheet by applying Equation 2. This graphical tool not only practically and systematically facilitates the selection of orifice plate diameters, but also provides more insights into the well-aquifer hydraulic relationship.

The graphical tool is a family of orifice plate diameters-injection rates curves for a recharge injection well. Given the specific characteristics of the well (length) and the drop pipe (length and material), A_{i1} and H_o are defined as fixed parameters. Also, C_{d1} and C_{c1} are fixed parameters obtained from the standard hydraulic tables. A_{i1} is in function of the orifice plate diameter. Substituting all the previous parameters in Equation 2 allows for the estimation of the flow rate. This procedure was executed by trial and error because H_l (head loss) in the drop pipe is in function of the injection flow rate. By running Equation 2 for a range of orifice plate diameters and assuming a constant P_d (pressure value in the drop pipe (injection line)), a set of orifice plate diameters-injection rates curves are estimated for different depth to water (DTW) values.

Results and Discussion

Graph Interpretation and analysis

Figure 2 shows the curves for a well with a depth of 180 ft, and a PVC drop pipe of six inch (in) diameter and 175 ft long with constant pressure of 12 psi. This figure provides useful insight regarding the different flow rates capacities for the wells for different combinations of orifice plate diameters and DTW values. According to the figure, this VZR could have a theoretical injection rate capability ranging from 83 to 1409 gpm. This means that this well could be suitable for vadose zone sites with recharge capacities within the previously mentioned range.

Because of the nonlinear nature of Equation 2, it was expected that a direct nonlinear relationship would be found between orifice plate diameter and injection rate. For instance, the 60 ft below land surface (bls) DTW curve indicates an injection rate of about 100 gpm for a 1 in orifice plate and a much greater rate of 960 gpm for a 3 in orifice plate. According to the figure, the curves with shallower DTW values present the lower injection flow rates. For instance, for an orifice diameter range of 1-3 in, the 20 ft bls DTW curve gives an injection flow rate range from 80 to 710 gpm; however, the curve with the deepest DTW (160 ft bls) has an injection flow range from 160 to 1,420 gpm. This seems logical considering that a lower head (deeper DTW) in the casing will represent less resistance to the water flowing out from the injection line (through the orifice plate) to the casing. The absolute difference of injection flow rates across the curves increases as orifice place diameter increases. For example, for a one inch diameter orifice plate, the absolute difference in injection rates is 81 gpm (164 gpm - 83gpm); whereas, for a three in diameter orifice plate, the injection rate's absolute difference is near to 703 gpm (1409 gpm - 706 gpm). This situation is due to the exponential increase of head losses as injection flow rate increases.

As discussed in the previous paragraph, if only the interaction between the injection drop pipe and the casing of the well are considered, for a given orifice plate diameter, injection rates increase as the DTW gets deeper, and vice versa. However, when taking into consideration the interaction between the well and the vadose zone, the opposite should be more plausible: the deeper the DTW (which means lower head in the well), the lower the injection rate into the vadose zone, and vice versa. More specifically, let's assume that we have a VZR well with the characteristics shown in Figure 2 and with an orifice plate's diameter of 2 in. According to the figure, for a DTW equal to 160 ft bls, which gives a water level head (depth of water) of 20 ft, the VZR well is expected to have an injection rate of 649 gpm. However, according to the authors' experience, given the cited conditions, it is quite impossible that the vadose zone surrounding the recharge well is able to handle that injection rate for such a low water head in the well.

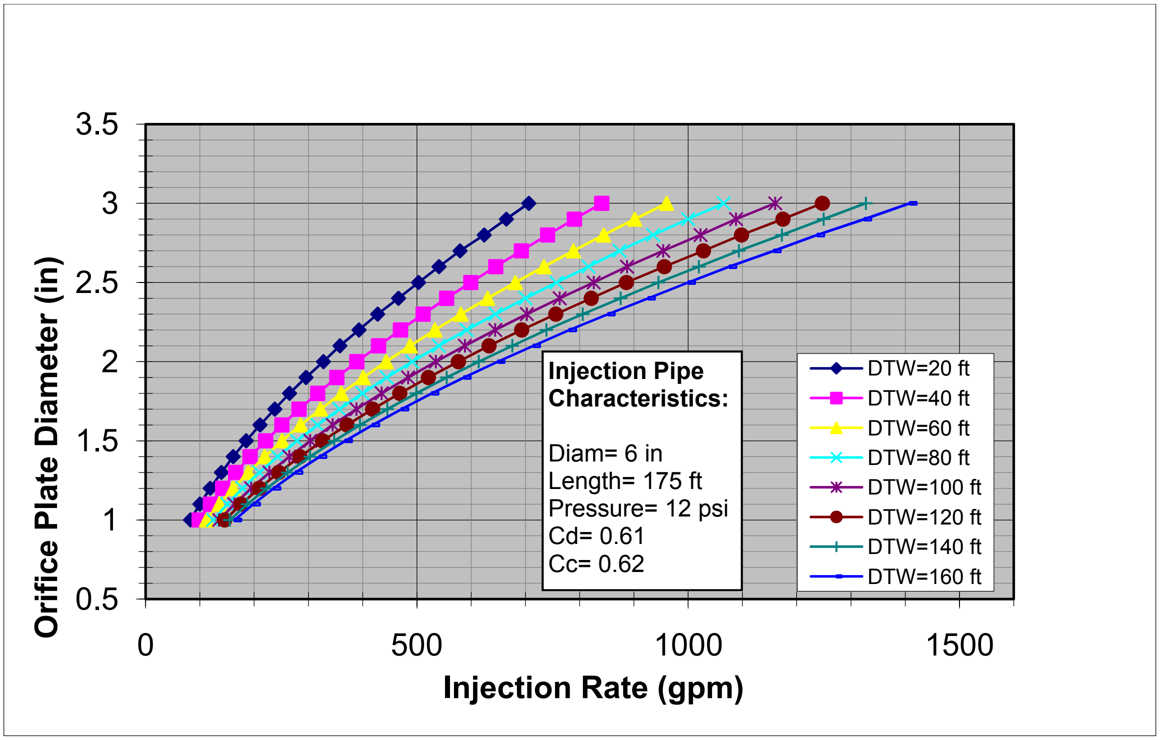

Figure 2. Orifice Plate-Injection rate curves for recharge wells (using orifice plate equation)

Potential applications/conditions

The application of this graphical tool, such as Figure 2, was implemented for two main conditions: replacing orifice plates in current VZR wells; and defining an orifice plate for new VZR wells. For both conditions it is recommended to identify the vadose zone potential infiltration rates. Most of the time, these values are obtained just after the construction of the well by performing an injection recharge test. Assuming a potential infiltration rate of 500 gpm and a DTW of 60 ft bls (which gives a recharge specific capacity of 4.2 gpm/ft for a well of 180 ft deep and with the same characteristics depicted in Figure 2), it will be required to have an orifice plate diameter of 2.1 in. In some instances, it will not be possible to know beforehand the VZR well's potential infiltration rate; instead, an average range infiltration rate will be known. For instance, let's say that the average infiltration rate range for a VZR well is from 500 to 1,000 gpm according to performance of nearby artificial recharge wells. With this information, we can identify, assuming a VZR well with the same characteristics of the well of Figure 2, that the most appropriate orifice plate diameter would be near to 2.6 in (considering the average flow rate of 750 gpm and a DTW of 70 ft bls). Even though this analysis was mainly dedicated to VZR wells, same family of curves can be applied to define the optimum orifice plate for ASR wells.

Verification

The above graphical tool was tested and verified for different VZR wells located at different recharge facilities in the Phoenix metropolitan area: The City of Scottsdale's Water Campus Facility, City of Glendale Arrowhead Recharge Facility, and the Surprise SPA-1 Recharge Facility.

Figure 3 shows the verification efforts performed on RS-25 VZR well, one of City of Scottsdale Water Campus Facility's VZR wells. This facility consists of a water reclamation plant designed to treat sewage from Northern Scottsdale as well as surface water from the Central Arizona Project (CAP). The water reclamation plant facility treats reclaimed water for reuse and stores the excess water in the aquifer for future use. It includes a 20 million gallons per day (mgd) water reclamation plant, a 16 mgd advance water treatment plant composed of microfiltration and reverse osmosis, 27 VZR wells, and 28 VZR emergency wells (Gastélum et al., 2009). For selected years since its operation in 1999, Figure 3 displays measured and estimated injection flow rates, and measured DTW. The absolute difference between measured and estimated injection flow rates ranges from 6 to 17% with an absolute average of 8% disregarding the data point (11/2005) that has absolute difference of almost 900%. This measured injection flow rate (11/2005) does seem to be an outlier. Given the corresponding measured pressure (0.9 psi) and a sustained DTW (156.6 ft bls), this injection rate should be higher than the recorded value.

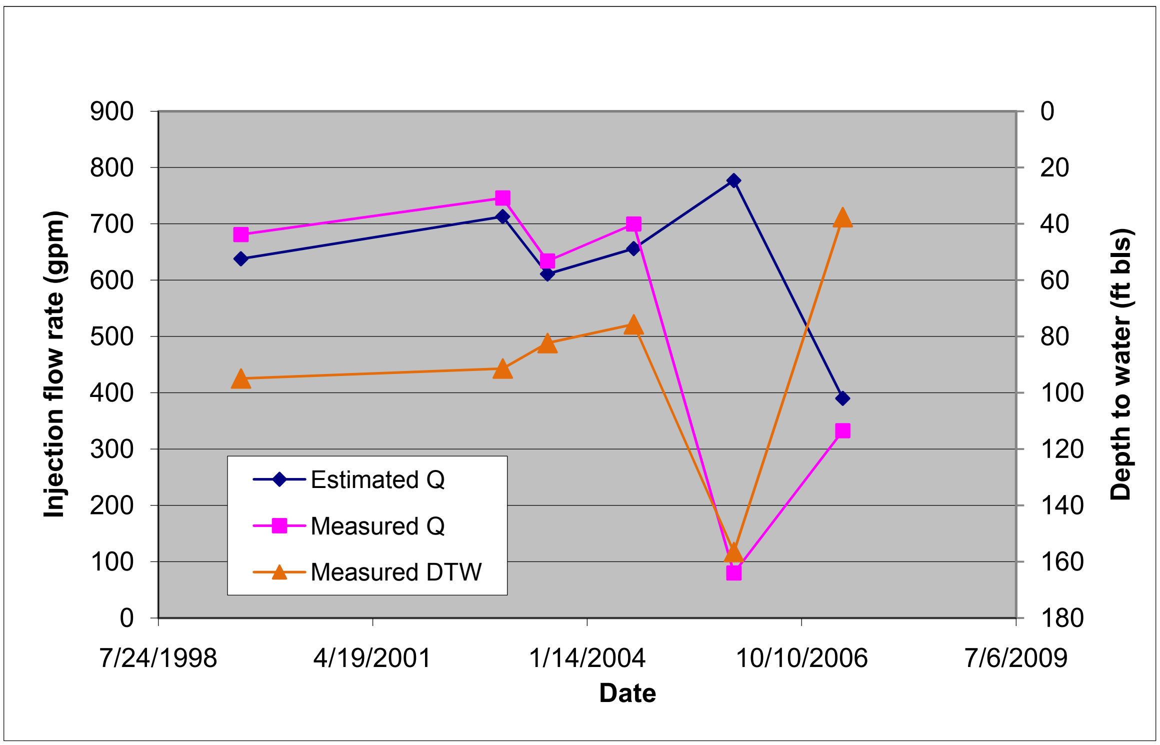

Figure 3. Comparison between real and estimated injection flow rates for City of Scottsdale Water Campus' RS-25 VZR well

Limitations and Recommendations for improvement

Morris and Quinn (1999)'s approach to simulate aquifer recharge capacity near to the recharge well by using the standard orifice equation is an ingenious procedure. However, given the physical differences between an orifice and an aquifer (or vadose zone), it comes with a challenge to give a meaningful interpretation to parameters in orifice Equation 3, such as coefficient of discharge (C_{d2}) and coefficient of contraction (C_{c2}). More important, it is in question to assure the universal application of this equation given the complex heterogeneity of aquifers (or vadose zones). Also, this equation could not account for the dynamic behavior of the efficiency of the aquifer (or vadose zone), which is affected by clogging factors or well interference. Therefore, the authors suggest using more conventional equations involving hydraulic conductivity to represent the soil recharge capabilities. For instance, Bouwer (2002) provides with a holistic analysis of different hydrogeologic and engineering concepts associated with most of the artificial groundwater recharge systems. He provides with an extensive review of infiltration literature including the compilation of equations to estimate infiltration and recharge rates for different artificial groundwater recharge systems. Gastélum et al. (2009) applied the specific capacity concept, which is normally a standard procedure in pumping wells to determine their efficiency, to evaluate the historical behavior of VZR wells' infiltration recharge specific capacity. Future research would be focused to investigate the coupling of Equation 2 with an equation involving hydraulic conductivity in order to define a more comprehensive hydraulic relationship between artificial recharge well and aquifer (or vadose zone).

Conclusions

Artificial recharge wells have become a powerful tool in conjunctive management of surface water and groundwater. More attention should be paid to their design, evaluation, and maintenance as well as establishment of new standards.

A graphical tool, a family of orifice plate diameters-injection rates curves, was developed for optimal design of an orifice plate, which is key instrument to regulate flow and to prevent air entrainment into the well. Successful verification of the graphical tool's values with historical values for different artificial recharge wells located in the Phoenix metropolitan area was implemented.

This graphical tool provides a discernment of relationships occurring between injection flow rates, DTW, and the diameter of an orifice plate. Moreover, it was shown that the practicality of the implementation of this graphical tool for defining orifice plate diameters for both existent and new VZR wells with known artificial recharge injection rates.

As noted, there are limitations to estimate the aquifer recharge capacity in the area surrounding the artificial recharge wells using the orifice equation. The recharge capacity is controlled by hydraulic conductivity of the aquifer. Therefore, future research will be aimed in coupling drop pipe equation with aquifer (vadose zone) equations, which incorporate more conventional parameters such as hydraulic conductivity.

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MANAGED AQUIFER RECHARGE AND PUBLIC PARTICIPATION

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Abstract

People are more and more aware of the possible impacts and changes in their living environments. Large managed aquifer recharge (MAR) projects for potable water treatment have met strong criticism both in legal public participation processes and by voluntary participation. The objective of this paper is to present the influence of public participation on the planning and realization of large MAR projects. Examples are presented using two MAR projects as case studies. The interactions between the environmental impact assessment processes and the public participation are described. Public participation has turned into public resistance in both MAR plant areas. In both cases there are common ways of action, which are summarized in the paper. Based on the experience gained, a project management point of view on public participation is presented: (1) it is an essential part of the project, not to be neglected, (2) it sharpens the activity of the project management, (3) the project gains new knowledge due to fulfilled claims from the public, (4) the project may be delayed by years – this should be taken into account when making schedules, and (5) it is essential to put emphasis on continuing communication with all stakeholders, including the shareholders.

Keywords

Finland, Managed aquifer recharge, Public participation

Introduction

Managed aquifer recharge (MAR) is used for the production of drinking water in Finland. The first MAR plants still in use were built during the 1970's and at the moment the total number of plants is ca. 25. The share of MAR of the total drinking water production in 1999 was 13 %, whereas natural groundwater and surface water accounted for 46 % and 41 %, respectively (Isomäki et al. 2007). MAR has a long tradition in the Nordic countries and the first MAR plant was taken in operation in Sweden in 1898 (Hanson, 2000). One of the major aims of drinking water treatment in Finland is the removal of natural organic matter (NOM). During a MAR process NOM is removed by physical, chemical and microbial processes (Lindroos et al. 2002, Kortelainen and Karhu 2006, Kolehmainen et al. 2007). MAR plants are usually situated on eskers. Basin infiltration has been used for decades, sprinkling infiltration (Fig.1) was also taken in use in the mid 1990's. When sprinkling infiltration is used, there is no need to dig and construct basins and the direct physical effects on the landscape are reduced. The use of well infiltration or injection is being studied.

Figure 1. Sprinkling infiltration tests being conducted at a Tavase MAR site in 1997.

Two large-scale potable water MAR projects are being conducted in Finland: TAVASE project and Turku Region project. Such MAR projects can be large by both the planned production capacity and the time-frame. The time-frame from the planning initiative to the start-up of the water supply is usually several years or even decades. The order of magnitude of such projects is not only a condition of public participation, but it also provokes public participation in a modern society where people are more and more aware of the possible impacts and changes in their living environments. The objective of this paper is to present the influence of public participation on the planning and realization of MAR projects.

TAVASE MAR project

The development of cooperation between the municipalities in the Tampere and Valkeakoski Region in water acquisition has a long history. The construction of MAR plants to the west and east of Tampere was proposed in the general plan for water protection and water services for Kokemäenjoki river basin by the National Board of Agriculture already in 1969. The possibilities to produce MAR water were studied in the esker formations west of Tampere in the 1970's. A more detailed general plan was prepared by the municipalities of Tampere and Valkeakoski Region ('TAVASE' concept) in 1993. Research for the MAR in the esker formations east of Tampere was started in the mid 1990's . The decision made by the municipalities to continue the development of the TAVASE concept was based on the following (Tavase 2001):

1. Population growth in the TAVASE area is expected to be 35,000 – 50, 000 during the next 30 years
2. Surface water sources and poor quality groundwater sources will be replaced with more efficient good quality groundwater sources and MAR. The future use of surface water would require the rehabilitation of surface water treatment plants and the extension of suction pipes to those locations in the lakes, where raw water quality is better.
3. The vulnerability of water distribution could be decreased by constructing the networks so that delivery can be arranged from various directions.
4. The conditions for the enterprises needing good quality water would be improved, and the establishment of new companies in the region would be made more attractive.

Tavase Ltd. was founded in 2002 and it is owned by seven municipalities. Tavase Ltd. aims to build a MAR plant to east of Tampere to provide potable water for the municipalities of Tampere and Valkeakoski Region (Fig. 2). The number of people living in the area is more than 300 000. The capacity of the plant will be 70 000 m^3/d. The environmental impact assessment (EIA) was conducted in 2002 – 2003. At the moment additional studies are conducted on site and the start-up of the water supply is expected to take place in 2015 – 2016.

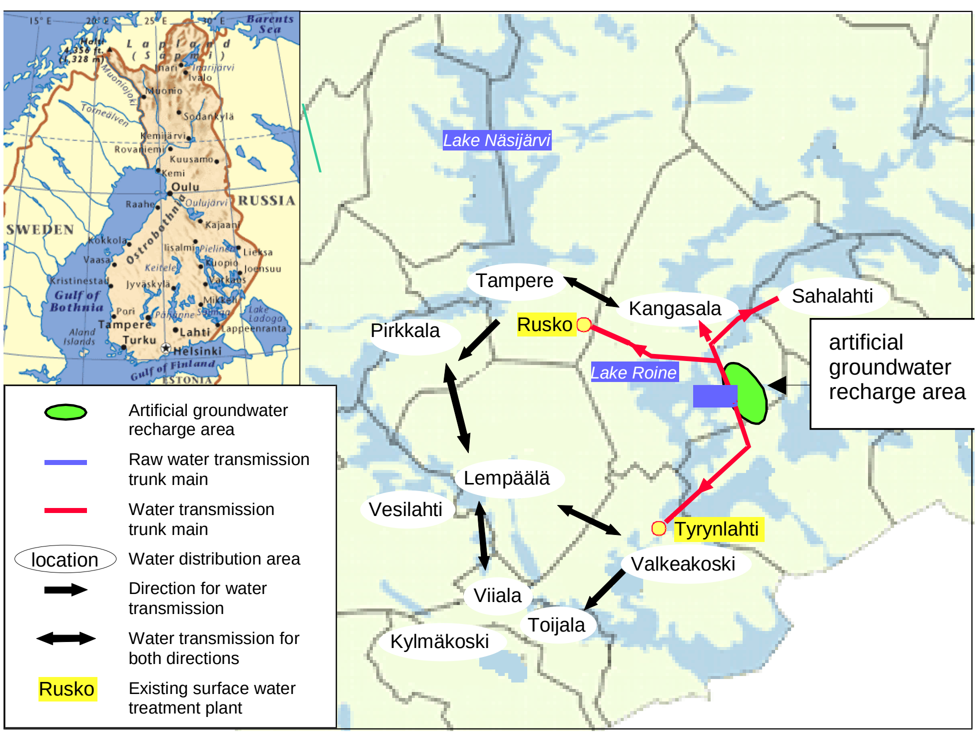

Figure 2. The location of the planned TAVASE MAR plant and the connection of the distribution networks.

Turku Region MAR project

Turku Region Water Ltd.'s MAR project is the solution to the water supply problems in Turku Region. The concept of MAR for Turku Region was introduced during 1960's. During long, dry periods the Turku Region's water supply services have suffered from lack of good quality raw water. During dry periods the diminishing supply of water leads to water scarcity, in addition to deterioration of the quality of raw water. Besides, the use of surface water as the raw water source implies a significant environmental risk due to, e.g., oil spills or similar chemical accidents.

Turku Region Water Ltd. is owned by seven municipalities. The company is constructing a MAR plant to provide potable water for the city of Turku and its neighbouring municipalities. The number of people living in the area is around 300 000. The capacity of the plant is 105 000 m^3/d. The general plan of the project was completed in 1999 and the decision by the municipalities to execute the project was made in 2001 (Fig. 3). The EIA was conducted in 2000 - 2001. The construction of the plant is being finalized. Full-scale trials were started in 2010 and the start-up of the water supply will take place in 2011.

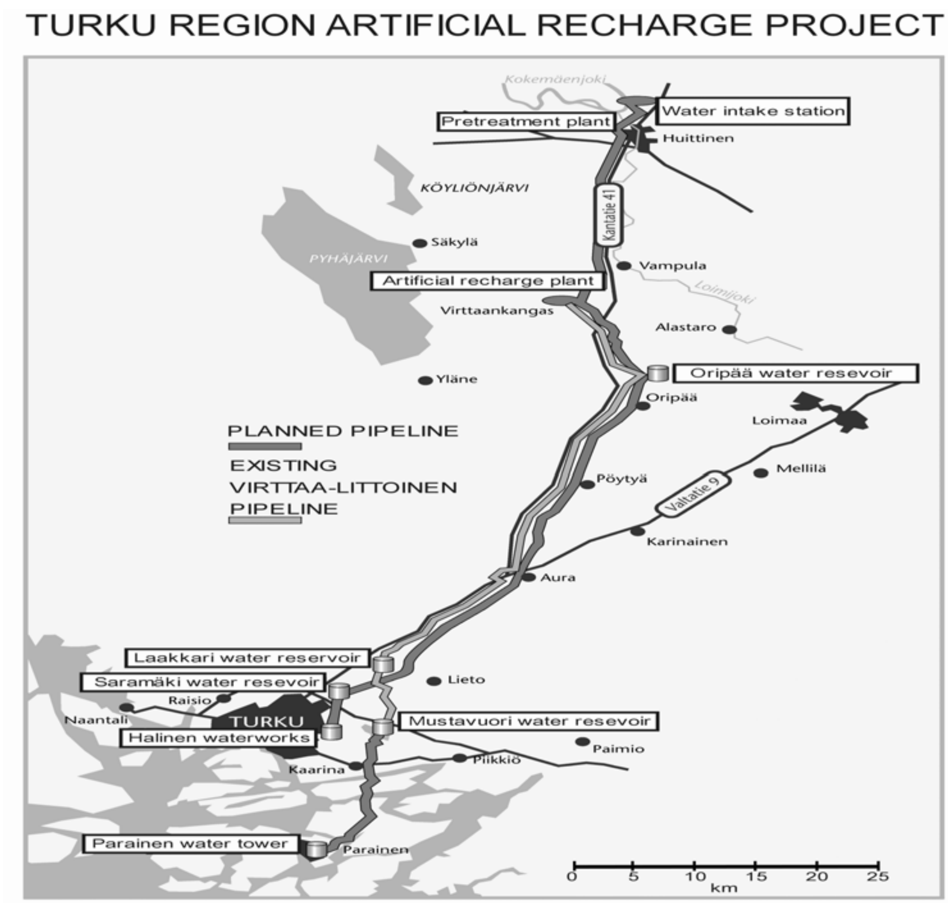

Figure 3. The locations of the Turku Region Water Ltd.'s MAR plant and the water distribution pipelines.

Public participation

People have different ways for public participation: some active people may utilize the political decision making system, some activate only when feeling threatened ('not in my backyard'), some form loose organisations to promote just one aspect (theirs) of the whole, some do not participate at all. There are several aspects of public participation, e.g., getting new relevant information for decision-making (Primmer and Kyllönen, 2006), social learning which enables stakeholders to adjust their views and attitudes by looking at problems from their neighbours perspective (Wolters et al., 2006) and institutionalized public participation (Reed, 2008).

According to the Finnish law an environmental impact assessment (EIA) must be conducted for large MAR projects. The objectives of the EIA procedure are to enhance the assessment and consideration of the environmental impacts on planning and decision-making, and to improve the citizens' possibilities to receive information and to participate. The EIA procedure consists of two successive parts: first a programme for the EIA is prepared and submitted for public commentary, then the assessment is carried out according to the approved programme. The assessment includes several sub-tasks, which are reported in the EIA report.

During the planning of TAVASE MAR project and the EIA process altogether 16 information, consultation and discussion events for the public and other stakeholders were arranged (Suunnittelukeskus Oy, 2003; Hukka and Seppälä, 2005). These events were targeted at, e.g., the public, landowners, entrepreneurs and municipal councils. In order to increase participation, an EIA task force and an EIA steering committee were established to guide the process, to introduce the views of local and various other stakeholders, and to give information on the project at the grass-root level.

The inhabitants, entrepreneurs and other stakeholders in the area and surroundings were heard, for instance, by interviewing. Technical field visits to existing MAR plants were organized. Newspapers have also reported on the project on a regular basis. The brochure on the water acquisition cooperation in Tampere and Valkeakoski Region was published, for instance, on websites.

The EIA process conducted by Turku Region Water Ltd. (Turku Region Water Ltd. et al., 2001; Turku Region Water Ltd. and Jaakko Pöyry Infra, 2003) was augmented by a separate social impact assessment (SIA). Several project presentation meetings were arranged for the public. Turku Region Water Ltd. made a general brochure and a video on the artificial ground water project. It was possible to follow up and give comments on the development of the project through the internet pages of Turku Region Water Ltd.

One part of the SIA consisted of sending questionnaires to more than 2500 people and entrepreneurs. In addition, interviews with entrepreneurs were conducted. Altogether about 600 hundred filled questionnaires were analyzed (Virtanen, 2002). According to the SIA the artificial groundwater project had created an atmosphere of uncertainty among the local people. About 50% of people thought that they did not have enough information on the project. Although several project presentation events were arranged for the public, about 15% of people mentioned that they had not been provided with enough presentations on the project. People opposed the project, stating following reasons: the need for water and the water treatment process are dubious, the ridge and the groundwater might get contaminated, the ridge might get wetted and the structure of the forests might change, the conditions for entrepreneurship might get worse, recreational conditions and general standards of living might get worse, the project costs are too big, and the images of the municipalities in the area might get worse.

Many of the local people in the area of TAVASE MAR plant (Fig. 4) expressed similar opinions during the EIA process. In addition, people were concerned about the impacts on landscape, soil stability under individual houses and the state of a nearby nature conservation area.

Figure 4. Two of the three planned MAR sites of Tavase Ltd. are located on the esker in the municipality of Kangasala. There are gravel pits on the east part of the esker, the MAR sites are planned to be constructed on the west part of the esker.

From public participation to public resistance

Public participation can provide support for a project, especially if the project has been initiated among the public itself and the public feel motivated towards the goals. These kind of projects may be smaller by size, which make them easier to comprehend. However, when the project size increases , more people are involved and the inputs and outputs of the project do not necessarily meet at the same location, there is an increased possibility for critical public participation or public resistance.

Public resistance may occur in different kinds of large infrastructure projects. Wastewater treatment plant projects have generated public resistance in Finland. A groundwater withdrawal project in Canada faced critical participation when the people felt that 'their' groundwater was going to be led to the neighbouring cities (Hofmann and Mitchell, 1998). Wastewater reuse project was abandoned in Australia due to public resistance (Hurlimann and Dolnicar, 2010).

Both MAR projects described here have generated strong criticism and generally opposing public participation. The critical attitude did not decrease during the EIA-processes, however, it is not the purpose of an EIA-process, either. The criticism is essentially the same for the both MAR projects and it is based on the concerns already reported during the EIA-processes. Public participation has taken the forms of loose public movements in both recharge plant areas. In both cases there are common ways of action, which are summarized in the following.

The main objective of public participation is to prevent the construction of a MAR plant. The objective is not to just reduce or remove possible negative impacts or seek for compromising solutions. Negotiation is difficult in that atmosphere. Public participation aims to affect the general public opinion and change it to be against the project. It aims also, directly and indirectly, to change the opinions of the decision-makers to make them vote and act against the project.

Delaying the project is an essential means to fulfil the main objective. When the project is delayed, there is more time to try to change the opinions of the decision-makers, or, in the case of political decision-making, to change the decision-makers in elections. One effective way to delay the project is to appeal to higher courts during permit processes. The ways of action are based on making the situation more critical than it actually is by exaggeration, telling lies or by the blackening of the project or persons working for it. The citizens are not responsible for their statements (cf. the company needs to act responsibly).

In practice, some of the features of critical public participation (public resistance) are:

- there are only a few high profile activists representing the movements
- academic merits of the activists are emphasized to show expertise
- charismatic behaviour is practiced in public appearances
- opinions are given as facts without the need to prove them ('the project will not succeed, because it is too big')
- political decision-makers and other stakeholders at municipal and regional level are directly contacted and given brochures and other supportive material
- a greater exposure is tried to gain by petitions at shops or other public places and inviting visiting people to sign them. Petitions are then delivered to courts to show that a lot of people are supporting the thoughts of the activists.
- local newspapers write frequently on domestic topics, defending the opinions of those who subscribe to the newspapers
- the activists write constantly to the newspapers' letters to the editor –sections. In addition, some send journals 'expert articles', which are based on critical opinions.
- the activists run organizations, the names of which represent 'good values', such as 'nature conservation society'. Declarations of the organizations are directed against the project.
- the activists arrange seminars and demonstrations (Fig. 5) to support their opinions. The seminars can be disguised as being 'scientific'.
- people send claims to the company responsible for the project. These claims can include, e.g., demands to remove monitoring wells or pay additional compensations for proposed or future damage.
- the activists contact the owners (municipal councils) of the company responsible for the project. The members of the councils are sent letters and other information describing the project as not viable. This kind of action may confuse some of the owners of a company, which has several municipalities as owners.
- people can utilize passive resistance, e.g., by being absent from predetermined gatherings and by forbidding conducting necessary measures such as groundwater monitoring
- some candidates in municipal and parliamentary elections try to gain greater success by either promising to act against the project or by giving exposure for critical opinions in their web sites
- the activists emphasize the use of the internet. Critical opinions are encouraged and information is spread fast with interpretations at the discussion boards
- the activists do networking with other public movements, e.g., between TAVASE and Turku Region projects
- some of the activists promote vandalism, e.g., by spilling oil to the ground or pouring organic solvent to a ground water monitoring well.

Figure 5. A demonstration against Turku Region project in 2000. 'You get the water, we get the pollution'.

The project management point of view

Public participation is an essential part of the planning and realization of large-scale water projects. The procedures for public participation and interaction based on laws are important, but not comprehensive enough for the project management. According to Creighton (2005), public participation must take place while the management is still considering options and, preferably, when the problem is being defined. Further, the essence of an effective public participation process is that the public is engaged at those points in the process where the public can influence decisions.

Based on the experience gained from the two MAR projects, the project management needs to address public participation from the beginning to the end of the project. The EIA process cannot be considered more than a start or an intermediate stage of public participation. Public participation must be taken into account when planning schedules, the project financing and deciding the necessary resources: the Turku Region MAR project, e.g., was cited by the media nearly 800 times during a 13 month period in 2008-2009 (Fig. 6) and over 100 hundred times during one month alone. Coping with the vast amount of information in the citations -coupled with interviews by the media - needs to be taken into consideration when allocating personnel resources.

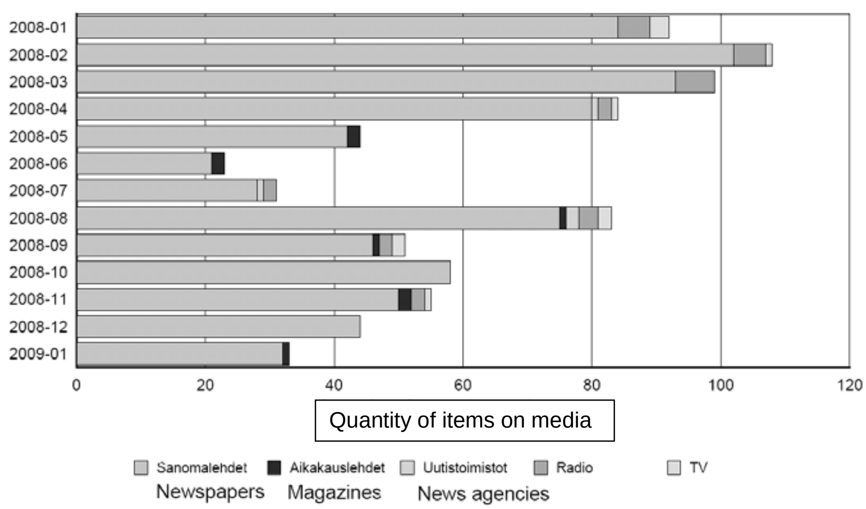

Figure 6. The number and distribution of citations by the media on Turku Region MAR project during a 13 months period in 2008-2009.

Public participation sharpens the activity of the project management. The management needs to concentrate on the real concerns of the citizens and provide them with carefully thought answers. On the other hand, the management must correct the false information given in propagandistic statements by the activists. Because the public participation aims at proving the project unnecessary or harmful, the project management needs to try to crystallize the essential basic facts and objectives of the project. This will make it easier to give exact, but clear answers to the frequently asked questions. It has been found that people want to receive simple information, which is easy to assimilate.

The interaction with public increases the overall knowledge on the project: the public may demand additional investigations or research, the results of which can be beneficial to the project. It can also be reassuring, that practically all possible points of view on the project planning and realization will be considered until the construction is finished.

The major constraint with critical public participation is the possibility that the project is delayed. In the case of TAVASE MAR plant, the research permit processes lasted almost five years due to appeals to higher courts. When the project is delayed, it may have several effects. If the project is targeted to build new infrastructure (MAR plant) that replaces old infrastructure (surface water treatment plants) the maintaining of the old infrastructure may be problematic or costly in time. The operational project costs increase while waiting for permits to proceed. The investment costs may change in time depending on the economical situation in the country and the trade partner countries: the costs may either increase or decrease. The delay gives the project management time to concentrate on more detailed planning, whereas some plans may need to be updated. Delay may also cause confusion among the owners of the company responsible for the project.

The project management needs to put effort on continuing communication and spreading information on the project. Information should be targeted at the public and specific target groups, such as landowners, the controlling authorities and the representatives of the owners of the company. In the case of companies owned by municipalities, the information for owners should be given to both politically elected members of the boards and councils, and municipal officers responsible for implementation of technical and economical measures. The project management needs to be active in communication: it is better to spread information in advance than to just respond to others' initiatives. The spreading of information should be continuous and the same information should be repeated frequently. The facts, which are self-evident for the project management, are not necessarily well-known among the other stakeholders.

Conclusions

The order of magnitude of large MAR projects is not only a condition of public participation, but it also provokes public participation in a modern society where people are more and more aware of the possible impacts and changes in their living environment. Large-scale MAR projects have generated strong criticism and generally opposing public participation in Finland. Even though the different project stages have so far been, step by step, approved by the controlling environmental authorities and by the agencies granting permits, the criticism has not decreased. Public participation related to the standard environmental impact assessment procedure is a good tool in gathering and distributing data on the project. However, it is not a means to prevent criticism.

The public, including activists, demand continuing participation between them and the project management. From the project management point of view following aspects on public participation should be considered:

(1) It is an essential part of the project, not to be neglected,
(2) It sharpens the activity of the project management,
(3) The project may gain new knowledge due to fulfilled claims from the public,
(4) The project may be delayed by years – this should be taken into account when making schedules, and
(5) It is essential to put emphasis on continuing communication with all stakeholders, including the shareholders.

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Expanding Experience at Intermediate and Deep Water Levels

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Abstract

Driven by economic, environmental, legal, political, and technical factors, the huge worldwide potential for aquifer storage & recovery (ASR) is translating into a sizeable and growing body of practical knowledge on the design, construction and operation of injection and pumping wells. With particular emphasis on settings with water levels from 500 to 2500 feet (150 to 750 m) below ground, this talk will summarize the collected experience with issues of equipment & material choices, drilling, data acquisition & control, maintenance, operational optimization & troubleshooting. In this context, it will also highlight examples relating to the interplay of our groundwater and oilfield experiences.

Key words: *well, management, operation, aquifer, storage, recovery, recharge, material, equipment, design, engineering, drilling, completion, installation, maintenance, failure, troubleshooting, depth, strength, joint, thread*

Introduction

The vast majority of this planet's liquid freshwater (10^7 km^3) is contained in intermediate (100 - 200 m) and deep (200 - 2000 m) aquifers[1]. For example, the Middle-East features prolific formations such as the Wasia aquifer with some 200 trillion barrels (32000 km^3) of water recharged during the last glacial epoch[2]. Partially empty (after having been pumped down), such formations are excellent candidates for ASR projects. Many are similar to aquifers found in Colorado and the desert Southwest of the United States of America. There, the author has been involved with many ASR projects, as most of them require downhole flow control valves. And those wells also have been a source of most practical ASR knowledge to date. In fact, we have painfully learned that bad things can happen when you go deep - not just in deepwater drilling for oil & gas[3] (leaking Gulf of Mexico well).

In some instances, we have pushed the envelope for design, construction and operation of injection and pumping groundwater wells to depths greater than 2500 feet (750 m). There, run-of-the-mill materials, conventional design choices, and standard drilling, completion, operating & troubleshooting methods start to reach their limits. At a minimum, they need to be carefully re-thought and, as necessary, modified. Note that, while for legal and political reasons no individuals or organizations are named below, all of the bad scenarios and incidents described have actually happened.

Collapse strength of casing and screen

Thermoplastic polyvinylchloride (PVC) is occasionally used on shallower wells but may not be suitable for deeper wells exceeding 500 - 1000 feet (150 - 300 m). As downhole temperatures rise due to the local geothermal gradient, during completion (e.g. heat of hydration of a cement job), or operation (e.g. waste heat from pump or inadvertent water circulation), the softening and weakening of the PVC becomes an issue. Conventional steel, such as used for line pipe or merchant pipe from water well supply houses, retains its typical yield strength of 30000 - 35000 psi (210 - 240 MPa). Oilfield casing offers even higher yield strengths of 55000 - 110000 psi (380 - 760 MPa) and beyond. However, depending on the local environment, water quality, and proximity of materials with different electropotentials, corrosion will gradually diminish wall thicknesses. As loads increase with depth, and temporarily during events like earthquake-induced liquefaction, steel casings too may reach or exceed their limits - and collapse. To prevent such incidents, collapse strengths of the casing must be calculated to meet the applicable requirements[4].

Joint strengths

Joints often are the "weak link" of a column pipe, carrying the static and dynamic loads of the pump, water column, and column pipe itself. While we observe a continuation of the trend from the use of conventional steel towards stainless steel (SS), either 304L or 316L alloy, normally annealed SS also only offers yield strengths of 30000 to 35000 psi (210 - 240 MPa). On special order, SS pipe can be made without the annealing process, which results in a yield strength of from 55000 to 75000 psi (380 - 520 MPa). Such material allows for reduced wall thicknesses at less cost, especially when one considers no loss of wall thickness due to rusting. Baski recommends consideration of 40-ft (12 m) long pump column pipes made from non-annealed SS, to further reduce the number of connections, installation effort, and, ultimately, cost.

Historically, vertical turbine pumps have been used at depths up to 500 feet (150 m) where it was acceptable to use relatively thin-walled column pipes with low joint strengths. Sometimes these pumps are set up to 1000 feet, which increases the potential for failure. First, consider that ASTM-312 standard pipe schedules[5], allow for the actual wall thickness to be 12.5% less than the nominal wall specified. This is not a well-known fact, not even within the industry. Pipe manufacturers exploit this allowance to reduce the cost of manufacturing the pipe. Based on many years of Baski measuring actual wall thicknesses, one must count on the walls being at least 10% thinner than the nominal wall thickness listed. In other words, the strength of pipe already starts at least 10% lower than you would otherwise expect - which also contributes to lower joint strengths. The remaining wall under the threads can be reduced by up to 17%.

Of course, thread designs also impact joint strengths. Unfortunately, today there are no thread standards for vertical turbine column pipes. Every now and then, due to imperfect thread fit, joints fail, and pump column & bowl assembly drop to the bottom of the well. National Pipe Taper (NPT) and BSPT (British Standard Pipe Taper) are generally not recommended for pump settings deeper than 500 feet (150 m). An alternative are oilfield casing threads, much better suited for pump columns going to such depths.

Water well submersible pumps typically use vertical turbine bowl assemblies with a submersible motor on the bottom. The right hand thread pump column pipes tend to be loosened by the reaction torque of the submersible motor. This may unscrew the joints or even cause them to come apart. We have seen instances of vertical straps across the coupling that have been welded to the pipe to prevent unscrewing. We have also seen couplings welded to the pipe on both ends. We recommend against such welds as they distort the pipe and / or the couplings. It is better to tighten threaded joints made from the correct material, with well-designed threads, to the proper torque and to thereby prevent the submersible motor from unscrewing pump column joints. Note that some oilfield submersible bowl assemblies turn in the opposite direction so that the reaction torque tightens the right hand pump column threads.

Finally, the welding of steel or stainless steel casing, especially in larger diameters, must be done properly to prevent the joints from failing downhole. We are aware of a stainless steel weld joint that failed downhole due to poor welding and incorrect selection of the welding rod. Such incidents get very expensive. When the broken weld joint becomes misaligned, the well pump may not pass during installation and sand pumping problems can result too.

Equipment choices and installation

For pump setting depths exceeding 1000 feet (300 m), oilfield submersible pumps are typically recommended. These pumps are more resistant to higher temperatures and offer smaller motor diameters as they operate at 3600 RPM, i.e. at least twice the speed of typical water well bowl assemblies. This choice enables the use of smaller casing and ultimately can result in lower cost wells. However, adequate clearances and tolerances must be maintained.

For example, consider a 2000 foot (610 m) deep well that uses 8-inch nominal casing (216 mm outer diameter) and a submersible pump measuring 40 - 60 feet (12 - 18 m) in length. If the well was drilled crooked or has dog legs, insertion of this pump can run into trouble. Friction may prevent it from being lowered. Forcing it to bend and follow the casing at a minimum may shorten pump life and can result in a pump stuck in the well. Further complications may arise when the pump column joints are of marginal strength and cannot take the force to pull the pump back up.

Another type of difficult-to-recognize problem that we have encountered, involves vertical line shaft pumps set at depths of 400 - 1000 feet (120 - 300 m). Most pump contractors calculate the elongation of the shaft for a particular application to make sure that the bowl assembly can account for the vertical movement of impellers. However, in these deep wells an outdated standard calls for use of Schedule 30 pipe for the pump column which can result in a thin pump column pipe wall having an excessive elongation causing the impellers to interfere in the bowl. In other words, bad things happen where the designers of the pump system oversize the shafting so that its elongation does not match that of the column pipe.

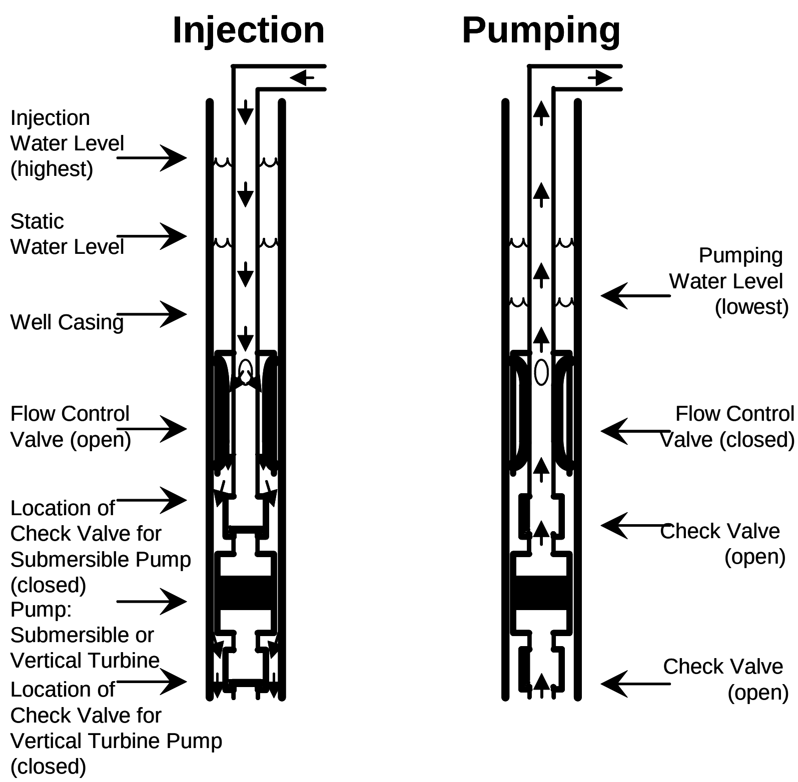
Controlling the well atmosphere

At many locations, ASR taps aquifers that receive little or no natural recharge from rainfall. This is often the case in semi-arid regions. In such cases, water levels can fall to the point where conditions change from artesian to a water table within the aquifer. Gas (air) from the well and formation will fill voids in the unsaturated part of the aquifer. If these are compressible gases such as atmospheric oxygen and nitrogen, they can cause problems as they limit the rise of water levels in the aquifer during injection. One solution for this is to evacuate the annular space between the pump column pipe and the casing which will remove air and draw water vapor into the unsaturated part of the aquifer and the annular space. Now the water level can freely move up and down, as the water vapor simply re-condenses to liquid when water levels rise in the unsaturated portion of the aquifer, rather than being compressed as in the case of air. This approach also can eliminate oxidation and aerobic biological growth in the well casing and aquifer.

Operation

Establishing and following proper operating procedures through written documentation, checklists and training for operators becomes more important as wells get deeper and the stakes higher. An instructive incident involved a 1900 foot (580 m) deep ASR well with a 7-inch (178 mm) pump column pipe, with a FCV set at 1800 feet (550 m), with a deep water level at 1450 feet (440 m) inside a 12 inch (305 mm) casing. The downhole flow control valve was open; the pump was not running, and the operator filled the column pipe at a high rate. This resulted in compression of the air inside the column pipe, forcing it through the FCV and air-lifting the well. Water and air shot out of the well vent at the top of the pitless unit, entered the electrical conduit at the pitless unit, and got into the transformer. Of course, the correct procedure would include closing the downhole FCV and slowly filling the column pipe.

Another type of incident involves running well pumps with a downhole valve at least partially open. As shown in the diagram below, when pumping water the downhole flow control valve needs to be fully closed so that all water is pumped to the surface. Otherwise, water recirculates within the well and heats up - in one case excessively so, damaging both the pump and valve. We have seen three cases of this happening due to improper operation of a manual system or a SCADA system. In this way, these incidents are instructive with respect to both programming for automated operations and manual operation procedures.



Deep wells require, in addition to the normal well head piping, an adequate pressure relief valve. For instance, consider a pump set at 2000 feet (600 m) with a 1500 feet (450 m) pumping water level that can vary. It must exhibit a steep pump curve, which also means that the shutoff pressure for the pump at the surface can be quite high. In order to avoid over-pressurizing the well head piping, a pressure relief valve must be installed upstream of any other valves to effectively protect the system. In this regard, a variable frequency drive (VFD) is highly desirable to ramp up to the desired pumping rates and prevent over-pressurization of the system.

Conclusions

Most ASR wells are still drilled and completed vertically. But for economical reasons, deeper wells generally also need to yield greater pumping and injection rates. Horizontal wells, commonplace in the oilfield, can deliver these. Therefore, we expect to see growing adoption of horizontal wells completions in ASR too. Further out are hybrid completion networks, combining horizontal wells with intersecting vertical pumping holes.

As discussed above, deep wells pose special challenges that require careful consideration to avoid serious problems. However, they also offer special opportunities. For example, during injection one can relatively easily produce electrical power by running the pump in reverse. In fact, the U.S. Department of Energy funded a study[6] that showed how a mixed flow, multi-stage bowl assembly can effectively be used to rotate either a downhole submersible motor or an above ground vertical turbine motor to function as an induction generator to produce electrical power. Baski has built substantial intellectual property in this area and we expect more projects to integrate the use & storage of electricity and water.

We expect these concepts to deliver improved economics, both in terms of lower average unit and total system lifetime costs. But such projects also pose greater challenges in their engineering and execution.

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Australian Guidelines for Managed Aquifer Recharge and their International Relevance

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Abstract Australian national guidelines for managed aquifer recharge (MAR) were approved in July 2009 with the aim of setting scientifically-based criteria and procedures to ensure protection of public health and the environment for all types of MAR projects and water sources and uses. This includes recycling stormwater or treated sewage effluent via aquifers for irrigation, industrial, non-potable household use as well as for drinking water supplies. These guidelines are intended to provide sound foundations for proponents and regulators and thereby to facilitate appropriate investment in MAR. The guidelines are founded on the risk management framework that is used to set Water Safety Plans common to various WHO Guidelines for Water Protection as also followed in Australia's National Water Quality Management Strategy (NWQMS) guidelines of which the MAR Guidelines now form a part.

This paper has two objectives. Firstly it summarises the key features of the guidelines, including their consideration of water quality improvements and deteriorations that can occur during aquifer storage, and the staged approach used for managing risks in the face of uncertainty from the outset of a project. Secondly, it sets out to explore the relevance of these guidelines in different national settings representing countries where MAR has the potential to make significant contributions to water security. In the six countries where the guidelines have been applied, their appetite for data is discussed in settings where there is a constrained ability to investigate, monitor and control water quality and quantity. The entry level assessment in the guidelines was generally applicable at all sites and gave a checklist that addressed all issues needing to be covered at those sites. However, in the second stage maximal risk assessment the value of the assessment was diminished by lack of information at most sites. Virus information in source waters and training in quantitative microbial risk assessment were the largest gaps in knowledge with the most significant impact on health protection at MAR sites. Demonstration projects are warranted to determine circumstances where simplified assessments could be shown to be as safe as the more rigorous approach of the guidelines for wider regional application.

Keywords: groundwater recharge, water quality, contaminants, pathogens, environment protection, groundwater management

Introduction

A deterrent to implementation of innovative managed aquifer recharge projects in a number of countries has been a lack of guidance available for proponents and regulators concerning protection of human health and the environment, in particular aquifer protection. With a widening array of recharge methods (e.g. Tuinhof and Heederick 2003), sources of water (including recycled water derived from urban stormwater or treated sewage) and end uses of water (for economic or environmental benefit) it is important that principles based on science are promulgated for best-practice management. Examples of conflicting requirements of guidance or standards that are not founded on a scientific base are described in Dillon *et al* (2007). It is postulated that science-based guidelines are necessary for consistency in approach among jurisdictions and would be internationally applicable. However it is also recognised that community expectations may differ between countries and jurisdictions and even at a local level within jurisdictions.

Establishing the principles and allowing for adaptation of protection targets based on societal needs and priorities is a way that allows a consistent and transitional approach within jurisdictions in keeping with development and associated health and environmental risks (e.g. spanning less developed to more developed countries) as proposed by Anderson *et al* (2000) for guidelines for water recycling. Similarly differential protection of aquifers (Fig. 1) is advocated within any jurisdiction so that effort is expended commensurate with protection of existing beneficial uses (sometimes termed environmental values). Hence aquifers used as drinking water supplies or supporting pristine groundwater dependent ecosystems would be afforded greater protection than, for example, other aquifers that are too saline to be irrigation supplies.

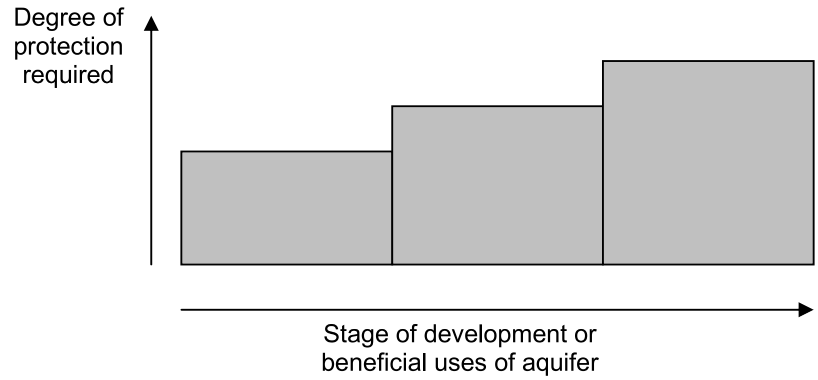

Figure 1. Differential protection policies are advocated so that costs of protection are related to accepted societal exposure to risk and to focus investments on protecting aquifers appropriate to their value.

Characteristics of Science-Based Guidelines
A systematic framework is needed so that scientific knowledge can take its place in influencing MAR operations to ensure health and environmental protection. One example is the framework of twelve elements listed in the Australian Guidelines for Water Recycling (NRMMC–EPHC–AHMRC 2006).

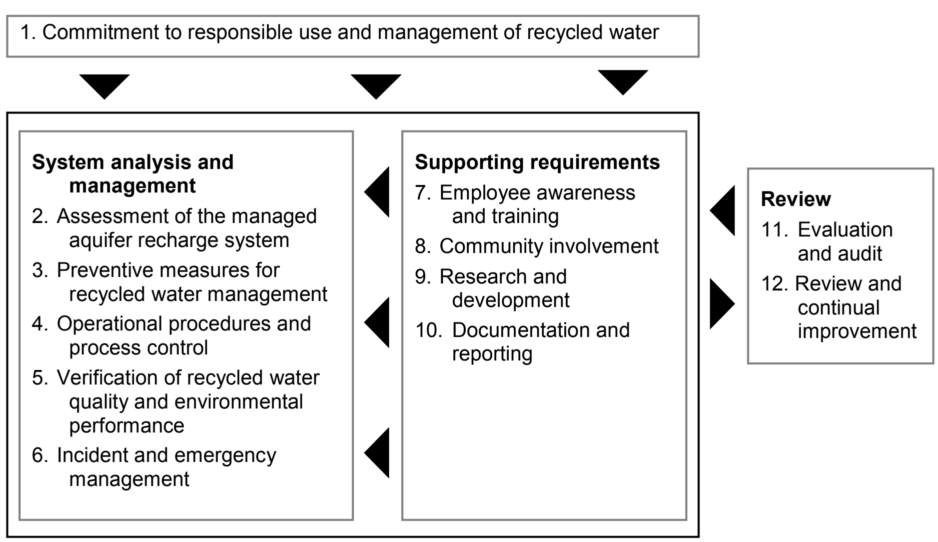

Figure 2. Elements of the framework for managing risk in MAR (*source NRMMC-EPHC-NHMRC 2009a*)

Science has a distinct role to play in system analysis and management, including the supporting research and development, and through review which is a stimulus for improved assessment and operation. In particular scientific input is needed to:

- identify the quality of the source water for recharge and native groundwater, and the hydraulic and mineralogical characteristics of the aquifer and the variations in these variables
- assess the water quality targets to achieve the required level of health and environmental protection for the aquifer and for recovered water
- perform rigorous assessment of exposure to and risk from hazards originating in source water and the aquifer, and from hydraulic considerations, and determine the preventive measures required to achieve targets
- validate the ability of preventive measures, including removal processes in the subsurface, to manage the identified risks, and
- verify that targets are achieved by efficient monitoring and evaluation with feedback into operational practices

Australian MAR Guidelines

In July 2009 the Council of Australian Governments, a composite of state and national governments, approved the Australian Guidelines for Managed Aquifer Recharge (NRMMC–EPHC–NHMRC 2009a) as one of four documents within the Australian Guidelines for Water Recycling (AGWR). The MAR Guidelines deal with all source waters, not just recycled water. AGWR are part of a broader program, the National Water Quality Management Strategy (NWQMS), conceived in the 1990s as a set of principles, and progressively implemented since then through the production of currently 24 national guidelines (most relevant ones listed in references under ANZECC, ARMCANZ, NHMRC, NRMMC). Recent documents all follow the same risk management framework (Fig. 2) that was adopted in the drinking water guidelines, and share the common goal of protecting the ambient environmental values (beneficial uses and ecosystem support services) of water in the environment. The NWQMS Guidelines are not enforceable documents per se, but States normally require adherence with NWQMS Guidelines as part of their enforceable water management regulations.

All water quality criteria to be met for drinking water supplies are taken from the Australian Drinking Water Guidelines (NHMRC–NRMMC 2004) and AGWR Augmentation of Drinking Water Supplies (NRMMC–EPHC–NHMRC 2008). Water quality criteria to be met for all other environmental values, including irrigation, stock water and ecosystem support for each of three classifications of conservation value, are drawn from the Australian and New Zealand Guidelines for Fresh and Marine Water Quality (ANZECC–ARMCANZ 2000a).

The MAR Guidelines allow for staged development of managed aquifer recharge projects, recognising that the information required to allow reliable risk assessment will generally not be available when new projects are first proposed (Fig. 3). Extending beyond water quality issues, the MAR Guidelines also address aquifer pressures, discharges and leakages and hydraulic impacts on groundwater dependent ecosystems (Dillon *et al* 2009b). The guidelines also reinforce the need for public consultation processes particularly where third parties may potentially be impacted by MAR projects.

Australian entitlement and allocation systems for surface water and groundwater (that come under separate legislation at State level) are reinforced in the MAR Guidelines by requiring that water entitlements and allocation plans are addressed before embarking on human health and environmental risk assessments for new MAR projects. The new opportunities and considerations that managed aquifer recharge present water policy makers and the proposed application of National Water Initiative principles (COAG 2004) to address these are described in a National Water Commission report (Ward and Dillon 2009).

The MAR Guidelines allow two processes for risk assessment (Fig. 3). Where MAR projects are small and of inherently low risk (meeting defined criteria), a simplified assessment encourages a blanket permitting process by the jurisdiction. For all other cases an entry level assessment is undertaken to establish the likely viability and degree of difficulty of a project. This is designed to be as simple as possible to enable a proponent to quickly understand the issues to be addressed and the investigations and costs required in order to be able to proceed to project implementation. This allows screening out of improbable projects at an early stage.

Entry Level Assessment

The first part of the entry level assessment *'viability assessment'* deals mostly with water quantity:

1. is there sufficient demand for recovered water?,
2. is there an adequate source of water available for allocation to recharge?,

3. is there a suitable aquifer for storage and recovery of the required volume?,
4. is there sufficient space available for capture and treatment of the water?, and
5. is there a capability to design, construct and operate a MAR project?

Questions on economic viability are beyond the scope of the guidelines. However in subsequent stages the preventive measures required to achieve low risks are determined, and if the proponent considers the costs of implementing these measures as excessive the project is non-viable, as shown in Fig. 3.

If the project is apparently viable, the proponent proceeds to the second part of the entry level assessment *'degree of difficulty assessment'* which is a checklist of 14 simple questions on topics listed below. These help inform on the amount of effort and types of information that will be required in subsequent investigations in order to manage human health and environmental risks at the project location.

1. source water quality compared with groundwater environmental values (water quality)
2. source water quality compared with quality requirements for uses of recovered water
3. source water quality with respect to clogging
4. native groundwater quality compared with quality requirements for uses of recovered water
5. native groundwater quality compared with quality requirements for drinking water
6. groundwater salinity and potential recovery efficiency constraints
7. potential for reactions between source water and the aquifer
8. proximity of nearest groundwater users, connected ecosystems and property boundaries
9. aquifer capacity, confinement and groundwater levels
10. protection of water quality in unconfined aquifers
11. fractured rock, karstic or reactive aquifers
12. similarity to successful projects
13. management capability
14. planning and related requirements

The answers to each specific question indicate whether this attribute has a high, low or unknown degree of difficulty. A high or unknown degree of difficulty suggests more detailed investigation is required for that aspect of the project in stage 2.

Stages 2 to 4

Investigations for Stage 2 will normally involve drilling, aquifer characterisation, establishing the quality of the source water and identifying any preventive measures (including treatment processes) needed to meet water quality requirements. The MAR Guidelines also contain advice (in Chapter 6) on operational issues (such as clogging and recovery efficiency) and their management because in some aquifers these are a tighter constraint on the viability of a project than the criteria to protect human health and the environment (Chapter 5).

If information from investigations reveals that risks can be adequately managed for each potential hazard according to criteria defined in Chapter 5, the project can be constructed at pilot scale or full scale, and commissioning tests performed with validation and verification monitoring. Guidance on monitoring is provided in Chapter 7. Existing knowledge on sustainable attenuation rates in aquifers for pathogens and organic chemicals is provided in Appendices.

Hazards addressed in the risk assessments of the second and subsequent stages are:

1. Pathogens
2. Inorganic chemicals
3. Salinity and sodicity
4. Nutrients
5. Organic chemicals
6. Turbidity and particulates
7. Radionuclides
8. Pressure, flow rates, volumes and levels
9. Contaminant migration in fractured rock and karstic aquifers
10. Aquifer dissolution and aquitard and well stability
11. Impacts on groundwater dependent ecosystems
12. Greenhouse gas emissions

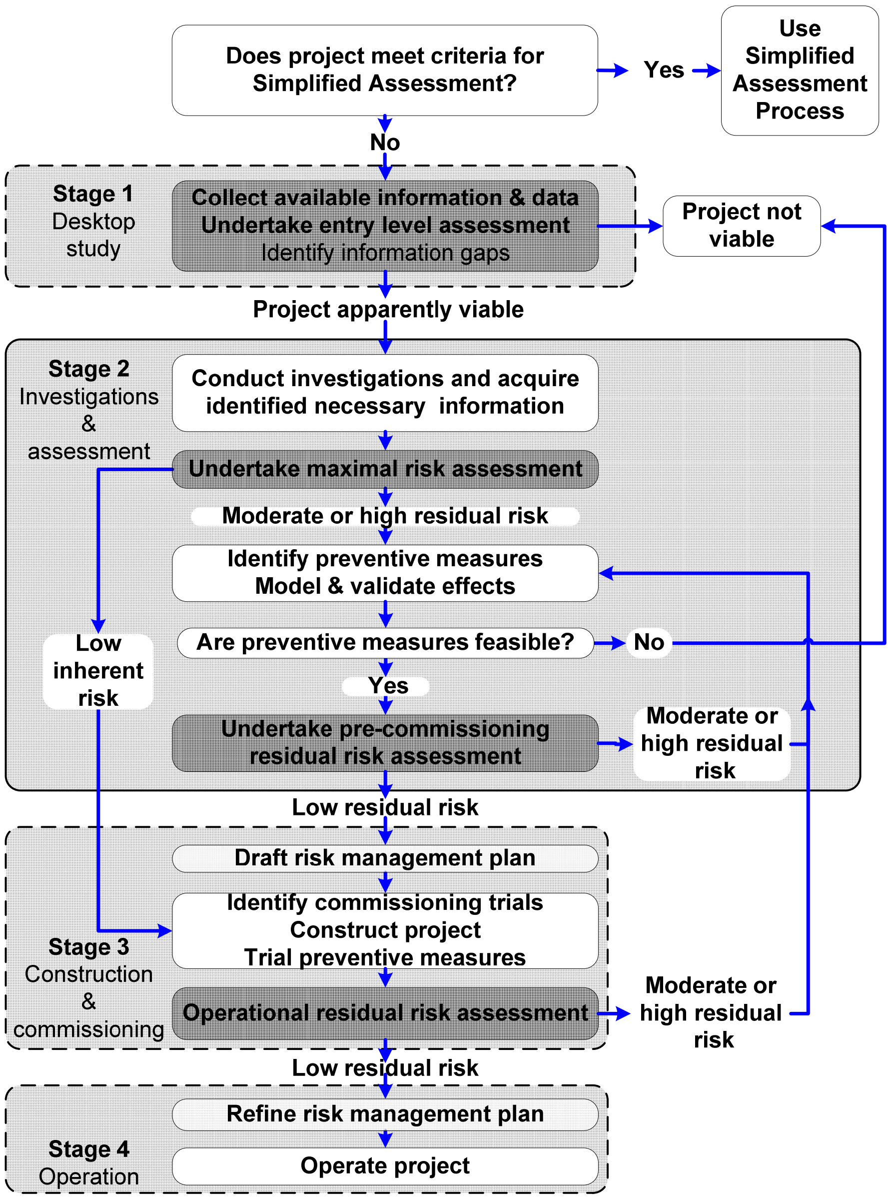

Figure 3. Stages in establishing a MAR project to meet human health and environmental needs in accordance with the Australian Guidelines for Managed Aquifer Recharge (NRMMC-EPHC-NHMRC 2009a)

For each hazard the guidelines describe sources or causes; effects on public health and the environment; means of managing risks, including preventive measures; the proposed validation, verification and operational monitoring; and list the acceptance criteria for the various stages of risk assessment that parallel the stages of project development.

Reactions Between Recharged Water and Aquifers

A distinctive feature of the MAR Guidelines is that they allow for an attenuation zone beyond which at all times, and within which after a defined time, all ambient environmental values (e.g. beneficial uses) of the aquifer are protected. This relies on information concerning inactivation rates of pathogens and degradation rates of degradable organic chemicals.

A simplistic view that treating water to near drinking standards before recharge will protect the aquifer and ensure the recovered water is fit for use is incorrect. For example, chlorination to remove pathogens that would have been removed in a warm aquifer regardless, can result in water recovered from some aquifers containing persistent excessive chloroform. In some locations, drinking water injected into potable aquifers has resulted in elevated arsenic concentrations on recovery due to reactions between oxygen in injected water and pyrite that contains arsenic. Source water that has been desalinated to a high purity dissolves more minerals within the aquifer than water that has been less treated. Similarly, reduction of nutrients to very low levels in recharge water can impede the co-metabolism of some trace organics which otherwise would have been degraded within the aquifer. The American Water Works Association Research Foundation, along with Australian, European and American partners have supported much of the research in this area (e.g. Dillon and Toze 2005; Vanderzalm *et al*. 2009). Consequently the MAR Guidelines adopt a scientific approach that takes into account three ways that aquifers interact with recharged water:

1. Sustainable hazard removal. The guidelines allow for pathogen inactivation and biodegradation of some organic contaminants during the residence time of recharged water in the soil and/or aquifer within an attenuation zone of finite size, where removal is validated.
2. Ineffective hazard removal. These hazards need to be removed prior to recharge because they are either not removed (e.g. salinity) or removal is unsustainable (e.g. adsorption of any metals and organics that are not subsequently biodegraded, or excessive nutrients or suspended solids).
3. New hazards introduced by aquifer interaction (e.g. metal mobilization, hydrogen sulphide, salinity, sodicity, hardness, or radionuclides). There is a need to change the quality of recharge water to avoid these (e.g. change acidity-alkalinity or reduction-oxidation status or reduce nutrient concentrations) or to treat recovered water.

Application of Australian Guidelines to International MAR projects

The MAR Guidelines were designed for use with all types of source waters, all types of end uses and all types of MAR operations by implementing appropriate preventive measures including water treatment (Fig. 4). They have been applied to nine Australian case studies (Page *et al* 2010), results for two of which are given later. Five workshops have been run in Australia and views polled on the utility of the guidelines (Dillon *et al* 2010). Their application to various MAR projects in other countries, including developing countries was tested by coauthors, and the utility and constraints in applying them was reported.

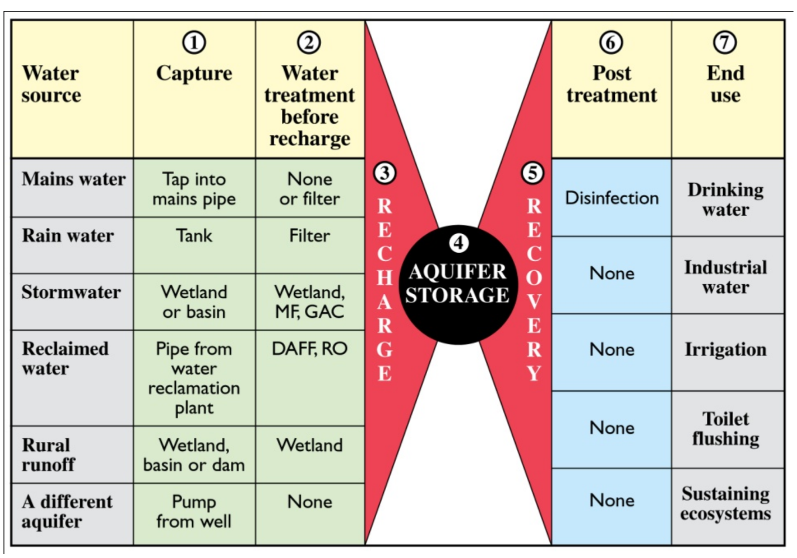

Figure 4. Typical sources of water, methods of capture and pre- and post-treatment for MAR *(from Dillon et al 2009)*. All sources of water, in combination with the appropriate treatment, can be recovered from the aquifer for any end use. This applies for all methods of recharge *(see Dillon 2005)*. Note actual pre-treatments and post-treatments will be project-specific to achieve effective management of risks and clogging (MF is microfiltration, GAC is granular activated carbon filtration, DAFF is dissolved air flotation and filtration, RO is reverse osmosis).

The guidelines were applied to case studies in China (2), India (2), Jordan, South Africa and the United States of America by researchers or water utility managers actively involved in those projects. For five sites the guidelines were applied retrospectively to existing MAR operations: groundwater dam at Longkou, China (Wang *et al* 2010); a dug well recharge scheme in Gujarat, India (Jain 2010); a check dam (percolation tank) at Kodangipalayam in Tamil Nadu India (Gale *et al* 2006, Dillon *et al* 2009); an infiltration system at Altantis, South Africa (Tredoux et al 2002, Cave and Tredoux 2002); and water spreading at Rio Hondo, USA (Los Angeles County Department of Public Works, 2010). The guidelines were also applied to four sites that are undergoing investigations or commissioning trials: soil aquifer treatment (SAT) at Alice Springs, Australia (Miotlinski *et al* 2010); aquifer storage transfer and recovery (ASTR) at Salisbury, Australia (Page *et al* 2009); ASTR at Jinan, China (Wang *et al* 2009); and an infiltration basin at Wadi Kafrein, Jordan (Sawarieh *et al* 2009; Zemann et al 2009). Results of stage one entry level degree of difficulty assessment and stage two maximal risk assessments are condensed in Table 1.

Those applying these guidelines for the first time reported that they found the entry level assessment easy. The assessment did not reveal any issues that the proponents had not already considered, nor did it fail to address any issues that the proponents had considered. Several proponents reported that the entry level assessment and the maximal risk assessment provided check lists that they thought would be useful for assessing future projects. No proponents recorded any difficulty in undertaking the entry level assessment, which is in keeping with the objective in developing the guidelines to provide a very simple starting point based on minimal information.

However, completing the maximal risk assessment revealed that all project reviewers had difficulty in assessing risks to human health, particularly from pathogens that were likely to be present in source water for recharge. There were several reasons for this. Firstly, there is a lack of data on the viral content of source waters originating from streams, urban stormwater or water reclamation plants. Furthermore, proponents generally did not feel confident in their ability to perform a reliable quantitative microbial risk assessment, or in making conservative assumptions about source water quality.

Possibly because all the sites had hydrogeological investigations data available, the level of uncertainty in the fate of recharged water was relatively well defined, with the natural exception of recharge to the karstic aquifer beneath Jinan. Factors affecting the relevance of the guidelines were predicted to be:

1. the ability to establish accepted environmental values for an aquifer and hence water quality requirements for aquifer protection
2. variability of source water quality and the capability to measure water quality parameters of the most importance for risk assessment (including human pathogens and micropollutants)
3. ability to acquire information on aquifer hydraulic and geochemical properties
4. ability to measure attenuation rates in the soil-aquifer system for the hazards of highest concern
5. reliability of power supplies, preventive measures and controls
6. ability to invest in monitoring

It is not possible to dismiss these factors based on the relatively few cases studied, and it is suggested that further applications may reveal whether further simplification of the Stage 2 risk assessments can be made that both protect human health and the environment and reduce investigations and analysis costs. Success of MAR operations is important and for the immediate future, it is suggested that further case studies are used to form an evidence-based approach to determining circumstances where it is possible with less effort in investigations to reliably protect health and the environment commensurate with local standards (as proposed by Anderson *et al* 2000).

Conclusions

Australian Guidelines on Managed Aquifer Recharge now provide a science-based approach to assure protection of public health and the environment. These guidelines are intended to give more certainty to risk assessments and project approval processes, prevent failure of managed aquifer recharge projects and uphold the confidence of investors, regulators and the public in future innovations in MAR projects for water supply and groundwater protection. The MAR Guidelines allow for logical and efficient stage-wise development of projects commensurate with risks, account for water quality changes in aquifers, provide for water quality and pressure effects in aquifers and connected ecosystems, address greenhouse gas emissions and allow for monitoring to inform continuous improvement in their application.

When the MAR Guidelines were applied to nine MAR projects in six countries it was found that they appeared to perform consistently with existing practice at entry level stage and therefore provided a systematic check list for project initiation. There were difficulties encountered in the second stage risk assessment due to the need for data that may not be available, and in particular, virus numbers in source water and viral attenuation rates in the aquifer and other engineered treatments. This suggests that the guidelines may have identified a common weakness in investigations at MAR sites, but further assessment is necessary. There is the possibility that a simplified version may be more useful in remote and rural areas of developing countries, but such analysis should be performed making use of local data at demonstration projects and in the context of local health standards. As a general principle, MAR should never be used to experiment with public health, but with adequate safeguards may provide a low cost means of improving the safety of drinking water.

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The development of managed aquifer recharge guidelines was supported by the National Environment Protection Council and the National Water Commission. The task of writing the guidelines was overseen by a Working Group consisting of members of Commonwealth and State Authorities with an active interest in governance of MAR, and who reviewed drafts and provided extensive advice. The Working Group, chaired by Leon English, reported to the Joint Steering Committee responsible for the Australian Guidelines for Water Recycling, chaired by Chris Bell and later John Williamson, which reported to three Ministerial Councils of Commonwealth and State Governments: the Environment Protection and Heritage Council, the Natural Resource Management Ministerial Council, and the National Health and Medical Research Council. Secretarial support of Haemish Middleton and Ian Newberry of National Environment Protection Council Service Corporation is gratefully acknowledged. Comments of eighteen respondents in the public consultation process, and two expert reviewers, Prof Jorg Drewes and Dr Jack Schijven, are sincerely appreciated. The authors gratefully acknowledge helpful reviews of a draft of this paper by Kerry Levett and Konrad Miotlinski of CSIRO Land and Water.

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Table 1 (This table is not to be included in paper but is used to assemble information). Australian Guidelines were applied to existing MAR projects or to projects in development to allow an opportunity for feedback on their utility elsewhere

MAR project type and location	Source water and water treatment before recharge	Post-recovery treatment (if any) and end use of water	Particular health or environ-mental issues of potential concern	Degree of Difficulty Assessment																	Stage 2 Maximal Pre-commissioning Risk Assessment														Preventive measures required to reduce all residual risks to low
				1. source cf g/w env values	2. source cf end use	3. source cf clogging	4. g/w cf end use	5. g/w cf drinking water	6g/w and recovery efficiency	7. reactions in aquifer	8. proximity to sensitive uses	9. aquifer storage capacity	10. unconfined aquifer for drinking	11. fractured, karstic, reactive aq	12. preceding projects	13. management capability	14. planning requirements	No. with high degree of difficulty	No. with unknown deg. of diff		1. pathogens	2. inorg chems	3. salinity	4. nutrients	5. org chems	6. turbidity	7. radionuclides	8. pressure, levels, vols	9. contam fractured/karstic	10. aquifer dissolution	11. GD ecosystems	12. greenhouse gases	No. with high maximal risk	No with uncertain max. risk	
				1	2	3	4	5	6	7	8	9	10	11	12	13	14	H	U		1	2	3	4	5	6	7	8	9	10	11	12	H	U	
Soil aquifer treatment at Alice Springs, NT, **Australia**	sewage effluent / 2ndary treatment, dissolved air flotation	None / irrigation	Nutrients, Groundwater levels	L	L	H	U	L	U	U	L	U	L	U	H	L	U	2	6		H	L	U	U	U	L	L	U	L	L	U	U	1	6	DAF treatment, Clogging management, Irrigation management
Aquifer storage transfer & recovery at Salisbury, SA, **Australia**	Urban stormwater / Wetland filtration	None and to be determined / Irrigation, industry and potentially drinking	Pathogens, Inorganic chemicals, Salinity, Organic chemicals, Turbidity, Aquifer dissolution	L	H	U	H	L	H	H	U	U	L	H	L	U	U	5	5		H	H	H	L	H	H	L	U	L	U	L	L	5	2	Source control, Reedbed & aquifer treatment, UV, chlorination, iron removal
Groundwater dam at Longkou, Shandong Province, **China**	Stormwater and river water / Filtration	None / Irrigation, industry and drinking water	Salinity, Inorganic chemicals	L	L	L	L	L	L	L	L	L	H	L	L	L	L	1	0		U	L	H	H	L	L	L	L	L	L	L	L	2	1	Aquifer treatment, Irrigation management Clogging management

Table 1 (continued) MAR project type and location	Source water and water treatment before recharge	Post-recovery treatment (if any) and end use of water	Particular health or environ-mental issues of potential concern	Degree of Difficulty Assessment																	Stage 2 Maximal Pre-commissioning Risk Assessment														Preventive measures required to reduce all residual risks to low
				1	2	3	4	5	6	7	8	9	10	11	12	13	14	H	U		1	2	3	4	5	6	7	8	9	10	11	12	H	U	
Aquifer storage transfer & recovery Jinan, Shandong, **China**	Urban stormwater / Filter and GAC	t.b.d. / Potentially drinking water	Inorganic chemicals, Aquifer dissolution, Contaminant migration in - karst aquifer	L	L	L	L	L	L	H	L	L	H	H	L	H	U	4	1		L	H	L	L	U	H	L	L	H	U	U	L	3	3	Source control and treatment, Clogging management aquifer protection
aquifer storage and recovery via irrigation dug wells in Gujarat, **India**	Rainfall runoff from agricultural fields / siltation chamber & filtration	No treatment for agricultural use	Total suspended solids	L	L	H	L	L	L	L	L	L	L	U	H	L	L	2	1		U	L	L	H	H	H	U	L	L	L	L	L	3	2	Data not available to complete residual risk assessment
Percolation tank, Karanam-pettai, Kodangipala-yam, Tamil Nadu, **India**	River water mixed with treated effluent	No treatment for agricultural use / *occasional drinking*	Total suspended solids	L	L	H	L	L	L	L	L	L	L	U	U	L	L	1	2		-	-	-	-	-	-	-	-	-	-	-	-	-	-	Not assessed at stage 2
				H	H	H	L	L	L	U	L	L	H	H	U	U	L	5	3																
Percolation tank with irrigation releases at Wadi Kafrein, **Jordan**	Rainfall runoff water / stormwater	No treatment for agricultural use	Pathogen loads, emerging pollutants, sedimentation	H	L	H	L	L	L	H	H	L	H	H	L	L	H	6	1		U	L	L	H	H	U	U	L	L	U	U	U	2	6	Monitor fate of pharm-aceuticals

Table 1 *(continued)* MAR project type and location	Source water and water treatment before recharge	Post-recovery treatment (if any) and end use of water	Particular health or environ-mental issues of potential concern	Degree of Difficulty Assessment																	Stage 2 Maximal Pre-commissioning Risk Assessment														Preventive measures required to reduce all residual risks to low
				1	2	3	4	5	6	7	8	9	10	11	12	13	14	H	U		1	2	3	4	5	6	7	8	9	10	11	12	H	U	
Soil-aquifer treatment recycling for potable use. Atlantis, Cape Town, **South Africa**	Secondary treated domestic sewage + urban stormwater Reedbed filtration	Softening (weak base ion exchange), chlorination / Drinking water	Salinity, DOC, turbidity & particulates	H	H	H	L	H	M	M	H	H	H	M	H	H	H	10	0		H	H	H	L	L	H	L	H	L	H	L	L	6	0	Source control, Reedbed, Basin design, Eh control Fe removal
Rio Hondo spreading grounds, California, **USA**	Stormwater mixed with reclaimed water	No treat-ment for agricultural use	Pathogens, emerging pollutants, sediment	L	H	L	L	U	L	U	L	L	L	L	L	L	H	2	2		U	L	L	L	L	H	L	L	L	L	L	L	1	1	Existing stormwater & reclaimed water controls

Aquifer Storage Recovery: Economics and Recent Technical Advances

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Abstract

The relative cost for storing water underground through ASR wells compared to other water management options has been poorly understood for many years, inhibited by the lack of comparative data. Typical 2010 unit capital cost in the United States is $300,000 per million liters per day (MLD) (this corresponds to $0.30 per lpd, or $1.14 per gallon per day [gpd]) of recovery capacity, within a range of $270,000 to $340,000 per MLD. Significant contributing factors include well yield, depth and number of wells. Australia reported a unit cost averaging of $1,070 per ML recharged.

Once the viability of ASR has been demonstrated at any new location, marginal cost pricing of water during both recharge and recovery becomes possible, providing a new opportunity for cost-effective water management and a valuable new tool to help resolve intergovernmental disputes regarding water ownership and control.

Several important ASR technical and regulatory advances have occurred recently in the United States, each contributing to reducing the cost of ASR and increasing its general applicability. Among these, the "Target Storage Volume" concept is increasingly accepted as an economical and viable tool for ensuring acceptable recovered water quality, particularly for water storage in aquifers containing brackish or poor quality groundwater, and in aquifers containing metals such as arsenic. Significant advances have occurred recently in federal and state ASR regulatory programs.

Keywords: aquifer storage recovery; economics; managed aquifer recharge; marginal costs; target storage volume, wells

INTRODUCTION

The global need for storing more water to achieve goals of water supply reliability and sustainability is increasingly recognized. However uncertainty has often existed with regard to whether it is more cost effective to achieve these goals through storage of water aboveground in surface reservoirs or below ground through wells or surface recharge facilities. These considerations are independent of other important factors for most projects such as technical feasibility, overall viability, and environmental impact.

ASR wells store water underground in suitable aquifers through wells when water of suitable quality is available for recharge. The stored water is recovered from the same wells when needed. ASR wells comprise an important part of Managed Aquifer Recharge (MAR). A 2009 inventory conducted by the U.S. Environmental Protection Agency (EPA) in the United States indicated over 500 ASR wells in operation at more than 95 wellfields in twenty states.

Except where otherwise indicated, all costs presented in this paper are in US Dollars, adjusted to June 2010 using standard cost indices.

METHODS

USA Analysis of ASR Economics

Florida ASR Wellfields

A recent study of ASR economics compiled capital and operating cost data for nine ASR wellfields in Florida. This was prepared by the author and an associate, Mark McNeal P.G./ ASRus, and was included as a chapter in a larger report on water supply economics (CDM, 2007). Capital costs were identified for three wellfield components: ASR wells, monitor wells and wellhead facilities, all constructed since 1999. All costs have been adjusted to a common cost basis using commonly accepted cost escalation methods (ENR Construction Cost Index of 8805, June 2010). Unit capital costs are expressed in terms of U.S. dollars per megalitre per day of recovery capacity ($/ MLD). Costs are also shown in parentheses as U.S. dollars per million gallons per day of recovery capacity ($/MGD). ASR well depths ranged from 122m to 370m. Individual well recovery capacities ranged from 3.8 to 18.9 MLD. The number of wells in each wellfield ranged from one to 12. The principal factors affecting unit capital costs for the new ASR wells were primarily well yield and secondarily well depth, number of wells, and the requisite amount of data collection and testing during well construction. Single well projects have higher unit capital costs than multiple well projects. Deeper wells are more expensive than shallow wells, and high capacity wells are more cost-effective than low yield wells. Projects requiring extensive monitoring and reporting are substantially more expensive than others with reduced monitoring requirements. Unit capital costs are summarized in Table 1.

Table 1 – Unit Capital Costs

Construction Cost Item	($/ MGD)	$/ MLD
ASR Well Construction Costs		
Average	$245,000	65,000
Range	$66,000 to $418,000	
Monitor Well Construction Costs		
Average	$242,000	64,000
Range	$46,000 to $771,000	
ASR Wellhead Facilities Construction Costs		
Average	$508,000	134,000
Range	$191,000 to $1,111,000	
Total ASR Construction Costs		
Average	$1,079,000	285,000
Range	$346,000 to $2,052,000	

Operating costs were available for only four of the twelve ASR wellfields. These are expressed in terms of $/ year/ MLD of recovery capacity, and also in terms of $/ ML. Volumes estimated to be recovered annually from the four wellfields ranged from 378 to 3,785 ML, averaging 2,100 ML. Recovery durations were typically 90 days. The reason for presenting unit operating costs both ways is that ASR wellfield projects typically store and recover a wide annual range of water quantities, from the small quantity required to meet a peak weekend demand each year (about five days for Wildwood, New Jersey) to the much larger quantity required to meet needs during an extended drought (up to about 180 days for the Peace River/Manasota Regional Water Supply Authority, Florida). Unit operating costs based on cost per unit volume can be very high if the recovered volumes are low, and vice-versa. Both types of projects may be quite cost-effective compared to other peak water supply alternatives, however if the unit costs are compared with much lower unit costs from conventional water sources and treatment facilities that operate throughout the year, incorrect conclusions may easily be drawn regarding the relative cost-effectiveness of ASR. Care is required to compare “apples to apples.” Both methods of comparison are valid, but for different reasons.

Annual operation and maintenance costs indicated a range of $16,000 to $46,000 per year per MLD of recovery capacity ($61,000 to $173,000 per year per MGD), averaging $28,000 per year per MLD of recovery capacity ($106,000 per year per MGD). Recovery capacities ranged from 4 to 45 MLD (1 to 12 MGD). When compared on the basis of $/ML, the values averaged $320/ML, within a range of $209 to $513/ML.

The same report (CDM, 2007) also presented surface reservoir unit capital costs and operating costs. This was based upon eight existing or planned surface reservoirs in Florida, ranging in storage capacity from 7,350 ML to 234,000 ML. Total project surface area for each of the reservoirs ranged from 397 to 6,643 hectares (980 to 16,414 acres), including the reservoir area and surrounding buffer property. Unit capital costs ranged from $2,435 to $17,697 /ML ($1,975 to $14,353/acre ft), averaging $7,042/ ML ($5,711/ acre ft). Unit operation and maintenance costs ranged from $15 to $297/ ML, averaging $122/ ML ($12 to $241/acre ft, averaging $99/acre ft).

Both surface reservoirs and ASR wells are proven, viable technologies for water storage, each offering certain benefits and costs. In many cases the best water storage solution is a combination of the two technologies working together, not one or the other. This takes advantage of the positive aspects of each technology. Usually a much greater volume of water can be stored and recovered while the overall cost is greatly reduced.

To achieve a given storage volume such as 5000 ML, these unit capital costs may be utilized to compare construction costs of surface reservoir storage and ASR storage. Based on the Florida economics data, an average surface storage reservoir would cost about $35 million. ASR storage cost would depend upon the installed recovery capacity. Assuming typical recovery duration of 90 days the installed recovery capacity would be 56 ML/D and the associated capital cost would be $16 million. ASR construction cost would vary depending upon the intended duration of recovery and the associated number of wells.

San Antonio Water System Twin Oaks ASR Wellfield

A recent study (Malcolm Pirnie, 2010) of ASR historic operations, including capital and operating costs, was prepared by the author for the Twin Oaks ASR Facility, San Antonio Water System (SAWS), San Antonio, Texas. With 227 MLD recovery capacity, this is the third largest ASR wellfield in the United States, behind Las Vegas Valley Water District, Nevada (594 ML/D), and Calleguas Municipal Water District, California (257 MLD).

As of March 2008, SAWS had invested approximately $238 million to construct the ASR facility, thereby avoiding the need to spend $600 million for an alternative water supply, buying Edwards Aquifer water rights. The major capital cost items are shown in Table 2.

Table 2 San Antonio Water System ASR Wellfield Capital Costs

Cost Item	Cost ($ million)
Transmission pipelines	92.1
Water treatment plant	56.2
ASR wellfield facilities	46.8
Wellfield mitigation program	5.3
Land acquisition	8.4
Engineering, legal, permitting project management, etc.	29.2
Total	US$238.2

There are about 68 km (42 miles) of large diameter transmission pipelines capable of conveying 227 MLD (60 MGD) of water to and from San Antonio to the wellfield, a straight line distance of about 48 km (30 miles), plus additional piping within the wellfield. Water treatment plant facilities include ground storage reservoirs, 114 MLD (30 MGD) treatment facilities for groundwater produced from the Carrizo-Wilcox aquifer, 227 MLD (60 MGD) chlorination facilities, and a 227

Ml/D (60 MGD) pumping station. The wellfield includes 29 ASR wells and three Carrizo-Wilcox aquifer production wells. Facilities were constructed in two phases, the first of which was completed in 2004 providing 114 MLD (30 MGD) recovery capacity and the second was completed in 2006, expanding capacity to 227 MLD (60 MGD).

As shown in Table 2, ASR wellfield facilities are estimated to have cost about $52.1 million, including a mitigation program to resolve any perceived adverse impacts upon water levels beneath surrounding agricultural lands. With 227 MLD (60 MGD) design recovery capacity, this equates to an “unadjusted” unit cost of $230,000/ MLD ($0.87 per gpd) of recovery capacity. Updating these unit costs to a June 2010 basis for comparison using commonly accepted construction cost indices, the current unit cost is about $270,000/ MLD ($1.04 per gpd). It is pertinent that the ASR wellfield cost is a relatively small component of the total capital cost, reflecting the considerable distance from San Antonio to the ASR wellfield, and the associated piping and pumping costs. Furthermore, experience to date suggests that operation of the water treatment plant will generally prove to be unnecessary if, as anticipated, ASR operations can be conducted in such a manner as to minimize or avoid recovery of Carrizo aquifer water, which tends to have low pH and elevated concentrations of hydrogen sulfide, iron and manganese.

Operating costs for 2009, after about five years of operations, totaled $2.6 million. This works out to approximately $11,361/ year/ MLD ($43,000 per year per MGD) of recovery capacity. Since some of the wellfield equipment is still within its warranty period, future actual operating costs may be slightly higher than this. For planning purposes it would be appropriate to budget comparable operations at about $12,000/ year/ MLD ($45,000 per year per MGD) of recovery capacity. This estimated operating cost most likely reflects not only operation and maintenance of the ASR and Carrizo-Wilcox production wells but also the water treatment plant, pump station, ground storage and transmission facilities associated with the ASR wellfield, plus the wellfield mitigation program. ASR experience at other wellfields suggests that an annual operating cost of about half this amount is appropriate for budgeting purposes, within a range of plus or minus 50%.

Beaverton, Oregon

As reported at ISMAR6 in Scottsdale, Arizona (Eaton and Winship, 2007), the City of Beaverton, Oregon, has had an active ASR wellfield in operation since testing began at the initial well in 1999. Experience has shown that this is a cost-effective water storage alternative compared to other potential water supply sources. Three ASR wells currently store drinking water in a basalt aquifer, providing 23 MLD (6 MGD) of recovery capacity to help meet peak summer demands. Capital costs were presented. Updating these capital costs to June 2010 dollars yields an equivalent capital investment totaling $7.82 M, or $340,000/ MLD ($1.30 per gpd) of recovery capacity. Total annual ASR operation and maintenance cost for storing an estimated 1,892 ML (500 MG) was $417,303, or $221/ ML ($70,000 per MGD) of recovery capacity. This also equates to $18,375 per MLD of recovery capacity.

Australian Analysis of ASR Economics

Great progress has been made in Australia with the development of MAR science and technology. A recent report (Dillon et al., 2009) addressed ASR economic issues and provided economic comparisons to other water supply alternatives. Unlike in the USA where most ASR projects store treated drinking water for urban water supply, usually to meet seasonal peak demands, ASR applications in Australia are mostly for aquifer recharge with stormwater and reclaimed water to meet non-potable and indirect potable needs. The primary focus in Australia is on aquifer recharge. Recovery of the stored water is the ultimate goal however recovery costs and such issues as recovery efficiency are not directly addressed in the economic comparison.

Twelve ASR projects in Australia were compared on the basis of “levelised costs,” defined as “the constant level of revenue necessary each year to recover all the capital, operating and maintenance expenses over the life of the project divided by the annual volume of supply." The working life of ASR wells was assumed to be 15 years, which is low compared to life cycle assumptions in the USA (30 to 50 years). All costs were originally adjusted to 2007/2008 values using the Reserve Bank of Australia Consumer Price Index for Goods and Services, and then updated to June 2010 and converted to US dollars assuming an exchange rate of 1 AUD = 0.9 USD. Combined recharge capacity for these 12 locations was 6,330 ML/year with individual projects ranging from 15 to 2,740 ML/year, averaging 528 ML/year. Eight sites were in limestone aquifers; three sites in fractured rock aquifers, and one site in alluvium. Ten sites stored stormwater while two sites stored reclaimed water. Maximum injection rates per well for all sites averaged 13 lps (1.1 MLD), within a range of 3.5 to 40 lps (0.3 to 3.5 MLD)(0.1 to 1 MGD).

Levelised costs ranged from US$2.86/kL to US$0.95/kL, declining as the recharge volume increased from 15 to 2,000 ML/year. Average cost was US$1.07/kL. Projects between 15 and 75 ML/year do not benefit as much from economies of scale. Land costs were not included and are typically quite small for ASR projects. Investigation costs averaged 11% of levelised costs and varied according to the complexity of the project. Comparison with alternative urban water supplies show levelised costs for stormwater ASR wells are typically 30 to 46% of the costs for seawater desalination and that ASR uses 3% of the energy.

The Australian report compared ASR wells to other water storage options on the basis of unit capital costs, concluding that ASR wells provide storage at $3,600 to $9,000 per ML. This is one to four percent of the cost of tank storage, however ASR wells occupy less than 0.5% of the land surface area. Of considerable interest is that ASR wells were found to have similar unit capital costs to lined earthen dam impoundments but occupy less than 0.2% of the land surface area. The reason for this probably relates to the "levelised cost" approach for comparison between alternatives. The storage volume range for the large dams ranges from 350 to 200,000 ML whereas for the ASR wells it ranges from 75 to 2,000 ML. The annual volume of supply is significantly different between the two storage options, tending to bias the cost comparison in favor of projects that store larger volumes and against smaller projects that may be more cost-effective for meeting seasonal peak demands. Both methods of cost comparison are valid for different reasons and applications.

MARGINAL COST PRICING

Once confidence in ASR viability and cost-effectiveness at a particular site has been achieved, it is appropriate to consider marginal cost pricing of water, either for recharge or for recovery. The marginal cost price of water for recharge typically includes only the cost for electrical power, chemicals and residuals disposal, as required to produce one more unit volume of water during off-peak times. Typical marginal costs for water utilized for aquifer recharge are in the range of $40/ML to $92/ML ($0.15 to $0.35/kgal).

Where wholesale purchase of water is required from a regional supplier, the marginal cost for water supply would include the purchase cost of the water utilized for storage. In many cases it may be possible to negotiate a reduced cost for water purchase during off-peak months. Precedent exists at a few locations for a 50% discount.

The marginal value of the recovered water during peak months may be quite high. At such times the ability to recover water at high rates for a short period of time may perhaps be able to delay or eliminate the need for construction of costly transmission pipelines, water treatment plants, pumping stations, etc, Unit capital costs for such facilities may be several times the cost of the ASR wellfield facilities.

The difference between the marginal cost of water utilized for recharge and the marginal value of water available for recovery provides a substantial business opportunity for governmental and private entities to capitalize on the potential cost savings.

RECENT TECHNICAL AND REGULATORY ADVANCES

Perhaps the most significant advance in ASR development in the United States since ISMAR6 in 2007 has occurred as a result of recent efforts by the federal government (U.S. EPA) to try to gain an understanding of ASR regulatory issues in different states. With the rapid nationwide growth of ASR and the emergence of substantial differences between states as to how ASR wells are regulated, EPA has tentatively determined that it is consistent with current Underground Injection Control (UIC) laws and regulations that measurement of compliance with drinking water standards may be measured at a suitably-located monitor well, not at the ASR well during recharge. Furthermore, adequate time may be allowed for natural physical, geochemical and microbial processes to occur in the subsurface that would enable the stored water to comply at the monitor well with drinking water standards. Taken together, these two positions have significantly opened up the opportunity for storing water underground without the need for incurring what would otherwise be very substantial pretreatment costs such as for deoxygenation of recharge waters to inhibit arsenic mobilization, or disinfection to kill or inactivate low concentrations of microbiota.

The Target Storage Volume (TSV) concept, developed by the author (Pyne, 2005, 2007), is now increasingly accepted by regulatory agencies nationwide as a viable and cost-effective tool for achieving high recovery efficiency while storing fresh water in brackish and saline aquifers, and other aquifers containing poor quality water. Instead of requiring multiple cycles over a period of several years to develop the TSV, it is more efficient to add this water to the well immediately after well construction and prior to cycle testing. A much smaller, shorter and more cost-effective cycle testing program is then implemented.

The TSV is also a useful and viable approach for storing water in aquifers containing minerals such as arsenopyrite that can release metals such as arsenic into the recharge water or into the recovered water. Effective implementation of the TSV concept can be utilized at many, but not all, sites to manage the mobilization and attenuation of arsenic so that it does not occur either in the recovered water or in the aquifer around the well at significant distances. This is based upon analysis of data from 52 ASR wells and 41 storage zone observation wells at 12 ASR wellfields in Florida that have been operational for up to 25 years. A strong relationship is evident between recovered water arsenic concentrations and cumulative storage volume, typically with r^2 values exceeding 70%. Those wells in early stages of cycle testing and with little water volume stored, or that recover all of the stored water in each operating cycle, tend to have high arsenic concentrations. Wells that have been operational for several cycles and have built up a large storage volume typically do not experience elevated arsenic concentrations. The relationship is generally applicable except for sites where significant upconing of poor quality water occurs during ASR recovery. Arsenic concentrations tend to attenuate with increasing storage volume, increasing number of storage cycles, distance from the ASR well and also with storage time. This also assumes that design and operation of the ASR well does not allow cascading of water into the well, with associated air entrainment. It is postulated that the arsenic usually does not move very far from an ASR well, typically a few hundred feet, and is instead sorbed onto ferric hydroxide floc that is trapped in the pore spaces of the aquifer close to the well. The arsenic stays sorbed unless there is a significant change in ambient water quality, such as may occur if recovery brings ambient groundwater too close to the well. It is necessary to form and maintain a buffer zone around the well in order to effectively control arsenic concentrations. The buffer zone is a portion of the TSV. (Pyne, 2004, ASR Systems 2007, 2008)

The scale of ASR programs is tending to become larger. Existing smaller programs are being expanded and new programs are being planned to address not only local but also regional water management needs. Increasingly it appears that environmental considerations are driving ASR programs, thereby restoring river flows during dry months and droughts while storing large volumes of drinking water underground to ensure reliable and sustainable water supplies for urban areas. Consideration is being given at some locations to augmenting low flows from ASR storage in addition to meeting urban needs. Concerns regarding climate change are also driving

ASR programs, storing water underground when it is available for recovery when needed. The outlook is bright for ASR as an important element of Managed Aquifer Recharge.

CONCLUSIONS

Analysis of ASR economics based upon capital and operating cost data from 11 different ASR wellfields in the United States and 12 in Australia, suggests the following. All costs are in U.S.A. dollars adjusted to June 2010.

- Unit capital costs for ASR wells and wellhead facilities in the United States average about \$300,000/ MLD (\$1.14/ gpd) of recovery capacity, within a range of \$270,000 to \$340,000 per MLD of recovery capacity.
- Less data is available for operation and maintenance costs in the United States. Average annual costs are \$12,000 to \$28,000 per MLD of recovery capacity, averaging \$16,000 per MLD of recovery capacity (\$60,000 per year per MGD of recovery capacity).
- Unit costs for ASR facilities in Australia averaged \$1,070 per ML recharged, within a range of \$2,860 declining to \$950 per ML as the recharge volume increased from 15 to 2,000 ML/year.
- It is not easy to compare the US and Australian data since different measurement criteria are utilized. US ASR projects are typically evaluated in terms of unit costs for recovery capacity to meet peak demands while Australian projects are evaluated in terms of unit costs for annual aquifer recharge volume.
- Marginal cost pricing of water, whether recharged into ASR wells or recovered from ASR wells, can facilitate forward progress with projects to store water underground and thereby help to resolve complex and expensive water management issues.
- During ASR recharge, measurement of compliance with drinking water standards should be measured at one or more suitably located monitor wells, providing adequate time and distance for natural physical, geochemical and microbial processes to occur underground.

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MAR Technical, Regulatory and Policy Challenges, Barriers and Evolving Solutions in the United States

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ABSTRACT
Globally, there is wide recognition of increasing environmental needs for surface water resources, projected effects of climate change including the timing and availability of surface water to meet demands, and the realization that few surface water reservoirs will be constructed in the future. The result in the United States (US) is increased emphasis on managed aquifer recharge, which is being met with a mixture of successes, and technical, regulatory, political challenges and barriers. Arsenic in Florida and disinfection byproducts in California have been barriers for implementation of aquifer storage and recovery (ASR) projects, and coliform bacteria in stormwater recharge has also been a challenge. In an attempt to develop solutions, a number of federal, state and private initiatives are ongoing. Acknowledging the groundwater industry needs, the US Environmental Protection Agency held a meeting of ASR experts in Chicago to start looking for solutions to regulatory challenges to ASR, and has been continuing that dialogue through participation at public meetings and individual meetings with industry. EPA has initiated national rulemaking to establish a program to reduce stormwater discharges and make regulatory improvements to its stormwater program. The American Ground Water Trust has been the leading proponent of ASR over the past decade through conferences, publications and recently developed a video on ASR. The Groundwater Protection Council has also been leading dialogues with state regulators and EPA on ASR issues and challenges. In 2009, the National Ground Water Association formed an ASR Task Force, will conduct a technical session in March 2010 in Colorado to discuss technical and regulatory challenges and options with EPA, and is developing Best Suggested Practices for ASR. This paper will summarize US challenges, barriers and proposed solutions through a snapshot of several case studies, and update evolving state and federal regulatory and policy directions and proposed changes.

KEYWORDS
Arsenic; disinfection byproducts; aquifer storage and recovery; waste discharge permit; regulatory; policy.

INTRODUCTION

The purpose of this paper is to discuss several key issues that are challenges and potential barriers to successful implementation of managed aquifer recharge (MAR), in particular, aquifer storage and recovery (ASR) projects, which currently exist in the United States. This information is presented through presenting US ASR policy background, a narrative summary discussion of meetings that have occurred, and a discussion of challenges, issues and progress.

United States National ASR Policy Background

The Safe Drinking Water Act (SDWA), originally enacted by the United States (US) Congress in 1974, was meant to protect sources of drinking water – both surface and ground water. SDWA required the US Environmental Protection Agency (EPA) to develop minimum federal requirements for injection practices that protect public health by preventing injection wells from contaminating underground sources of drinking water (USDWs) from underground injection of contaminants for waste disposal. Injection wells are overseen in an independent manner by either a state agency or one of EPA's regional offices, and States and tribes may apply for primary enforcement responsibility, or primacy, to implement the Underground Injection Control (UIC) Program within their borders. In general, state and tribal programs must meet minimum federal UIC requirements to gain UIC Program primacy.

Five classes of wells were created, each based principally on the injection to result in endangerment, Class I wells are for injection below the lowermost underground source of drinking water of hazardous and non-hazardous industrial and municipal waste, Class II for oil and gas production wastes, Class III for in-situ extraction of minerals, and Class IV inject into or above USDW radioactive or hazardous wastes (and are now prohibited), Class V includes anything that did not fit into the other four classes and has 32 subcategories of injection wells in 8 categories from dry wells to geothermal injection to domestic waste water disposal to recharge wells; and aquifer recharge was lumped into Class V. There are no specific federal regulations or guidance related to aquifer recharge, or aquifer storage and recovery wells. Under the UIC, only the injection is regulated and not the recovery.

Under the Safe Drinking Water Act, underground injection may not endanger drinking water sources. This basically means that any injection may not result in the presence in groundwater, which supplies or can reasonably be expected to supply any public water system, of any contaminant, and the presence of any such contaminant may not result in the systems not complying with any national primary drinking water regulation or otherwise adversely affecting human health.

States ASR Policy Background

Sixteen of the 52 US states have some sort of state-specific ASR policy, although the states regulations vary widely regarding water quality. Some states use maximum contaminant levels as standards for injection water quality, while others use more stringent standards. (Maximum contaminant levels (MCLs) are enforceable standards which are established by EPA to protect the public against consumption of drinking water contaminants that present a risk to human health.) Some states use the UIC permit process to regulate ASR projects, while other states use a waste discharge permit, or a general permit to regulate ASR projects completely outside the UIC.

Regulations regarding the quality of water injected varied by State with some states requiring treatment to meet state or federal standards, "fully consumable and/or reusable", or no requirement to meet standards. States allowing the injection of effluent for ASR have specified effluent be treated to State primary standards or recycled water criteria. Types of injectate used in ASR include treated and untreated surface and ground water, reclaimed water, and effluent.

Some states also use an antidegradation policy, which requires that existing high quality groundwater be maintained to the maximum extent possible. These policies typically allow lowering of quality if the change is consistent with maximum benefit to people of state, will not unreasonably affect present and potential beneficial uses, and will not result in water quality lower than applicable standards.

SUMMARY OF MEETINGS

A number of meetings have taken place over the past few years in the US to discuss MAR and ASR challenges and issues, and case studies. These meetings were held by a number of leading organizations, including the American Ground Water Trust (AGWT), Ground Water Protection Council (GWPC), National Ground Water Association (NGWA) and US Environmental Protection Agency (EPA).

American Ground Water Trust (AGWT – "Trust")

The Trust, established in 1986, is a not-for-profit organization headquartered in the US whose mission is to protect ground water, promote public awareness of the environmental and economic importance of ground water, and provide accurate information to assist public participation in water resources decisions and management. The trust develops high quality groundwater information materials, and brings stakeholders together to encourage open discussion about practical ways to develop, manage and protect ground water resources through workshops, symposia and teachers institutes.

The Trust held ASR meetings in consecutive years in Florida 2001-2009, in Denver 2004 and 2006, and in New Jersey in 2009. Some of the main issues which were discussed include arsenic endangerment versus using a containment zone approach; chloroform limit is inconsistent; the nomenclature of the UIC,"waste" terminology, waste discharge requirements being utilized by many states for ASR; some of the technical terminology is inconsistent; consideration of new UIC class for ASR.

Ground Water Protection Council
Founded in 1983, the Ground Water Protection Council is a national association of state ground water and underground injection control agencies whose mission is to promote the protection and conservation of ground water resources for all beneficial uses, recognizing ground water as a critical component of the ecosystem. Like the Trust, the GWPC provides a forum for stakeholder communication and research, but their focus is to improve governments' role in the protection and conservation of ground water.

Over the last couple of years, the GWPC held ASR sessions at its annual UIC Conference, Annual Forum and spring meetings which members of EPA attended. Many of the same ASR issues were discussed at these meetings (arsenic, endangerment, DBPs) however, there was more of a focus on trying to facilitate obtaining the right level of requirements, studies and information to make good, well-informed decisions on ASR projects, and to avoid failures.

U.S. Environmental Protection Agency (EPA)
EPA convened a meeting of ASR experts in May 2009. The ASR Expert Meeting was designed to bring together diverse parties with experience and interest in ASR, including Federal and State regulatory officials, private industry, environmental organizations, academics, and local water suppliers. The goal was to generate innovative ideas and individual input from participants on how to advance the use of ASR while protecting water quality within USDWs. EPA summarized the discussions and comments, made no commitment for further action, and the summary is posted at http://www.epa.gov/safewater/asr/pdfs/meeting_summary_090309.pdf.

EPA defined ASR as "aquifer storage and recovery" which involved the injection and recovery from one well, or two wells (one injection, one recovery well) within about 500 feet of each other.
Arsenic was identified as a contaminant of concern with occurrence in the majority of regions represented, and formation of trihalomethanes was problematic for many systems that chlorinated water prior to injection.

EPA recognizes that conjunctive use of surface water and groundwater is becoming prevalent, and therefore, ASR is becoming a mainstream option for water management in many areas of the US. With ASR growing, concerns and technical issues arise from EPA's perspective, including:
- Well construction design and testing requirements;
- Potential failure issues associated with the conversion of wells from drinking water withdrawal wells to ASR;
- Potential for particle rearrangement and aquifer porosity alteration in the aquifer due to injection and withdrawal;
- Trihalomethane degradation and concentration in aquifers with ASR wells;
- Leaching of metals from aquifer materials due to ASR wells; and
- Gathering and sharing of any additional lessons learned which will help in improving how AR and ASR wells are regulated.

National Ground Water Association (NGWA)
The National Ground Water Association, founded in 1948, is a nonprofit organization comprised of more than 13,000 U.S. and international groundwater professionals — contractors, scientists and engineers, equipment manufacturers, and suppliers. NGWA's purpose is to provide guidance to members, government representatives, and the public for sound scientific, economic, and beneficial development, protection, and management of the world's groundwater resources. NGWA accomplishes its purpose through a variety of

publications, educational programs, workshops, symposia, and meetings, through a Foundation, certification programs and more.

The topic of ASR has been discussed at the two NGWA major annual meetings (EXPO and Groundwater Summit), Most recently, in spring 2010 in Denver, Colorado, a special session was convened to discuss possible policy changes to the existing national ASR framework. The possible changes debated included whether to establish a new UIC class for all groundwater recharge and ASR projects, to remove ASR form the UIC altogether, to place ASR in the Class III (in-situ extraction of minerals), or to develop some over-riding regulatory/policy guidance including containment zones.

In 2009, NGWA formed a special ASR Task Force under the Subcommittee on Groundwater Protection and Management under NGWA Government Affairs. The ASR Task Force was formed to review and modify existing NGWA policy on groundwater recharge and ASR, to help develop technical information and provide oversight of NGWA activities at the federal level elated to ASR, and provides input regarding developing federal regulation and policy.

In 2010, NGWA initiated development a Best Suggested Practices for ASR. NGWA has a formal process for developing a BSP, and this includes outreach to the NGWA membership for participants, development of the BSP with formal reviews. The ASR BSP is anticipated to be relatively brief, to identify and summarize the works that exist including anoverview of ASR; components of an ASR project; source water and receiving aquifer; data needs, testing and analysis, planning and phasing; drilling methods, well design and construction; monitoring; economic considerations; regulatory and policy challenges; long-term operations.

ARSENIC AS A REGULATORY DRIVER

Arsenic is a known carcinogen and is regulated under the Safe Drinking Water Act, with a maximum contaminant limit of 10 micrograms per liter (ug/L).The heart of the debate of arsenic has been in Florida, where a number of ASR projects have had arsenic concentrations ranging in the 10’s to 100’s of ug/L in the recovered water. The main mechanism is believed to be oxidation and mobilization of arsenic from the aquifer matrix, releasing arsenic into the groundwater.

The main constraint that has arisen in Florida is that if the arsenic goes above the MCL of 10 ug/L on a project in the receiving water and of-property, the state and federal agencies consider this endangerment and injection has to halt. There is currently no regulatory or policy solution, although project proponents are continuing to have those discussions with state and federal agencies, however, there are several approaches being tested which other authors at ISMAR7 will no doubt discuss. Pilots are being conducted with treatment trains to deoxygenate, or remove the oxygen from the source water just prior to injection, There are also some pilots testing the concept of whether multiple cycles of storage and recovery results in a reduction and eventual stabilization of the arsenic levels at lower and possibly acceptable concentrations (below MCL).

DISINFECTION BYPRODUCTS AS A REGULATORY DRIVER

Disinfection byproducts (DBPs) are formed when disinfectants used in water treatment plants react with bromide and/or natural organic matter present in the source water. Different disinfectants produce different types or amounts of DBPs in drinking water for which regulations have been established in the US, including trihalomethanes, haloacetic acids, bromate, and chlorite. Trihalomethanes (THMs) are a group of four chemicals (chloroform, bromodichloromethane, dibromochloromethane, and bromoform) that are formed along with other disinfection byproducts when chlorine or other disinfectants used to control microbial contaminants in drinking water react with naturally occurring organic and inorganic matter in water. Haloacetic acids (HAAs) are a group of five chemicals (monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid) that are formed along with other disinfection byproducts when chlorine or other disinfectants used to control microbial contaminants in drinking water react with naturally occurring organic and inorganic matter in water. The maximum allowable annual average level for THMs is 80 ug/L, and for HAAs is 60 ug/L.

It is well documented that HAAs typically degrade fairly quickly in the subsurface, and are generally not an issue on ASR projects. However, equally well documented, THMs have been observed to degrade only in anaerobic groundwater conditions, and have been shown in certain cases to increase in aerobic conditions (presence of free chlorine and bromate).

Chloroform, which has been shown to be nearly ubiquitous in US surface waters and shallow groundwaters in urban areas, has been used as a key threshold in ASR projects in various parts of the US, and the concentration has been highly variable with inconsistent rationale. In New Jersey, 70 ug/L has been used as an acceptable standard for chloroform, while the state of Washington applies the standard of 7 ug/L, and California has applied both the total trihalomethanes MCL of 80 ug/L in the southern state, and has considered 1.1 ug/L chloroform in the northern Central Valley of the state.

There has been a good deal of work in this area, with many studies and scientific literature available on DBPs and THMs in groundwater, yet in the US, the practices vary widely at the federal and state level, from state-to-state, and even within the some states. This seems like an area for additional work on the federal level and perhaps some sort of federal guidance or policy development.

PUBLIC PERCEPTION CAN BE A CHALLENGE

Several examples exist of public perception as a challenge in implementing an aquifer storage and recovery project. These public perception issues can be moderate to severe, and are usually a result of not developing a specific strategy to educate, inform and receive input from local stakeholders during project planning and progress. Once the public perception challenge starts, it is very difficult if not impossible to reverse.

In Sonoma County in the 1990s, the Sonoma County Water Agency conducted well testing to evaluate the potential for a well injecting Russian River source water from riverbank filtration wells (collector wells), which is very high quality, low turbidity water and is chlorinated for potable use. The project was stopped when the local community got the impression that the source water was to be waste water and reacted badly, even though the source water was planned to be Russian River high quality water from collector wells.

In the City of Rockledge, Brevard County, Florida, the proposed ASR project involves injecting highly treated waste water and injecting into a carbonate aquifer. Some members of the community consider this project a threat to the aquifer. A few of the reasons from the community to oppose include that it is partially treated sewage being injected in to the ASR well in Rockledge into the Underground Source of Drinking Water; it Endangers adjacent ecosystems because of migrating pollutants; it is not beneficial for the public, but enables the State to justify continued uncontrolled growth; the project uses experimental technology in a gamble to conserve drinking water for future growth.

The City of Roseville, Sacramento County, California, developed a project involving high quality, very low turbidity Folsom Lake water derived from Sierra Nevada snowmelt. The project takes the Folsom Lake water which is disinfected and injects in into volcanic material aquifer matrix, which contains moderately total dissolved solids (400-500 mg/L), at the end of the supply system. The recovery cycle simply extracts water from the ASR well, which is now moderately high TDS water being pumped out of the aquifer. The customers were not warned but immediately noticed the moderately high TDS water, and a retirement community has attempted to challenge the environmental documents on the project.

DISCUSSION

Several challenges and issues have surfaced as being more important to address in the US to ensure successful planning and implementation of ASR projects. These challenges and issues have been discussed at several different meeting formats over the past few years and some progress is being made in the US.

Existing US national and state ASR policy is partially derived from and based on pollution control and contamination cleanup, involving hazardous substances regulations. This pollution-based approach causes challenges in the way the projects are considered from a regulatory perspective, in how some of the projects are permitted, and may also cause confusion from a perception perspective.

The UIC Program has its roots in waste disposal and protection of aquifers from pollutants. Many states use waste discharge permits to regulate projects. This association of waste and pollutant with ASR is not good policy, and some sort of change probably should be considered, both from a policy/regulatory perspective and also from a public perception standpoint to minimize confusion.

ASR projects involve the mixing of source water and receiving water of differing chemistry, resulting in changes to the receiving water quality. Some of the changes may be considered improvements, for example lowering the total dissolved solids, while other changes may be considered negative, e.g., DBPs. What becomes important in these considerations is the societal benefit of increased water supply reliability through implementations of these projects, and not restricting the analysis to water quality changes. Further, in basins with chronic lowering water levels, which includes many basins in the southwestern US, water quality will be degrading in the future if it is not already, and this factor should also be considered.

One of the challenges with weighing receiving water quality changes with ASR projects is that regulatory agencies with authority and charge to cleanup pollution from past industrial practices are responsible for oversight of ASR projects. These regulatory agencies in many cases do not have the technical resources nor the mandate to weigh the societal benefit of these water supply reliability projects in contrast to water quality changes. The City of Roseville ASR project may show some progress in this arena in the near future through a document being developed to address the state antidegradation policy regarding the introduction of DBPs from the source water but improvement of TDS in the receiving over time.

The arsenic and endangerment issue is a complex technical problem and difficult regulatory policy challenge. As additional data and more comprehensive information becomes available, a regulatory policy solution appears less difficult, although it also seems likely this whole issue will be solved technically versus through policy change.

Public perception can make or break the project, and needs to be considered and addressed with due respect and resources. This requires careful consideration, planning and outreach to local stakeholders with adequate time, education, and the opportunity to provide input on the planning an, design and implementation phases of the projects. Without a good public outreach and stakeholder program, you run the risk of having your project fail due to public opposition. Over time, as these types of conjunctive use an groundwater recharge projects become more abundant and commonplace, the need for addressing public perception may decrease greatly.

CONCLUSIONS

In the US, the association of "pollution disposal" and "waste" with groundwater recharge and ASR is not good policy, makes it difficult in some areas to facilitate the projects, and may even confuse the public. Changes to these policies should be further considered.

All ASR projects involve some water quality change due to differences in source water-receiving water chemistry. From an antidegradation policy perspective, weighing the benefits of an ASR project should not only be in terms of water quality changes but should also in terms of maximum societal benefit, which would also relate to water supply reliability and other factors.

Arsenic and endangerment continues to progress in Florida, although a policy change seems less likely than a technical solution, such as deoxygenating prior to injection, or multiple cycles reducing arsenic concentrations over time.

Public outreach and stakeholder involvement are necessary elements of an ASR project, especially those involving treated waste water. Over time, as these types of projects become more common and accepted public perception may be a less critical issue; however, currently public perception can make or break the project so it is critical to address with a strong public outreach program.

BIO - TIMOTHY K. PARKER

Mr. Parker recently joined Layne Christensen as the National Groundwater Management Practice Leader. He has worked in public and private sector, previously running his own consulting company, Parker Groundwater, and has worked for several consulting firms, California Department of Water Resources, California Geological Survey, and Department of Toxic Substances Control. Mr. Parker's groundwater experience spans more than 25 years and includes water policy analysis, groundwater resources development, groundwater management, monitoring, and contaminant hydrogeology. He is a California Professional Geologist, Certified Engineering Geologist, and Certified Hydrogeologist. Mr. Parker serves the Groundwater Resources Association of California (GRA) as a Director and Legislative Committee Chair, the California Groundwater Coalition as Director, and American Ground Water Trust as Chair. Mr. Parker is a member of the Oversight Work Group for Pilot Projects for the Nationwide Ground Water Monitoring Network, under the Subcommittee on Ground Water, Advisory Committee on Water Information, U.S. Department of the Interior. Mr. Parker also serves as a Director on the National Ground Water Association-Scientists and Engineers Division. Mr. Parker co-authored the books ***California Groundwater Management***, Second Edition published 2005 by GRA, and ***Potential Groundwater Quality Impacts resulting from Geologic Carbon Sequestration***, published by the Water Research Foundation 2009.

Quantitative Microbial Risk Assessment to determine pathogen risks for a Managed Aquifer Recharge project

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Abstract

With the release of the Australian Guidelines for Water Recycling: Managed Aquifer Recharge (MAR), aquifers are now being included as a treatment barrier when assessing risk of recycled water systems. A MAR research site recharging secondary treated wastewater in an unconfined carbonate aquifer was used in conjunction with a Quantitative Microbial Risk Assessment (QMRA) to assess the microbial pathogen risk in the recovered water. The assessment involved undertaking a detailed hydrogeological assessment of the aquifer at the MAR site and determining the decay rates of reference pathogens from an *in-situ* decay study. These factors were then used in the QMRA which demonstrated that the recovered water at this site did not meet the Australian Guidelines for recycled water when used for irrigation (viral hazards > 1×10^{-6} DALYs). The results also confirmed the importance of obtaining local hydrogeological data as local heterogeneity can influence residence time in the aquifer which, in turn, influences the outcomes. QMRA can be used to determine the residual risk from pathogens in recovered water and showed that it can be a valuable tool in the preliminary design and operation of MAR systems (a pre-commissioning risk assessment) and the incorporation of complementary engineered treatment processes to ensure that there is acceptable health risk from the recovered water for a variety of uses.

Key words: aquifer treatment; rotavirus; *Cryptosporidium*; *Campylobacter*; QMRA;

INTRODUCTION

Water reuse is increasingly regarded as an appropriate and cost effective option for augmentation of urban water supply needs (NRMMC-EPCH-AHMC 2006). Common drivers have included severe water shortages in dry periods, climate change, stricter regulations on waste discharge to the receiving environment and growing urban populations. These drivers have resulted in an increasing interest in a range of reuse methods, including Managed Aquifer Recharge (MAR). MAR can utilise a variety of non-traditional source waters such as reclaimed wastewater. However, the role of the aquifer in the treatment train has often not been considered with the same rigor as other engineered components such as filtration or disinfection, even though it may lead to large improvements in microbiological water quality (Page *et al*. 2010). This has lead to an under appreciation and under valuation of the treatment capacity of aquifers for pathogen removal.

This paper discusses the use of Quantitative Microbial Risk Assessment (QMRA) to assess the human health risk from microbial pathogens in the recovered water when secondary treated wastewater is recycled via MAR for irrigation of green open spaces. Special attention has been given to the contribution of the aquifer barrier and its importance in managing human health risks. This paper adopts the approach given in the Australian Guidelines for Water Recycling 2C Managed Aquifer Recharge (NRMMC-EPHC-NHMRC 2009) to determine $\log_{10}$ removals for the aquifer treatment component, based on pathogen decay rates and subsurface residence time to assess the human health risk from pathogens for the Floreat Infiltration Galleries MAR scheme.

METHODS

Location and design of MAR site

A research MAR scheme known as the Floreat Infiltration Galleries (FIG) using infiltration galleries to recharge secondary treated wastewater from a local wastewater treatment plant to an unconfined aquifer was established in Perth, Western Australia. The infiltration galleries are located in the top of a 10 m-thick unsaturated zone consisting of approximately 7 m of clean, naturally deposited, iron-coated siliceous sand, grading into predominantly carbonate cemented sand with low organic carbon content; the latter, referred to as the Tamala Limestone aquifer, is variably cemented and has dual porosity with groundwater storage between grains of sand and large solution cavities.

Two parallel infiltration galleries are in place at the site and under normal operating conditions, each receives 25 kL of secondary treated wastewater per day. This equates to a total mean hydraulic loading rate of 1 m/day across the two galleries. The recycled water is extracted from the aquifer by a pumping bore located 50m hydraulically down-gradient from the galleries. The natural groundwater table at the recharge site is relatively flat and the magnitude of regional groundwater flow is low. To produce an aquifer residence time that fitted with the three year research project time frame, a forced hydraulic gradient of 0.2% was produced by pumping continuously from the extraction bore at 250 kL/day. The residence time was estimated from groundwater flow modelling with MODFLOW using a hydraulic conductivity derived from the analysis of pumping test data (Bekele *et al.* 2009). Monitoring of the water quality in the aquifer within the MAR zone was undertaken from sampling a number of piezometers located upgradient and between the infiltration galleries and the recovery well. Further details of the MAR research site are reported by Toze *et al*. (2010).

Risk assessment methodology

The development of MAR projects generally follows a sequence; from concept design; to desk-top and laboratory investigations to develop the concept and enable a detailed design for a trial project; followed by approvals, construction, commissioning trials; and finally implementation of an ongoing operation. Integral to these steps are stages of risk assessment designed to ensure protection of human health and the environment. The quantitative microbial risk assessment (QMRA) presented for the FIG site follows the approach outlined in NRMMC-EPHC-NHMRC (2009) using a stochastic assessment approach. In stochastic risk assessment approaches (also know as Monte Carlo simulations), uncertain inputs in a model are represented using ranges of possible values known as probability distributions functions (PDFs). By using PDFs, variables can have different probabilities of different outcomes occurring. Deterministic variables and PDFs used in the Monte Carlo simulation are presented in Table 1.

The credit of $\log_{10}$ removals for the aquifer was calculated as the product of the residence time (in days) in the aquifer and the pathogen decay rate (in $\log_{10}$/day). Residence time in the aquifer was calculated from the break through of a conservative tracer (chloride) during the flushing of the brackish aquifer with fresh stormwater, and pathogen decay rates were measured *in situ* in the aquifer using the pathogen decay chamber method details of the approach are reported by Toze *et al*. (2010).

Three representative pathogens, rotavirus, *Cryptosporidium* and *Campylobacter*, were used to assess the human health risk posed by the presence of enteric viruses, protozoa and bacteria through exposure to the recovered water. Frequency of exposure to ingestion of sprays from irrigation was assumed to be 90/person/year with an associated volume of 0.1 mL (NRMMC-EPHC-NHMRC 2009). The final risk estimates are probability PDFs which can have variable distribution types, and the outputs are typically given as the mean, median and 95^{th} percentile. The tolerable mean risk adopted is taken from the national guidelines, i.e., 10^{-6} DALYs per person per year (NRMMC-EPHC-NHMRC 2009).

The risk models for simulating hazard reduction, consumption, infection and disease burden (expressed as DALYs, Disability Adjusted Life Years) were constructed using MS Excel program [2003] enhanced with @Risk Industrial v. 4.5 [Palisade Corp, USA]. @RISK adds functions to MS Excel for defining PDFs and analysing QMRA output results. All results were calculated down to < 1×10^{-10} DALYs per person per year.

Table 1 QMRA input parameters for three reference pathogens used in this study

Pathogen	rotavirus	*Cryptosporidium*	*Campylobacter*	PDF type	Reference
Pathogen source water numbers (n/L)	26, 77	28, 52	1 – 10^3 *	Uniform (μ, σ)	Toze *et al*. (2010)
Subsurface storage (residence time days)		70, 20		Normal (μ, σ)	Toze *et al*. (2010)
Pathogen decay rate aquifer (T_{90})	42, 8	31, 1	2, 0.2	Normal (μ, σ)	Toze *et al*. (2010)
Dose-response parameters	α = 0.253 β = 0.426	r = 0.059	α = 0.145 β =7.58	Single value	NRMMC-EPHC (2006)
DALYs per infection	0.013	0.0015	0.0046	Single value	NRMMC-EPHC (2006)

* Uniform distribution (minimum, maximum)

RESULTS AND DISCUSSION

In order to protect human health in water recycling via aquifers an integrated approach to managing risks needs to be adopted which includes assessment of the aquifer treatment barrier. In valuing the treatment capacity, integrity and independence of aquifers, MAR can be brought to the same level as conventional engineered water treatment in an integrated system with the same tools used to prioritise capital investment, plan capital maintenance, and develop plant maintenance schedules to minimise the risk of system malfunction or failure.

The pathogen hazards in the source water were a primary concern to human health. Aquifer treatment characteristics were derived from the residence time in the aquifer and the reported $\log_{10}$ decay rate for pathogens (Table 1). The aquifer treatment barrier capacities are reported as $\log_{10}$ removals (Table 2) which convey the order of magnitude of the removal. All $\log_{10}$ removal capacities accredited to the aquifer were capped at a maximum of > 6.0 similarly to how other treatment technologies capacities are reported.

Table 2 Aquifer and other treatment $\log_{10}$ removal characteristics

		Aquifer	Secondary treatment^	UV disinfection	Reverse osmosis	Chlorination
Rotavirus	Min	0.2	0.5	> 1.0		1.0
	Most likely	2.1			> 6.0	
	Max	> 6.0	2.0	> 3.0		3.0
Cryptosporidium	Min	0.2	0.5			0.0
	Most likely	2.4		> 3.0	> 6.0	
	Max	4.9	1.0			0.5
Campylobacter	Min	2.8	1.0	2.0		2.0
	Most likely	> 6.0			> 6.0	
	Max	> 6.0	3.0	> 4.0		6.0

All treatment $\log_{10}$ removals except for the aquifer are from the Australian Guidelines for Water Recycling (NRMMC-EPCH-AHMC 2006).
^Secondary treatment refers to dual media filtration and coagulation

The value of the aquifer barrier was determined by the relative $\log_{10}$ removal characteristics with respect to the reference pathogens (Table 2). The most likely calculated $\log_{10}$ removal for

Campylobacter was > 6.0 $\log_{10}$, a similar value attributed to other water treatment technologies such as reverse osmosis (Table 2, NRMMC-EPCH-AHMC 2006). For *Cryptosporidium* the most likely $\log_{10}$ removal of the aquifer was similar in value to the range reported for UV disinfection. Rotavirus had the lowest removal (Table 2) due to the very low decay rates reported in the aquifer. For comparison in valuing the aquifer treatment, water treatment technologies providing a similar $\log_{10}$ removal were UV or Chlorine disinfection. It is recognised that the treatment capacity of the aquifer is affected by other factors including the temperature, redox status, nutrients of the recharge water.

The residual risks calculated for the FIG MAR scheme are shown in Table 3. The FIG scheme had a mean unacceptable risk ($< 1 \times 10^{-6}$ DALYs) for rotavirus whilst *Cryptosporidium* and *Campylobacter* risks were acceptable. The 95th percentile gives an estimate of the variability of the risk, and where the mean and 95th percentile are both below the acceptable risk threshold (*Cryptosporidium* and *Campylobacter*), then the estimate of the risk is considered to be robust. The median value gives an estimate of the central tendency of the data where it is not normally distributed.

Table 3 Results of the human health risk assessment (in DALYs)

Pathogen		Ingestion of sprays (DALYs)
Rotavirus	Mean	$\mathbf{4.3 \times 10^{-6}}$
	Median	4.9×10^{-7}
	95th percentile	$\mathbf{7.0 \times 10^{-4}}$
Cryptosporidium	Mean	6.2×10^{-8}
	Median	1.6×10^{-8}
	95th percentile	2.5×10^{-7}
Campylobacter	Mean	4.6×10^{-7}
	Median	$< 1 \times 10^{-10}$
	95th percentile	$< 1 \times 10^{-10}$

Numbers in bold exceed the acceptable risk threshold.

The results using data from the pathogen decay data and hydrogeological information suggest that reclaimed water extracted from the Floreat Infiltration Galleries (as it is currently operated and modelled in this study) used for green space irrigation results in a mean DALY's value which exceeds the guideline value of 10^{-6} DALYs per person per year for rotavirus (Table 3). The FIG MAR scheme would need to be redesigned to either increase the residence time of the water within the aquifer to > 150 days; or an additional level of treatment such as UV disinfection be added either before recharge or post-recovery. The results, however, are only marginally above the DALYs guideline value.

CONCLUSIONS

The MAR risk assessment framework of NRMMC-EPHC-NHMRC (2009) provides a means of assessing the aquifer barrier of the FIG project which combines both natural and engineered treatment components. The proposed configuration of the FIG project has demonstrated that the risks from *Cryptosporidium* and *Campylobacter* are acceptable. Additional treatment steps or a longer residence time in the aquifer would be required for acceptable risks from viral hazards. In addition, QMRA was a useful tool in establishing the value of the aquifer within the treatment train. The aquifer was identified as a potent barrier, which can exhibit a high capacity for water treatment. The use of residence time in the aquifer is a conservative approach and decay rates and treatments will be aquifer specific. This approach can be further developed to extend this by adopting a wider-ranging, multiple barrier approach that looks at a broad range of 'preventative measures' for managing risks from catchment to tap, e.g. trade waste and catchment management.

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ARE WE STANDING ON A SOLUTION WITHOUT EVEN KNOWING IT?

The 3R approach for development and adaptation to climate variability and change.

Albert Tuinhof[1], Frank Van Steenbergen[2] and Arjen De Vries[3]

SUMMARY

The point of departure is the buffer function that groundwater provides – allowing one to deal with peaks and lows and the larger variability that in many areas is expected to come with climate change. The philosophy is to manage this buffer function through three subsequent steps – Recharge, Retention and Reuse. There are a large number of possible (technical) solutions to achieve this which we have grouped under the name 3R, covering the broader process of retaining and intercepting the rainfall and runoff, store it underground or in tanks at appropriate places and plan for its re-use during the dry periods.

The larger idea is that tackling a local water crisis is not so much about allocating scarce water, but to extend the temporal availability of the water within the basin. 3R is an upscaled approach – it looks not at an isolated intervention but aims to improve water storage in the entire hydrological unit providing water for drinking and for economic use while making a link with land use planning and managing both natural and artificial recharge. It manipulates the shallow groundwater and as a result greatly influences agricultural production and the vitality of rangelands and other ecosystems through the sub-irrigation and better soil chemistry that come along with it. In the particular case of large irrigation systems 3R makes way for better conjunctive management

The added value of 3R is the systematic approach to assess the technical, hydrological and socio-economic and institutional feasibility of planning and implementation of 3R systems on a (sub) basin scale and within the broader IWRM framework. The initiative will make a major contribution to realize effective climate change adaptation solutions in the large areas that cope with increasing droughts

KEYWORDS
Adaptation, groundwater, recharge, retention, reuse, storage,

INTRODUCTION

There is an urgent need to optimize the potential that groundwater provides to deal with current peaks and lows and the increased variability expected in many areas owing to climate change. The area of the largest interest buffer is the storage that is associated with the top layers of the soil (in the unsaturated zone above shallow aquifers) and in very shallow aquifers. In many places this shallow groundwater storage capacity can be used to carefully infiltrate rainwater and run-off, augmented by flow from rivers and irrigation. This stored water can be retained and re-used in a later phase. This makes up the 3Rs – of Recharge, Retention and Re-Use.

The 3R approach has a focus on groundwater, but does also include the harvesting and storing of rainwater in tanks and reservoirs, and surface water storage through small reservoirs. Managing the overall water storage capacity better is of vital importance – to serve water for people, water for food and water for ecosystems. The larger idea is that tackling a local water crisis is not so much about allocating scarce water, but to catch water and extend the chain of water use and its reuse as much as possible within a basin. Also water should not only be managed when it is scarce, as in arid areas, but also when it is abundant. What is being done might be different in arid and in humid areas but better water management and climate change adaptation are necessary everywhere.

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THE ARGUMENTS FOR MANAGING THE GROUNDWATER BUFFERS AT SCALE

Making use of the groundwater buffer for storage has many advantages. Groundwater is usually available (or available nearby) at the point of desired use and the same holds for subsurface storage. There are a number of other arguments that support an active management of the groundwater buffer. First of all, sub-surface storage has not only the advantage that evaporation losses, if at all, are low and that water is relatively protected against pollution but also that the quality is improved. Also through buffering in the subsurface, the quantity and quality of the water resources can be strongly incremented, making water potentially available during the dry season. Finally, managing the water buffer has some important positive side-effects. These are not always taken into account, but they can have a substantial impact. High groundwater tables are the key to adequate soil moisture. As such they contribute importantly to ‘green water management’. This is the management of soil moisture – as opposed to the management of blue water – which is the water in rivers, lakes and reservoirs. Assured soil moisture is directly related to high productivity in rain-fed agriculture both in arid and humid areas.

3-R TECHNIQUES

There are several techniques that can increase storage capacity at the scale of a sub-basin and improve its management. Some of these techniques are ancient and time-tested, others are new and innovative. Figure 1 gives a graphical presentation of a hypothetical basin with an overview of the technical interventions. These interventions or 3R elements contribute to improving the water buffer function in the entire basin or sub basin.

3R aims to design and integrate these individual solutions for the entire basin or sub-basin in a close link with all local planning activities: spatial planning, but also the planning of local infrastructure and land development, irrigation and nature areas. The essence of 3R is to set things right at scale and not get stuck in isolated improvements and interventions. The whole approach consists of three steps.

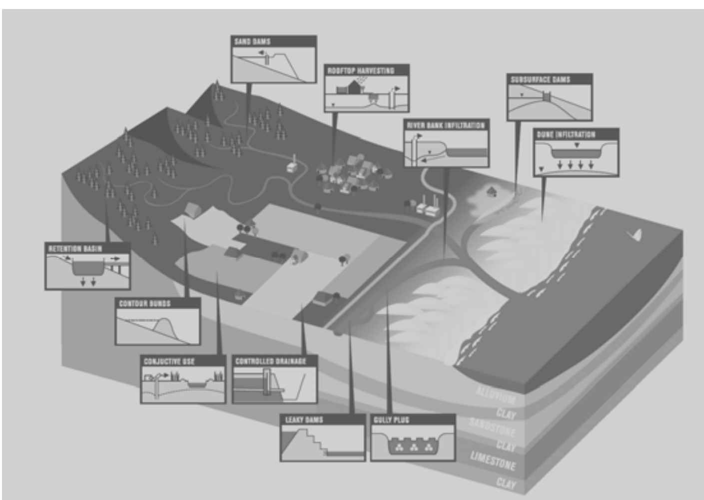

Figure 1 Possible 3R interventions at the catchment scale

Recharge adds water to the buffer and as such it adds water for circulation. Recharge can come from the interception of rain and run-off water (natural recharge), from increased infiltration of natural processes by manmade interventions (artificial recharge or managed aquifer recharge – MAR) or it can be the welcome by-product of for instance inefficient irrigation or leaking pipes in water supply systems. To do recharge at scale hence requires managing natural recharge, applying artificial recharge and controlling incidental recharge. There are many techniques for *artificial groundwater recharge* – some ancient and time-tested, some very innovative. They range from individual rooftop rain water harvesting systems and recharge wells to water harvesting at catchment level, as in spate irrigation.

It is equally important to manage the *natural recharge*. Natural recharge benefits from maintaining or constructing landscape elements that slow down and retain surface run-off: terraces, low bunds, depressions but also importantly by the clever design of roads and canal embankments. Natural recharge is also helped by making sure open spaces are accentuated in the design avoiding wide-spread development that is usually entirely impervious to natural recharge. Natural recharge is also linked closely to the condition of streams and rivers. The capacity of rivers to store and buffer floods should be safeguarded by not constraining them in narrow embankments, removing their gravels and sands, or by slowing stream velocity.

The second element of 3R is retention. Retention slows down the lateral flow of groundwater. This helps pond up groundwater and create a large wet buffer in the subsoil. In such conditions it is easier to retrieve and circulate water. Hence, retention makes it possible to extend the chain of water uses. Retention also raises the groundwater table. Slowing down or even controlling the lateral outflow water table affects soil moisture and soil chemistry and, therefore, improves the yield of rain-dependent agriculture.

There are many techniques for groundwater retention – from simple to sophisticated. On the low cost end, earthen gully plugs in drainage canals can retain groundwater in large areas. Subsurface dams and sand dams have the same effect of retaining groundwater and creating huge reservoirs by decreasing the outflow level.

The third element of 3R is reuse. The large challenge of 3R is to recycle water as much as possible. Scarcity is not only resolved by managing demand through reduction in use but also by keeping water in active circulation. In managing reuse three processes are important. The first is to *manage (non beneficial) evaporation.* Water that evaporates 'leaves' the system and can no longer circulate in it. This is an important concept – in some areas for instance 'efficient' irrigation reduces reusable recharge and makes a larger part of the water evaporate. This makes less water available for reuse and also may jeopardize the water balance. One source of evaporation is from the soil – particularly from depressions and moist stretches.

The second process in managing reuse is managing water quality. The possibility for reuse depends on the quality of the water – with different uses putting different demands on expected water quality. Water quality management is an important element in buffer management – avoiding that reusable water is mixed with lower quality water or avoiding up-coning or lateral flows from lower quality sources. Much care is also required to avoid that repeated reuse of water does not move water quality beyond safe thresholds.

The third aspect in optimizing reuse is to make sure that water does not move to an area from which it is difficult to retrieve and reuse. Here the difference between wet and dry buffers is important. Water which is recharged in a dry unsaturated buffer is difficult to retrieve and though it is not lost it is difficult to bring back into circulation. On the other hand, when the buffer is saturated it can be easily retrieved. An important challenge in 3R is to increase the wet buffers and manage the existing ones. By raising groundwater levels and slowing down lateral movement retention can create or enlarge the saturated zone. It is important to appreciate these nuances and not to assume that because a basin is a hydrological unit that all water related processes in the basin are one and the same.

In order to apply the 3R approach and the subsequent management techniques successfully, it is essential to know the characteristics of the physical system. It is important to understand 'what lies beneath' – the characteristics of the groundwater buffer. Not all buffers are the same. They differ in size, in hydrological interaction, in storage capacity, in vulnerability. The characterization of the groundwater system in a basin will require specialized input from a hydrogeologist to map out the different aquifers and its key properties.

Below we present a case where the use of 3R makes or can make a substantial impact on the water buffer in an area and with the livelihood and local economy. Other examples can be found in Van Steenbergen and Tuinhof (2009).

CASE 1: CONJUNCTIVE WATER MANAGEMENT IN SINDH, PAKISTAN

The persistent thought is that in irrigation systems tail-end areas are at the mercy of upstream water users and are water-short. The first point is correct, the second not necessarily. The Kotri Left Bank command in Sindh, Pakistan at the far end of world's largest irrigation system illustrates this. The problem in the Left Bank is not so much short supplies but erratic deliveries. Added to the salinity of groundwater in this command area, water logging has become widespread - creating a water management problem of humanitarian ramifications. The cause of the problem is a lack of conjunctive management.

The Kotri Left Bank area

The Kotri Left Bank area is supplied by three large irrigation canals off-taking from the Kotri Barrage – the last barrage on the Indus: the Lined Canal (also called Akram Wah), Pinyari and Fuleli. Of these three canals the Lined Canal is perennial – whereas Pinyari and Fuleli are semi-perennial. This means that they are supposed to receive water mainly in the wet kharif season. This arrangement dates back to the pre-regulation period – when a series of semi-perennial channels - with relatively high irrigation - duties accommodated the high flows in the summer. After the construction of the Tarbela and Mangla dams the hydrological regime changed but the water allocations were never officially adjusted. At present the semi-perennial Pinyari and Fuleli canal receives water in the rabi season as well. The Pinyari canal serves a command area of 318,000 ha and has a discharge of 388m3/second. The Fulelei has a discharge of 423m3/second and serves 450,000 ha. The perennial Akram Wah channel on the other hand serves 197,000 hectares with a design discharge of 102 m3/second. The official and unofficial water duties in three canal commands are very high compared to other commands in Pakistan and even in Sindh Province.

Effect on agriculture

The result of the high supplies is not high production but low production. With the help of the so-called SEBAL (Surface Energy Balance) algorithm water productivity was measured using a series of NOAH images for the 1999/2000 season. The evaporation of the Kotri Left Bank area is the highest, whereas the crop production measured in biomass is the lowest in Sindh Province. The explanation is that water delivery to the Kotri Left Bank is high but erratic and waterlogging is common place (WaterWatch 2005). This picture of high supplies and water logging holds true for the entire Province. In Sindh, during the drought period of 1999-2003 crop production for instance did not go down but went up (Government of Sindh 2006). In the drought period (with releases from the Tarbela dam 20% less) the area that was waterlogged reduced from 2,205,000 ha (38%) of the cropped area to 250,000 ha (5%)– mainly because there was a steep increase in the use of shallow wells to compensate for the reduced surface water supplies (Government of Sindh 2006). This created a better balance and more precision in water deliveries.

Figure 2 Groundwater based irrigation

Effect on drinking water

The effect of the prevailing irrigation management on drinking water supply is dramatic. The Thatta and Badin districts on the Left Bank – with a combined population of more than 2 million people - rank among the most disprivileged places anywhere. As mentioned, the groundwater is naturally saline and because of the oversupply, water tables have risen to the surface. There is hardly storage capacity in the soil layers even for semi-brackish water lenses to form therefore, saline groundwater is at ground surface.

Possible 3R approach and options

The way to go is conjunctive management in the Kotri Left Bank – dovetailing the groundwater situation with the surface water delivery. The aim should be to lower the high water tables and create enough opportunity in the upper layers for sweet/ brackish water lenses to form. This would improve the possibilities of water supply from relatively cleaner groundwater sources. Also the lowering of the water table would also improve crop production and reverse the low yield levels and massive non-beneficial evaporation.

What is required is to revisit the current surface water supplies and overhaul the irrigation system. This is long overdue following the completion of the large storage dams in the country. In targeted areas drainage infrastructure should be constructed and existing drains should be upgraded.

The problem with rethinking water deliveries in the large irrigation systems is that they are sometimes perceived as 'undiscussionable' and 'deadlocked' – not so much by agreement, but because they are little understood beyond a small group of people and are not transparent. Another bottleneck to overcome is the general lack of expertise in conjunctive water management. Despite the dependency on the irrigation systems and its conjunctive nature in the country there is little expertise in analyzing alternative water schedules. All this should not be as it should be possible to greatly improve the social and economic productivity in Kotri Left Bank and at the same time save water for irrigation expansion elsewhere.

CONCLUSIONS

Storage is a key component in every water supply system in order to bridge temporal gaps between supply and demand. Provision of seasonal storage (2-4 month) is generally a cost effective solution compared to other options such as long distance conveyance or treatment of available low quality water. Most developing countries lack sufficient infrastructure for surface water storage and Africa is oft-quoted as the least developed continent on the globe in this respect. What this ignores is sub-surface storage. Storage usually refers to surface water reservoirs behind dams of different sizes ranging from the mega dams (Aswan dam, Three Gorges Dam) to medium and small dams or micro dams. Apart from the debate on the impacts of the larger dams, the disadvantage of surface water storage in reservoirs is the high evaporation, pollution risks and decreasing efficiency due to sedimentation and siltation. A good alternative for surface water storage is sub surface storage (groundwater storage), which has the advantage that there are no evaporative losses, water is protected against pollution and sedimentation being a resource (for building up soils and maintaining fertility) and not a threat to storage longevity.

The 3R approach includes practical instruments to manage groundwater – recharge measures, retention measures and safeguarding water quality in the reuse chain. The approach has a strong institutional dimension – the promotion of local groundwater management (communication, capacity building) and the link to land use planning and the management of surface water (local rivers, irrigation systems and natural drainage.

The cases as illustrated in Van Steenbergen and Tuinhof (2009) clearly show the added value to implementing 3R in water management. It is a systematic approach to assess the technical, hydrological and socio-economic and institutional feasibility of planning and implementation of water storage systems on a (sub) basin scale and within the broader IWRM framework. Implementation of 3R can make a major contribution to realize effective climate change adaptation solutions in the large areas that cope with increasing droughts.

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Could climate change pose limits for the MAR? The case of the impact of climate change on the aquifer recharge of Geneva (Switzerland)

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Abstract

The artificial recharge of transboundary Genevese groundwater uses treated water from the Arve River to recharge the aquifer. This system has been in place since 1980. The water from the Arve River is extremely turbid because of the geology of its water basin and as a result of weather conditions. During the 2003 heat wave, the artificial recharge was less efficient because of the Arve's high turbidity levels during the summer months, preventing any artificial recharge of the aquifer whereas consumption demand was at its peak. The present article examines the effects of extreme climatic conditions on the natural and artificial recharge of the Genevese aquifer and raises the issue of the limited efficiency of artificial aquifer recharge systems in similar contexts.

Keywords: artificial recharge management; climate change; Alps; Arve river; Geneva.

INTRODUCTION

Geneva's drinking water is provided from the Lake of Geneva (roughly 80% of total supply) and about a dozen wells (accounting for around 20%) by pumping from the Genevese groundwater. The aquifer straddles the canton of Geneva in Switzerland and the French department of Haute Savoie (Upper Savoy) and is currently used on the Swiss side for ten wells and for five wells across the French border (Fig. 1).

During the 1960's and 1970's, as a result of uncontrolled over pumping and the lack of coordination among distributing and beneficiary entities, groundwater levels fell drastically, to the point where certain dried out wells had to be closed. That was when the warning bell was sounded in the face of this over-exploitation of groundwater resources.

The decision was then taken to set up an artificial aquifer recharge plant so as to recover use of the wells and take advantage of the large volumes of water that could be stored in the Genevese aquifer. An artificial recharge system was, therefore, inaugurated in 1980 to abstract water from the Arve River - which is the aquifer's main natural recharge source- treat it and channel it into the aquifer. This operation, effected via drains that are laid underground above the aquifer, ensure the maintenance of high groundwater levels as well as seasonal stockpiling of the drinking water resource. The system has been described in articles on artificial recharge (de los Cobos, 2002, 2007).

DESCRIPTION OF THE ARTIFICIAL RECHARGE PLANT

The Genevese artificial groundwater recharge plant comprises the following elements (Fig. 2):

1. Water catchment from the Arve River, 300 metres above the plant with a self-cleaning filter to eliminate large floating objects or sediment.
2. Piping (700 mm in diameter; 340 m long) of untreated water
3. Water treatment plant including desanding and micro flocculation ($FeCl_3$) through a triple-layered filter and chlorination (Cl_2). The maximum flow rate is 630 l/s.
4. Piping of treated water to the underground area, 800 mm in diameter and 700 m long, where the water is channelled into the aquifer.
5. An underground area of 3 hectares comprising 4,950 m of 2 m-deep absorbent trenches fitted with perforated distribution pipes 200 mm in diameter. These perforated pipes are laid at 7m above the water table in the unsaturated zone.

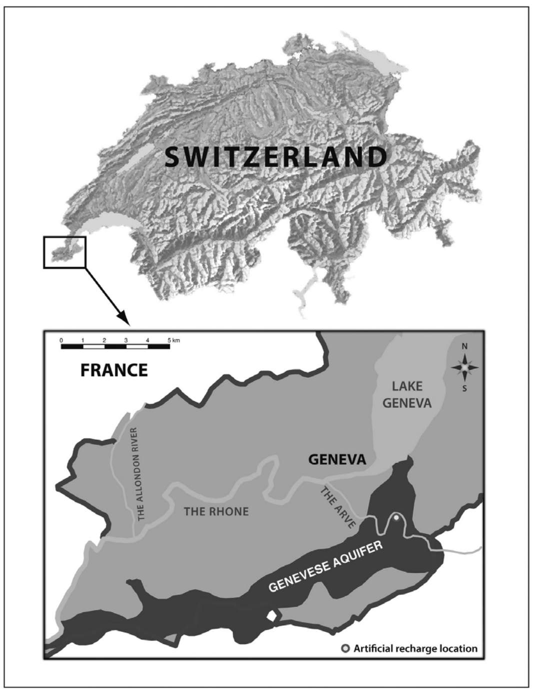

Figure 1: Location of the Genevese aquifer

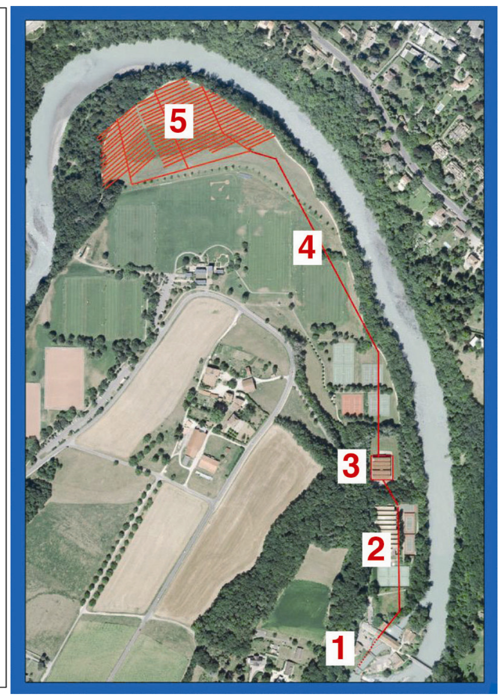

Figure 2: The Genevese aquifer recharge plant

In theory, the plant has a total capacity of 17 Mm^3/year (540 l/s). However, bearing in mind the high turbidity of the Arve River which forces closure of the plant during certain periods (snow melt, surges following heavy storms 65 days per year on average) and the automatic cessation of the plant in the event of pollution, real capacity lies more in the region of 9-11 Mm^3/year. Since the start of the plant's operations in 1980, after 30 years of exploitation, the Genevese artificial recharge system has channelled approximately 250 Mm^3 of treated water into the Genevese aquifer.

THE ROLE OF THE ARVE RIVER

The Arve River originates in the Mont Blanc mountain range, about 100 km away from Geneva and flows from a mountainous river basin of around 2,060 km^2, six per cent of which is made up of glaciers. The river basin is divided into three basins of different hydrological regimes (Fig. 3):

The *upstream basin* consists of glacier regime torrents. The glaciers provide a considerable amount of water in the summer, from the melting of snow and perennial ice, and provide for the stockpiling of solid precipitation in the winter which reduces the incidence of severe flooding.

The *middle basin* is exposed to western oceanic perturbations and thereby subject to a strongly pluvial-dependant flow regime**.** Precipitation is heavy (1400 mm), resulting in high flow rates. The surges are especially evident in the spring (from rain and snow melt) and to a lesser degree in the summer, due to storms.

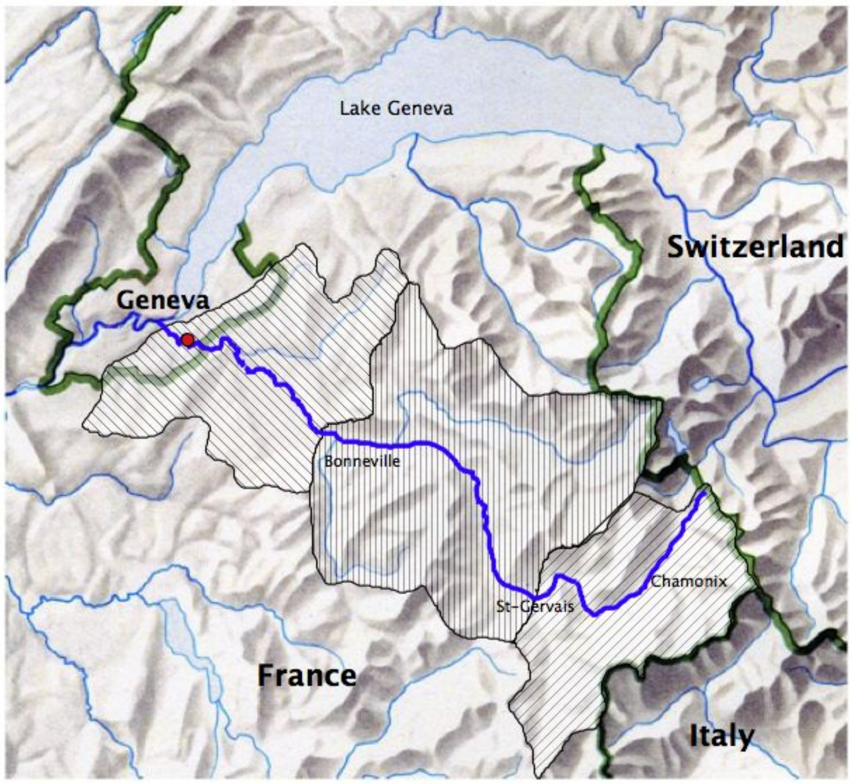

Figure 3: The Arve river basin: the upstream basin (right), the middle basin (centre) and the downstream basin (left).

The relief of the Arve's *downstream basin* comprises the foothills of the Alps but is not very well protected from the dominant western perturbations. Rainfall precipitation is considerable (>1000 mm annual average), attaining maximum levels between autumn and spring. This type of surge can, therefore, also take place in winter.

It has been generally observed that the regime governing the downstream basin is practically the reverse of its counterpart upstream. Overall, therefore, increased volumes of flow surges are equally likely to occur at all seasons of the year, although the highest levels are usually recorded in June, July and August and in October and November. The unique features of this river, then, are the significant flow variations, depending on the weather conditions, and extremely rapid surges (several hours upstream and under 24 hours downstream).

This situation creates high levels of turbidity in the river water, thus preventing the Genevese artificial recharge plant from functioning. In fact, once turbidity exceeds 120 NTU, artificial recharge ceases so as to avoid blockage of the system, at which point natural recharge takes over, benefiting from the large water volumes and allowing for groundwater recharge, particularly in the spring when the snow melts. This is the general pattern that has been observed for decades concerning the relationship between the river and groundwater levels for recharge of the Genevese aquifer.

THE 2003 HEAT WAVE

The high point of summer 2003 in Western Europe was the unusual heat wave it experienced. Geneva recorded 27 days of average daily temperatures of or above 25°C between June and August with temperatures peaking at 36°C for twelve consecutive days in early August. There was very little rainfall from January to August 2003 and, by the end of July, Geneva rainfall had dropped below the 10-year average for the corresponding period by over 39% (MétéoSuisse data). These climatic conditions were brought on by increased demand for water with more than 1.6 million m^3 being pumped per month from Genevese groundwater for June, July and August. The high demand drew heavily on the water table resulting in a 3-m decline in the piezometric level between early May and early September in the artificial recharge zone.

Although the system was introduced to stabilise peak demand in summer through year-long regulated management of stocks and piezometric levels, the artificial recharge system did not work from mid-May to mid-August 2003 due to the Arve's high turbidity levels. This seemingly paradoxical situation of low flow rates coupled with high turbidity in the Arve was due to the extreme climatic conditions which prevailed that summer. What happened was that the 0°C isotherm stood at an altitude of over 3,800 m for weeks, rising up to 4,600 m. This very rare occurrence triggered the melting of high-altitude ice and snow, thus destabilising the permafrost and filling these meltwaters with fine as well as coarse mineral sediment. As a result, for the entire summer, the river water was replete with sediment, preventing it from being used for artificial recharge which was essential when demand was at its highest. In the absence of surges, there was no natural recharge to take up the slack.

The recharge of Genevese groundwater for the period from spring to summer (May-August) 2003 can be summed up as follows (Fig. 4):

In terms of precipitation, low values were recorded in Geneva with rainfall of 227 mm from May to August as opposed to 317 mm for a normal year, with only four days of rainfall above 20 mm.

The impact on the river regime was a low average flow rate (around 90 m^3/s) with reduced surge peaks (< 260 m^3/s). Consequently, there was practically no change in the water table's piezometric level nor, by extension, was there any impact on the natural recharge of the aquifer. The piezometric level dropped steadily between mid-May and mid-August when artificial recharge was resumed. Artificial recharge was extremely low for the said period because of the high turbidity levels which prohibited the functioning of the plant (630,400 m^3 of water channelled artificially). Use of the plant had to be suspended for 78 days between May and August 2003, because of high turbidity.

The plant operators, SIG, undertook to force artificial recharge by increasing the admissible turbidity limit from 120 to 150 NTU. At the same time, some of the Genevese groundwater wells were brought to a halt while the pumping of others was restricted and the whole network reorganised so as to reduce demand on the water table to a minimum (distribution of lake water). These exceptional measures were necessary to recoup the global groundwater deficit and to take maximum precautions to face a yet uncertain autumn.

That summer of the heat wave, with the increased turbidity levels of the Arve, highlighted the possible implications of climate change for Genevese artificial recharge management. In fact, scientists working on climate change in the Alps predict rising temperatures in this part of the Alps, by the end of the 21st century, of some 4°-5°C compared to the average temperatures recorded during the 20th century (ClimChAlp, 2008). They further suggest that there could be greater seasonal differences in average and extreme precipitation with more intense precipitation events taking place in the spring and autumn and fewer in the summer. Spring meltwater flows are also expected to decrease because of more gradual melting of the reduced snow cover. Generally speaking, there could be more frequent water shortages and even droughts during the summer because of lower precipitation levels at that time of the year and greater water loss.

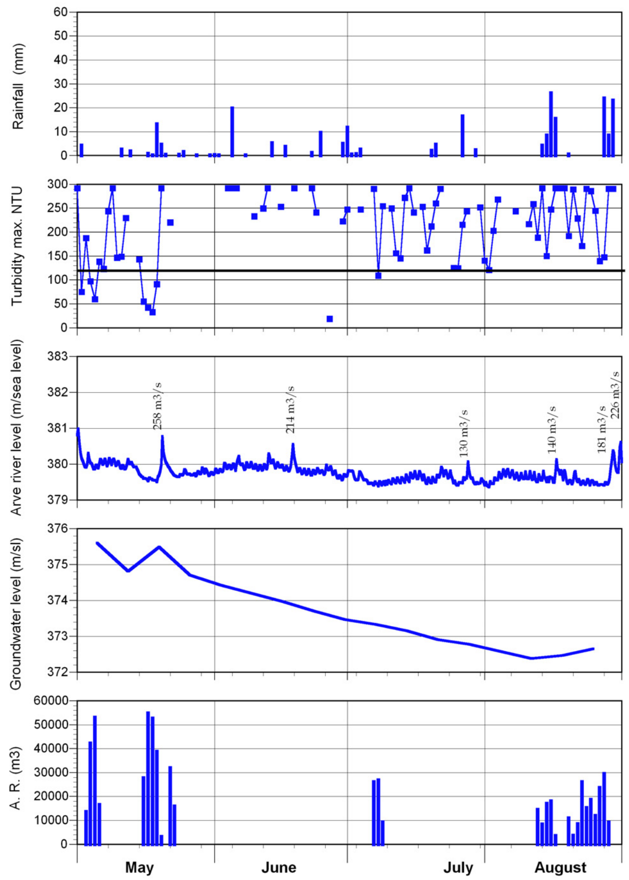

Figure 4: Impact of the meteorological conditions on the aquifer recharge – summer 2003

When meteorological records revealed, at the time, that July 2006 was the hottest month ever in the entire history of Swiss meteorology, the decision was taken to monitor the spring and summer periods more closely in respect of the conditions for natural and artificial recharge of Genevese groundwater. The aim was to evaluate the effect of climate change on groundwater management so as to better understand the relationship between the river and groundwater levels for natural and artificial recharge of the aquifer. This was a necessary step to ensure sound management of groundwater resources.

SPRING 2006

From experience, we know that natural recharge of the Genevese aquifer from the Arve usually takes place in the spring when the snow melts. The voluntary cessation of artificial recharge for maintenance work at the plant has facilitated a clearer assessment of the natural recharge from the river to the aquifer without the reaction of the water table to artificial recharge (Fig. 5).

Consequently, we were able to note a significant rise in the piezometric level, practically throughout the aquifer, as the piezometric levels measured close to the river revealed an increase of almost two metres between end of March and mid-May. This considerable recharge, estimated at close to 2 Mm^3 of water volume in the aquifer, occurred when the plant was closed, therefore resulting from natural recharge. This was borne out by a meteorological and hydrological analysis.

For the period running from March to June 2006, we noted an average rise in rainfall levels (400 mm of rainfall) with an extreme event taking place in April (59 mm in one day in Geneva). The impact of all these events on the entire Arve river basin was higher water levels (peaking at between 150 and 524 m^3/s), representing around 12 events above 200 m^3/s over a 4-month period. The average flow rate of the Arve for the same period was roughly 115 m^3/s.

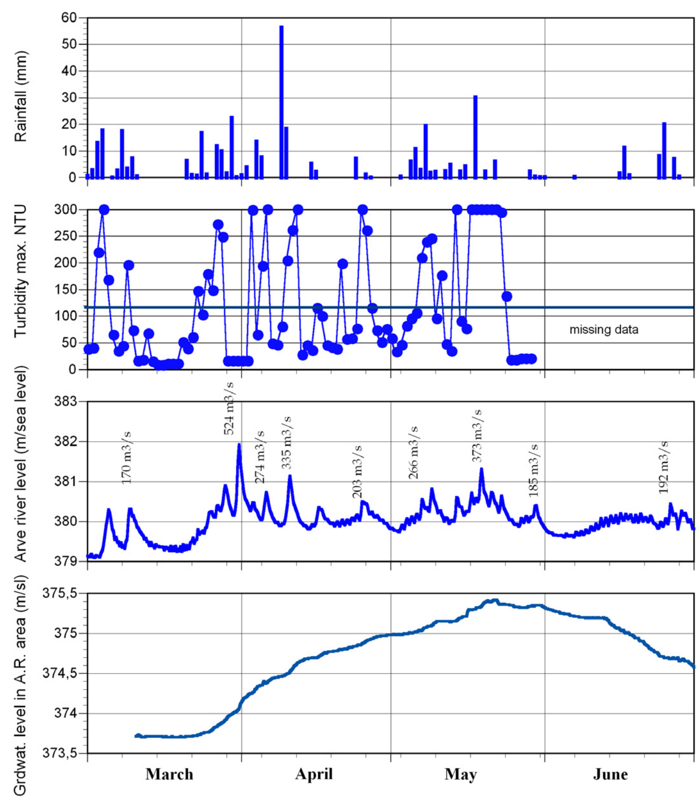

Figure 5: Impact of the meteorological conditions on the aquifer recharge – spring 2006

The natural recharge was welcome in view of the long-term work which had already started and which precluded recourse to artificial recharge. Just ahead of the summer, this natural input was essential for the management of the aquifer and assuaged fears of having a summer of water restrictions.

SUMMER 2007

Unlike summer 2003, summer 2007 was characterised by strong precipitation, especially from storms, with very moderate average temperatures (Fig. 6). Between May and August 2007, 582 mm of rainfall was recorded in Geneva, that is, over 50% of annual average rainfall (10 days with rainfall levels of above 10 mm and 5 days above 30 mm). As for the river regime, the result was strong surges with 10 events of above 300 m^3/s, yielding an average flow rate for those four months of around 180 m^3/s, which is relatively high.

These repeated events, with their high turbidity levels, caused the artificial recharge plant to be closed between mid-June and mid-August. However, those surges did not make it possible for us to conclude that the river had a strong influence on the natural recharge of the aquifer, since the piezometric levels did not surpass a rise of 50 cm, or an overall estimated volume of around 0.7 Mm^3 of water in the aquifer.

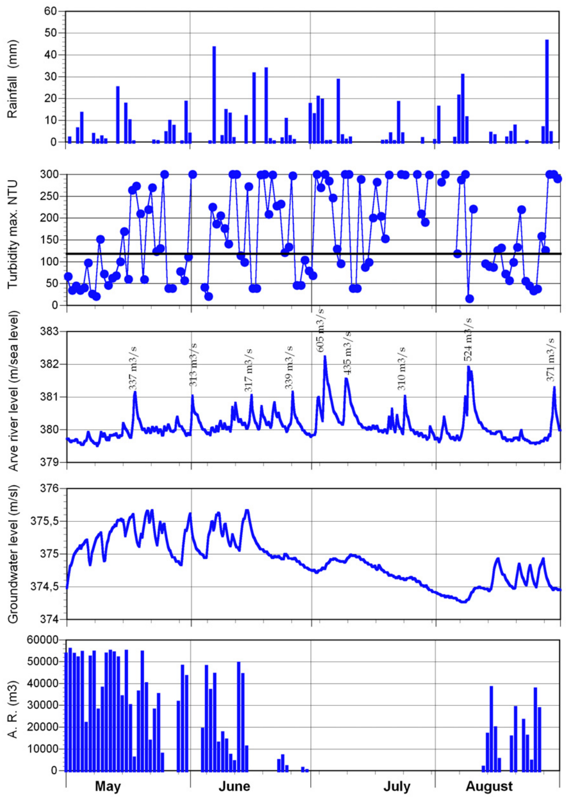

Figure 6: Impacts on the recharge–summer 2007

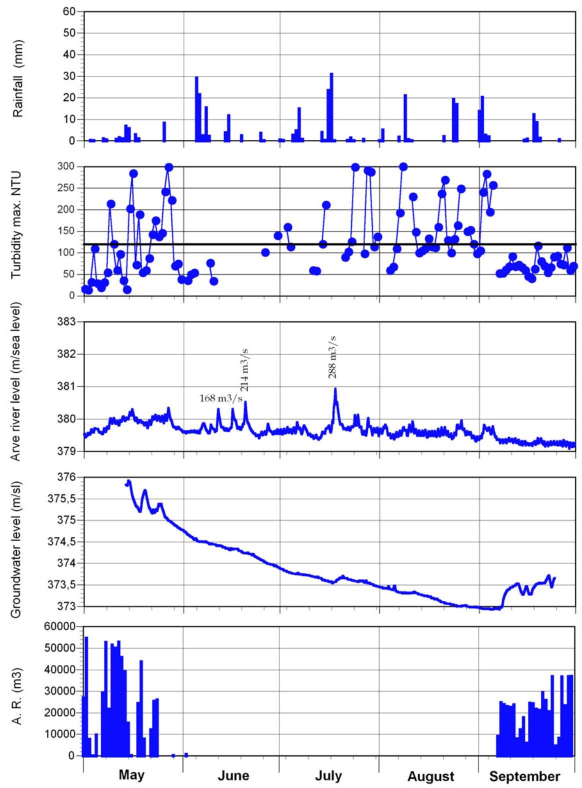

Figure 7: Impacts on the recharge–summer 2009

SUMMER 2009

Summer 2009 was marked by the voluntary closure of the artificial recharge plant in order to change the water chlorination systems. In the absence of any artificial recharge, we were able to gain a better idea of the natural influence of the river on the aquifer (Fig. 7). From a meteorological point of view, there was no noteworthy impact during the period from May to October 2009 except for some drought compared to the average. Nevertheless, there were six events of more than 20 mm of rain, including two events registering over 30 mm of rainfall in Geneva. With respect to the Arve river basin, there was a slight impact on the river regime with peaks between 160 and 215 m^3/s being recorded between May and June 2009. A single event in mid-July produced a surge peak of 288 m^3/s.

The effect on the aquifer was minor but perceptible, especially in June 2009 with volume being related to small successive water rises of some 0.3 Mm^3 in the aquifer. The most significant occurrence in July was a marked rise of the piezometric level of the part of the water table closest to the river with an estimated volume of 0.4 to 0.5 Mm^3 of water moving into the aquifer from mid-July to early September. This was not a vast amount, but, considering the declining piezometric levels of over 2.5 metres between mid-May and early September 2009, the additional volume of water resulting from the natural recharge of the river is an indication of the need for occasional surges to "cleanse" the river bed of its silt layer and allow for natural recharge to continue over the ensuing weeks, even if in small quantities.

CONCLUSIONS

Although it is not easy to accurately monitor the impact of the recharge of Genevese groundwater from the Arve River due to the fact that artificial recharge readily perturbs the direct influence of the river, there are a few preliminary conclusions that could be drawn from the analysis.

The assessment of spring 2006 events revealed that the influence of snow melt is essential for aquifer recharge from the river. That has been the prevailing situation until the present time.

Summer 2007, in total contrast to summer 2003, from a climate and weather point of view, showed that, in spite of the considerable number of surges in the Arve, the role played by the river in naturally recharging the Genevese aquifer was average to negligible. This situation does not compensate for the replacement of "lost" volumes when artificial recharge is discontinued because of high turbidity.

It can be seen, therefore, that 2003 and 2007 (opposite extremes from a meteorological point of view) yielded the same result in terms of the impact on the artificial recharge of the aquifer, namely, high turbidity levels in the river forcing closure of the plant. The suspension of activities was only slightly offset by natural recharge in the summer. Consequently, the conclusion could be drawn that in addition to climate change in the Alps possibly causing higher turbidity levels in the river water used for artificial aquifer recharge, these .conditions also limit the efficiency of artificial recharge.

Fortunately, in spite of the warning issued by climatologists, the 2003 situation has not been reproduced so far. However, we should not forget the risk it posed to the management of the Genevese aquifer. Thorough, continual studies on the river-groundwater relationship are envisaged for the coming years to analyse the impact of climate change on groundwater management and recharge, both natural and artificial.

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Planning Managed Aquifer Recharge in the Emilia-Romagna Coastal Zone (Italy) to Control Salt-water Intrusion and Preserve Farmland, Forests, and Wetlands

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Abstract
Subsidence, sea level rise, droughts, strong seasonal variations in water availability, and saltwater intrusion are threatening agriculture and natural areas along Italy's Adriatic coast south of the Po River (Emilia-Romagna Region). Freshwater in the unconfined aquifer is restricted to isolated lenses floating on saltwater within rows of dunes, isolated in between farmland, up to 5 km from the coastline. Most of the land in between these dune belts is used for agriculture. Once part of a swamp, it was drained and, as a result, it is now below sea level. In summer, abundant irrigation of farmland helps to maintain some freshwater lenses, but other lenses become completely salty. Until recently, the salinity of the unconfined aquifer has been neglected since it was not used for drinking water. Now it is realized that the natural areas and the farmland are at risk, because of the increasing salinization in the phreatic aquifer. The results of this project have shown some of the challenges for MAR in the Adriatic coastal zone including strong seasonality in water availability, high evaporation rates, and high hydraulic conductivity in the aquifer, water scarcity, and strict environmental limits on recharge water quality, subsea level topography, and water management in a polder environment. The scope of our study, helped by analytical and numerical modelling, is to identify areas where artificial recharge may help to store freshwater and prevent salinization of the aquifer and soil.

Keywords: Coastal Forests, Infiltration, Italy, Land use, Seawater intrusion, Ravenna.

Introduction
The coastal area of the Po Plain between the Po and Rubicone rivers is affected by strong land subsidence and saltwater intrusion in its sandy phreatic aquifer (Antonellini et al. 2008). This aquifer is unconfined in the east and confined in the west or where thin veneers of alluvial deposits overlay the sands; for this reason we refer to it as a whole to "surface aquifer" in this paper. In recent years local water authorities and nature administrators have suggested to assess the feasibility of a groundwater recharge system to control saltwater intrusion. In this study we report on this project.

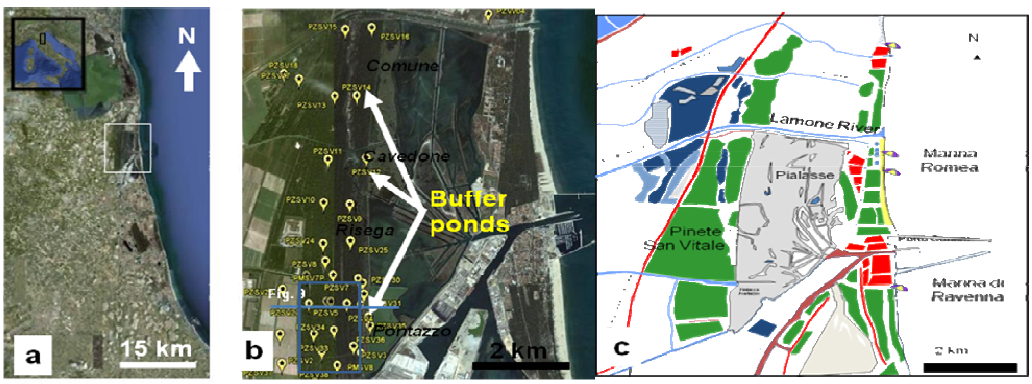

Fig. 1. (a) Location of the study area (white frame inset) in Italy. (b) Detailed aerial view of the study area with important features. The photo indicates the location of the piezometers used to calculate the natural recharge and the position of the buffer ponds. The blue rectangle indicates where an ASR well-injection system was evaluated and the light blue line is the location of the cross section in Fig. 9. (c) Schematic topographic map of the study area.

The surface aquifer in this area is fragmented (Fig.1a-c) by many rivers, canals, drainage ditches, harbours, wetlands, quarries and internal lagoons. The heterogeneous inland distribution of salty surface water bodies (Piallasse, saltworks, lagoons, etc,) increases the vulnerability of the phreatic aquifer to saltwater intrusion. The topography of the area is in many places below sea level and strong natural and anthropogenic subsidence don't allow for strong hydraulic gradients seawards.

Figure 2 shows a schematic stratigraphic section of the last post-Flandrian deposits across the Adriatic coast south of the Po River. The coastal phreatic aquifer is primarily located within the littoral sands, and locally in the shallow marine wedge deposits. The coastal aquifer is unconfined in the east but, 3–4 km from the coast, it is overlain and confined by the most recent alluvial fine-grained continental deposits (west). The thickness of the aquifer varies from a minimum of 8m to a maximum of 30 m (Amorosi et al. 2002).

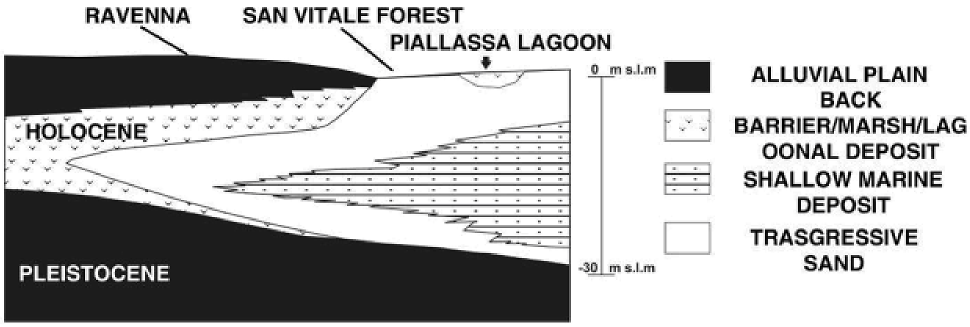

Fig. 2. Schematic stratigraphic reconstruction of the coastal aquifer below the study area (modified after Amorosi et al. 2002).

Subsidence in the area has been caused by shallow groundwater and gas extraction and it has reached a few meters with strong rates in the 60's and 70's. Since then, subsidence has decreased as groundwater and gas exploration was halted. Today subsidence rates along the coast are around 0.8-1.6 cm/yr (Teatini et al. 2006). This means that a lot of territory may disappear below sea level within the next 50 – 100 yrs.

The coastal phreatic aquifer of the Ravenna province is not used for human consumption but it has an important role in agriculture (soil-groundwater interactions, irrigation) and for the natural ecosystems of the coastal area. The coastal ecosystems include forests, wetlands, sand dunes, and coastal prairies and they are managed by the Regional Po Delta Park. Given the important role that natural areas have for the tourism industry and the risk of soil salinization in agriculture, the local authorities tried in the past, and are planning for the future, some measures to mitigate the extensive saltwater intrusion that affects the unconfined aquifer. Among the measures tried in the past are the construction of some infiltration or buffer basins between saltwater and the *San Vitale* pine forest. In this paper we review the results of this experience in view of the recent monitoring data.

A managed aquifer recharge system can be designed only if the boundary conditions of the aquifer and its hydrogeological characteristics are well known (UNESCO 2005). The surface aquifer has been studied extensively by Giambastiani et al. (2007) and Antonellini et al. (2008) who measured water levels mostly below sea level and extremely low gradients often from sea to land as well as anoxic conditions due to the very small hydraulic gradients. Arsenic is a natural pollutant of the phreatic water (Marconi et al. in press) and its concentration is in many places above the law limit for safe use. Another important issue in the area is the strong seasonality of the climate. Water in the form of rainfall and river flood water is available mostly in winter and spring whereas it is lacking in the summer and fall. This seasonal availability of the water puts some constraints on the type of managed aquifer recharge infrastructure to adopt and it needs to be addressed with storage systems appropriate for the local climate

Methods

Some important characteristics of the area need to be evaluated for the planning of an artificial recharge system. These are the natural recharge and the infiltration capacity of the soil as well as the evaporation rate from surface water bodies (both elements are important in designing infiltration ponds). We also characterize the actual state of saltwater intrusion in the area north of Ravenna in proximity to the buffer ponds set in place during the 90's.

Natural Recharge Determination

The yearly recharge in the area is variable; Antonellini et al. (2008) calculated that the yearly recharge in the areas drained by the land reclamation pumping machines is less than 20 mm/year. Here we calculate the average yearly recharge in the coastal dunes and forests with the method of Lerner et al. (1990). According to this method the average yearly recharge (R) in mm is given by

$$R = S_y * \Delta h$$

where Sy is the porosity of the aquifer and Δh is the yearly fluctuation in mm of the water level measured in a piezometer.

The average yearly recharge in the period 2004-2007 in the monitoring piezometers (Fig. 1) of the San Vitale Pine Forest is shown in Fig. 3. The average yearly recharge in the coastal dunes varies from 100 to 200 mm/year. Notice in Fig. 3 the high variability in recharge; the largest values are below the paleodunes where there is a large variation in water table height during the year – the smallest variation are close to the open water surface bodies in connection with the sea. During the wet period rainfall infiltrates the ground and is subsequently drained to the wetlands or the drainage canals. Hydraulic conductivity is rather high ranging from 5 to 120 m/d (Antonellini et al. 2008) allowing for fast draining of the recharge water.

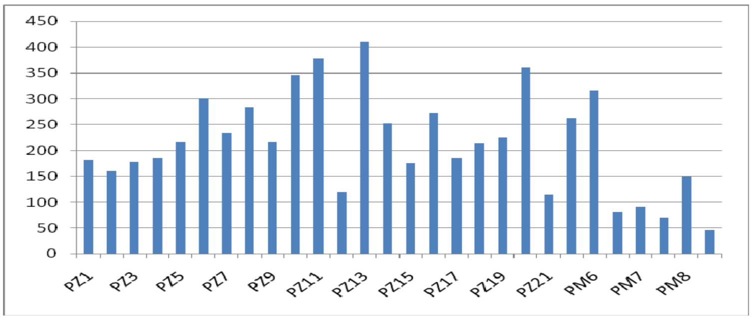

Fig.3. Average yearly recharge in the San Vitale Pine Forest (Fig. 1). Piezometer number on the x-axis and amount of recharge in mm on the y-axis.

It is important to understand the amount of natural recharge in the area to be able to estimate how much artificial recharge is needed in order to reach the tolerance range in water salinity required by the actual vegetation ecosystems (Antonellini and Mollema 2010) and the water needs of the plants in the coastal zone (Mollema et al. in review).

Evaporation from Open Surface Water Bodies

By using the daily values of Temperature (min, max), solar radiation, relative humidity (min, max), and wind velocity provided by the Regional Environmental Agency (ARPA 2010), we computed by means of the Penman equation (Penman 1948) the average daily evaporation rate from open water surface bodies (Fig. 4). Figure 4 reports the data for the year 2008; it appears that the yearly evaporation from open water can be up to 1500 mm/year, which far exceeds the yearly average rainfall of 610 mm/year (ARPA 2010).

This is an important observation, because it means that open water infiltration basins are probably affected by strong evaporation and the amount of water needed to fill them may be large. This is also demonstrated by the fact that the protective belt of recharge ponds of the *San Vitale* Pine Forest (Fig. 1) is dry during the summer period, the season when the water is needed most.

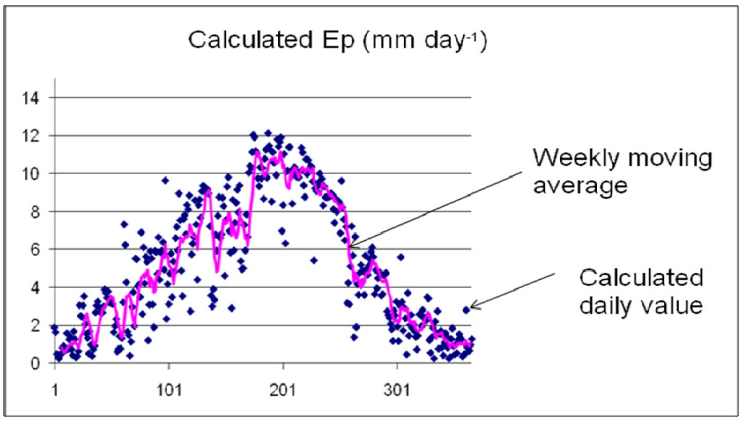

Fig. 4. Calculated daily open water evaporation rate (y-axxis in mm) at Ravenna for the year 2008. The red line represents a weekly moving average.

Salinity Monitoring

During the period 2004-2008, we monitored water and salinity levels in the groundwater and surface water of the area shown in Fig. 5 (*San Vitale* Pine Forest and *Piallassa*).

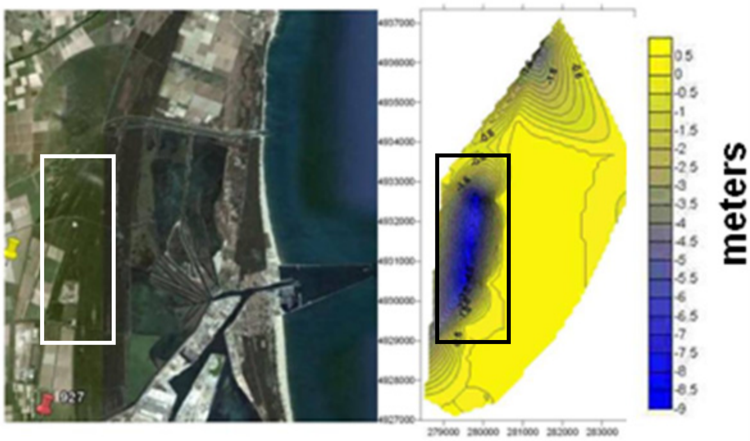

Fig. 5. Isobaths of the salinity isoconcentration surface at 1 g/l. Rectangles show correspondence in area.

This monitoring allowed constructing the average depth of the salinity isoconcentration surfaces during the period of monitoring. Figure 5 shows the depth of the 1 g/l salinity isobaths and Fig. 6 of the 15 g/l isobaths. Figure 5 shows that there is a lens of freshwater below the northern and central part of the San Vitale Pine Forest. Figure 6, on the other hand shows that mixed-brackish water up to a salinity of 15 g/l is also present below the Marina Romea Peninsula and in the southern part of the San Vitale Pine Forest. Figure 7 gives an idea of the thickness of the mixing zone between the concentration of 1 g/l and 15 g/l. It is apparent that where the freshwater lens is most developed (central and northern area), the mixing zone is thin and the interface between saltwater and freshwater is sharp. On the other hand, where the freshwater lens is less developed (southern zone), the mixing zone is very wide and the interface between saltwater and fresh water is diffuse.

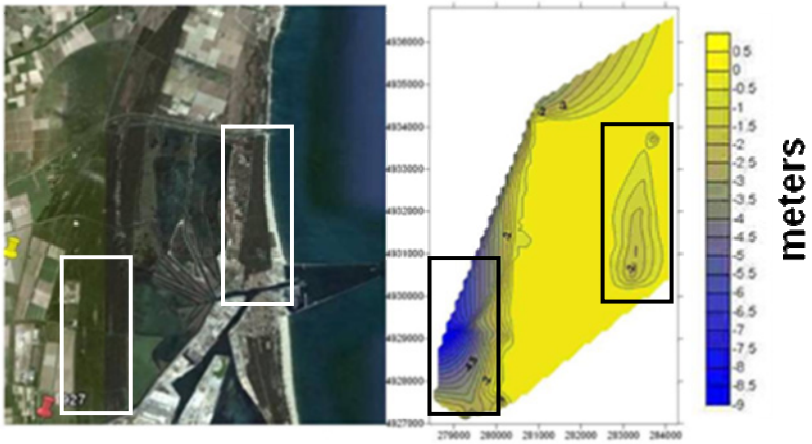

Fig. 6. Isobaths of the salinity isoconcentration surface at 15 g/l. Rectangles show correspondence in area.

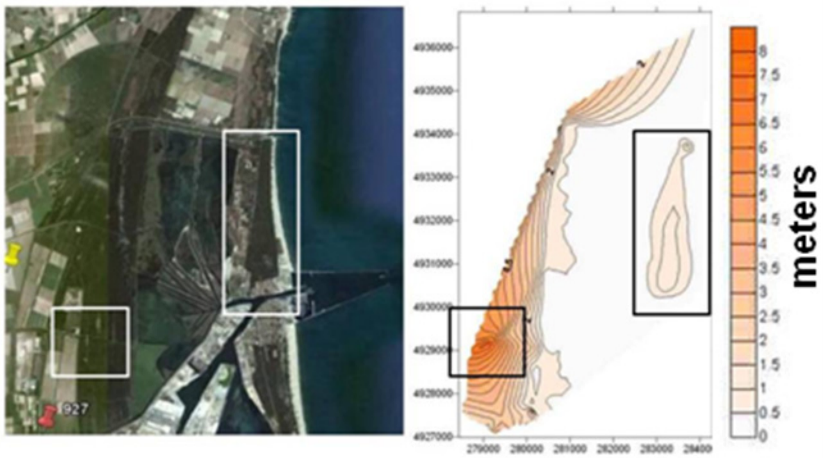

Fig. 7. Isopach of the mixing water zone between 1 and 15 g/l. Rectangles show correspondence in area.

Climate Models and Water Budget

In order to understand the actual water budget (average of last 30 years) in the area and its possible variations in future climate change scenarios, we have computed the different crop evapotranspiration from the typical crops of the Ravenna Province and normalized it to the percent areal extent for each crop in a way to have an average value for irrigated agriculture, horticulture and then for wetlands, open water surface bodies, barren soil, urban areas, etc. (Mollema et al. in review) The crop evapotranspiration was calculated using the FAO (2004) program CROPWAT. We also extracted from several GCM's climate models the future temperature and precipitation forecasts for the period 2070-2100 under the IPCC scenarios A1b and A2 (Mollema et al. in review, Solomon et al. 2007) at the node closer to the Ravenna area. Figure 8 shows the yearly, summer and winter hydroclimatic deficit (precipitation minus evapotranspiration) for a typical coastal basin in the Ravenna Province (Quinto Basin, Mollema et al. in review).

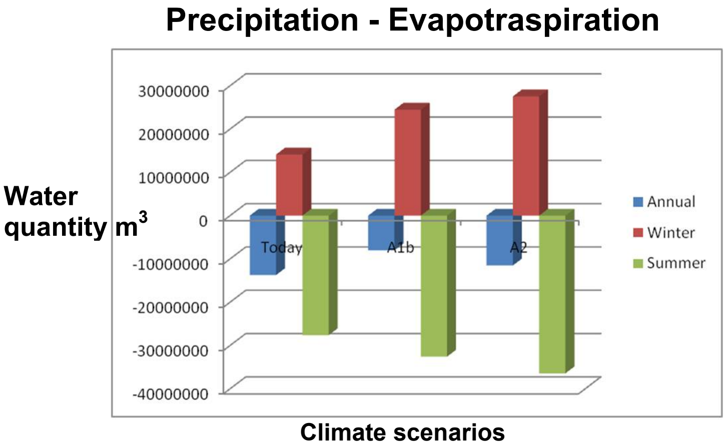

Fig. 8. Comparison of hydroclimatic budgets for the current conditions and for IPPC scenarios A1b and A2 for a typical basin of the Ravenna Province.

The analysis of the diagram in Fig. 8 allows making some guesses on the efficiency of groundwater recharge systems in a future of climate change. Figure 8, in fact, shows that the present yearly hydroclimatic deficit is larger in the present climate conditions than in the future; both A1b and A2 models indicate larger rainfall in the future than today. The precipitation, however, is not evenly distributed through the year; most rainfall will occur in the winter and less in the summer. This is well shown by an increase in water surplus during the winter for both future climate scenarios and by an increase in water deficit during the summer. It appears also that evaporation rates in the future may be smaller given the higher humidity. It is clear from Fig. 8 that in the future there will be more available water during the winter and much less in the summer; a groundwater recharge system has to take advantage of this situation. As of now the excess water is flushed to sea, maybe part of it could be re-infiltrated in the ground or injected in the aquifer during the winter to control saltwater intrusion and satisfy the needs of the coastal ecosystems during the dry periods.

Evaluation of Recharge Methods

The major objective of the managed aquifer recharge in our area of interest should be to fight salinization in the surface aquifer in a way to provide reasonable water quality and quantity conditions for agriculture and the natural ecosystems. Antonellini and Mollema (2010) have shown that in order to have enough vegetation biodiversity in the coastal area, we need surface groundwater salinity values below the 2-3 g/l. This indeed is a minimum objective that we want to achieve with artificial recharge. By using our work explained in the methods section and some previous studies (Giambastiani et al. 2006), we have evaluated the feasibility of different artificial recharge systems.

Artificial Well-Injection Recharge System

Giambastiani et al. (2007) have evaluated the possibility of constructing an artificial recharge system by means of well-injectors along the eastern boundary of the San Vitale Pine Forest (Fig. 1). The idea was to create a hydraulic freshwater barrier down to the bottom of the phreatic aquifer in a way to prevent saltwater intrusion. The planned recharge scheme was made of wells down to a depth of 20-30 m and at a spacing of 200 m. The results of the analytic and numerical simulations have shown that artificial recharge with this method would be possible but would also require large quantities of freshwater (from 2500 to 4000 m^3/day) according to the hydraulic conductivity of the aquifer. For this reason and for the lack of the necessary freshwater, this idea has been abandoned. Another issue is that the strict environmental law requires water with characteristics no less than drinking water standards to be injected in the aquifer. Such water, of course, is not available in the area if not at high costs.

Buffer Ponds

In the early 90's it became apparent to the Po Delta Park Authorities that the historical-roman-time *San Vitale* Pine Forest was suffering, because of saltwater intrusion from the adjacent salty *Piallassa* lagoon (Fig. 1). The city authorities, in order to fight this problem, decided to construct some dikes to isolate freshwater pools in the lagoon adjacent to the forest (Figs. 1, 5, 6, and 7). These pools (*Comune, Buca del Cavedone, Pontazzo*) are now filled with freshwater coming from the land reclamation area inland. The objective of the pools is to create a protective belt between the pine trees and the saltwater and to promote freshwater infiltration in the surface aquifer. During the wet season (November to April) surplus freshwater from the land reclamation pumping machines is directed to the pools that are filled up to 0.1 m above sea level (about 0.5 m from the bottom). During the dry season, most of the area in the pools dries out. Water salinity in this buffer belt varies from 5 to 10 g/l. Notice also that the belt is not continuous given that the *Chiaro della Risega* filled with seawater (Figs. 1, 5, 6, and 7). Another problem that affects the pools in terms of their use as recharge basins is that their bottom is covered by alluvial fine deposits of low permeability (loamy silt to very fine silt) that prevents a connection with the underlying aquifer (Fig. 9).

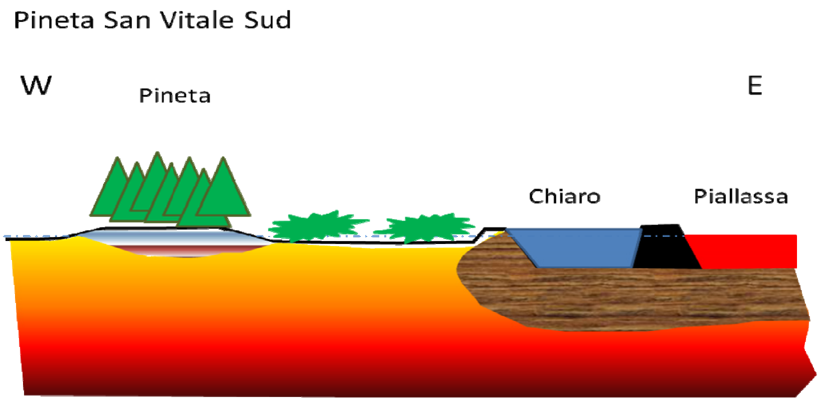

Fig. 9. Schematic cross section through the southern part of the San Vitale pine forest and the Piallassa.

Incidental Recharge

Incidental recharge due to over-irrigation in the area is a relevant artificial recharge method where agriculture is practiced. Mollema et al. (in review) and Marconi et al. (in press) have shown that in a polder basin (*Quinto* Basin) south of Ravenna, incidental recharge from over-irrigation of the corn fields is an important source of infiltration water to fight saltwater intrusion.

Results and Discussion

The work done so far shows that, in order to plan a managed aquifer recharge system in the area, it is important to consider the following aspects. (1) The source of recharge water; the most easily to obtain recharge water is from floods in the nearby rivers. The quality of the water in these rivers might be good enough for infiltration in a recharge pond but not for an ASR or ASTR well-injection system. (2) The high hydraulic conductivity (K = 30 – 120 m/d) and porosity (up to 0.45) of the aquifer are problematic for applying a well-injection system, because they require large amount of high quality injection water for a small increase in hydraulic head. (3) The fact that the surface aquifer is in part confined and in part phreatic requires having a mixed recharge system that needs to couple recharge ponds where the aquifer is phreatic and a well-injection system or deep recharge trenches where the aquifer is confined. (4) The strong seasonality in water availability; flood water to be used for artificial recharge is available only in a few months per year. The problem is to concentrate the recharge when the water is available or to recharge the water during the whole year by using some kind of storage system? (5) The fact that a large part of the area is below sea level and human activities occur only because of the drainage from the land reclamation pumping machines adds another level of complexity in planning an aquifer recharge scheme. Any infiltration and drainage of water needs to be strictly planned in integration between the different water authorities supervising land reclamation activities and ground as well surface water quality. There is an important integrated management issue that needs to be solved before implementing the system.

It is also important that there is an integrated team of people in the planning of the artificial recharge method. An example is the construction of the belt of buffer/infiltration ponds at the eastern margin of the *San Vitale* pine forest. Those ponds, in fact, are located above one meter of alluvial loam and clay that do not provide a good connection between surface water and aquifer and in the end make ineffective the whole recharge plan. In order to make these belt of ponds an effective infiltration area, the bottom of these pools should be scraped and dredged. The problem arising in this case is that the mud at the bottom of the pools is polluted by heavy metals and other pollutants causing important issue in the disposal of the mud. Furthermore, these brackish water ponds have become an important ecosystem for birds, especially during the nesting period, and park managers are worried about possible maintenance activities in these areas.

Conclusions

The different managed aquifer recharge systems examined have advantages and disadvantages. Today, the use of recharge ponds seems the easiest solution to implement in the coastal area of Ravenna. This method would be suitable both for the quality of the water available and for the high hydraulic conductivity characteristics of the surface aquifer. Serious problems related to this method would be the strong seasonality in water availability and the strong evaporation rates. Another problem will be clogging of the bottom of the ponds that will require continuous maintenance or a use in a wet-dry cycle. Some already existing ponds may be used in a more effective way if managed in a way to put in connection their bottom with the surface aquifer.

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Reactions between Chalk and Reclaimed Water and the Implications for Managed Aquifer Recharge in the Chalk Aquifer of the London Basin

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Abstract

The Chalk aquifer beneath London (UK) provides about 20% of Thames Water's public supply to the city, but to ensure its delivery careful water resource management is required. This includes using storage available in the confined Chalk and overlying Thanet Sands via potable water artificial recharge in the North London Artificial Recharge Scheme (NLARS). The scheme can provide about 6% of public supply during droughts, however if some predictions for future increases in demand for water as well as climate change forecasts are correct, pressure on water resources could significantly increase and greater demands may be placed on NLARS. Consequently increased artificial recharge may be required, which could be obtained by using reclaimed water from an Indirect Potable Reuse (IPR) scheme.

A pilot IPR plant is currently being operated in North London, with part of the research programme being to assess treatment performance and the viability of using reclaimed water for artificial recharge of the Chalk. A key consideration is modification of the reclaimed water chemistry and its impact on current groundwater quality. Research encompasses both the system hydrochemistry as well as the hydrogeological characteristics of the Chalk and overlying sands. This paper will focus on the hydrochemical aspects, including the chemistry and quality of the reclaimed water, implications for rock-water reactions within the aquifer, and abstracted water quality. This is being accomplished by interpreting reactions between samples of the simulated reclaimed water and aquifer core, in the context of a known aquifer mineralogy and in situ porewater composition. SEM examinations of the Chalk core have been conducted both pre- and post-reaction to observe and interpret any textural and/or mineralogical alterations. Initial results indicate mobilisation of fluoride moderated by fluorite solubility. As elevated fluoride has previously hindered development of an artificial recharge scheme in the UK Chalk, it is an area where there is much interest.

Keywords

Chalk, Fluoride, Hydrochemical Reactions, Indirect Potable Reuse, United Kingdom, Scanning Electron Microscopy

INTRODUCTION

Location

Large parts of the south-east of the UK are classified as seriously water stressed (Fig. 1a) by the Environment Agency, the environmental regulator of England and Wales (Environment Agency, 2007). One of the regions of serious water stress includes London and its environs where Thames Water Utilities is responsible for water supply (Fig 1b). The study area for this work is located at a Thames Water operational site situated in north-east London (Fig 1c & d).

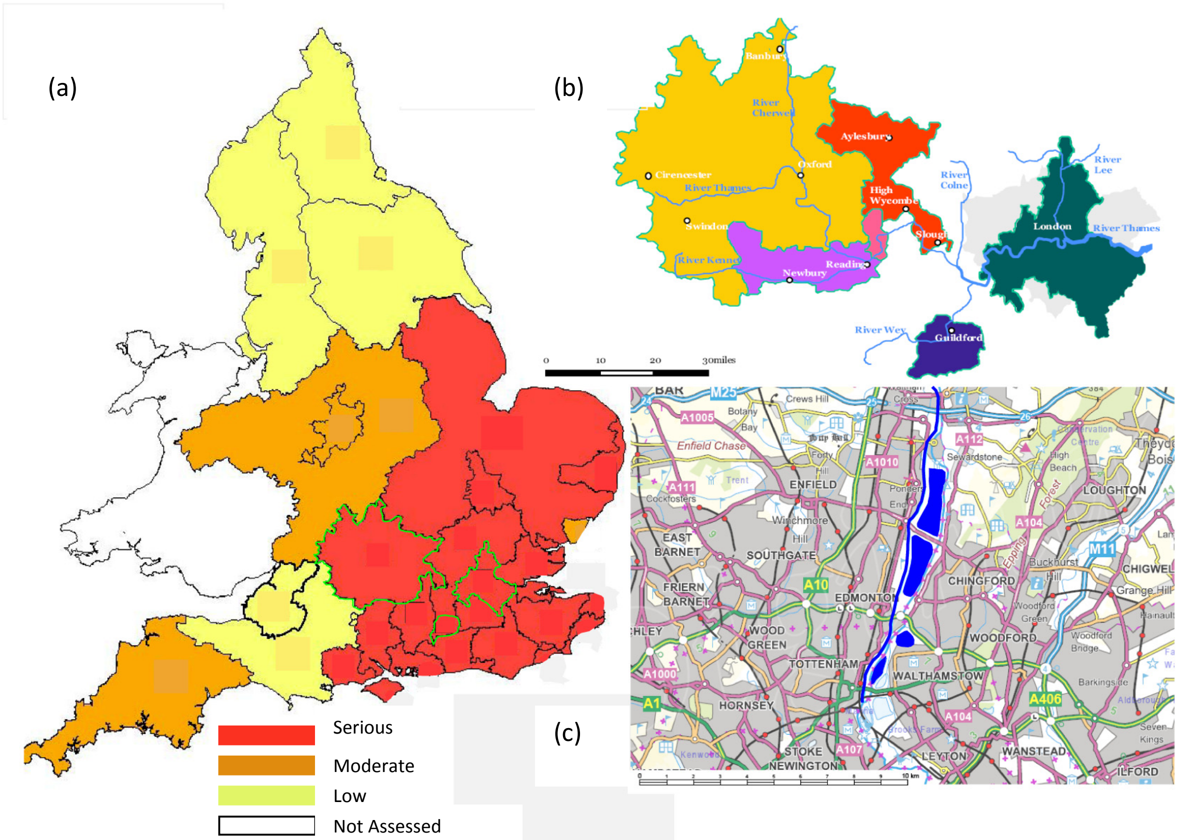

Figure 1 – (a) Map of water stressed areas of England & part of Wales (Environment Agency); (b) Illustration of Thames Water's supply area, showing water resource zones defined for planning purposes; (c) Map of research site location in north-east London

Motivation

The classification as seriously water stressed, highlights the way in which the existing water supplies to London and the South-East of England are increasingly being strained. Some forecasts indicate that a substantial deficit could be established between water resources available and consumer demand and in order to prepare, Thames Water has had to consider new ways in which to supplement available water resources (Thames Water, 2009). Thames Water has set out some of the more well defined options in its recent Water Resources Management Plan, a regulatory planning document covering the period 2010 to 2035.

North London Artificial Recharge Scheme

In 1995, the North London Artificial Recharge Scheme (NLARS) was licensed and began operation. As the UKs only operational artificial recharge scheme for public water supply, it was conceived to be used as a drought management tool, taking advantage of the significant available aquifer storage of the confined Chalk and overlying Thanet Sands. NLARS is an Artificial Recharge and Recovery (ARR) Scheme, which uses potable water from the mains network to augment natural recharge, enabling greater abstraction at a later date as well as replenishment of stored groundwater after use in abstraction mode. NLARS comprises of 48 boreholes, some equipped solely for abstraction and others for use as both recharge and abstraction points. When used for abstraction, these sources discharge water directly into either the New River aqueduct or the raw water reservoir system where they combine with the surface waters, negating the need for a dedicated raw water network for the scheme. The water produced by NLARS is thus treated along with other supplies at the local Coppermills water treatment works before going into public supply. The scheme is currently licensed to abstract an average of 180 Ml/d over the course of a year, but operational constraints restrict recharge to an average rate of approximately 80 Ml/d. A greater artificial recharge rate would be desirable to increase operational resilience and potentially provide options for enhancing the benefits from abstraction during drought.

Indirect Potable Reuse

With the predictions of future deficits between water demand and supply, Thames Water has been investigating a number of options for future potential augmentation of water resources. Since 2005, a research phase was initiated looking at the feasibility and acceptability of implementing planned indirect potable reuse (IPR) in the region. A holistic research programme was devised at the beginning of this work, and has been followed by a series of research studies aiming to demonstrate that such a scheme could meet the water quality, health and environmental objectives agreed with the regulatory bodies in the UK. As part of the research, a $600m^3/d$ pilot plant was designed and is currently operational, using the current state-of-the-art IPR technologies to treat the incoming effluent and generate reclaimed water. The product water generated from the treatment stream is highly purified and has the potential to provide an additional source of recharge to NLARS, not least due to the geographical location of the plant which is also situated in north-east London.

Implications of Initial Research

Investigations into the feasibility of implementing aquifer recharge at NLARS using the reclaimed water from the IPR plant have been ongoing since 2007. Initial work focussed on gaining a more in depth knowledge of the study region through both desk-study work conducted alongside preliminary fieldwork. This research led to a conceptual understanding of the hydrogeological system and a fuller understanding of the important processes at work such as double porosity diffusion effects. One area considered to be highly significant to the viability of the proposed aquifer recharge project was the hydrogeochemical effects of the interaction between the injected water and the aquifer. In previous UK experiences of aquifer recharge, abstracted water quality has been a significant limiting factor (Eastwood & Stanfield, 2001; Gaus *et al*, 2002) and as a result understanding the effects of the dual porosity of the Chalk was also considered a priority for this study.

Initial analysis of the immobile porewaters from the Chalk, as well as preliminary experiments combining Chalk with double deionised water to simulate the injection of reclaimed water into the Chalk aquifer, indicated that fluoride may be mobilised as a result of subsurface water-rock interactions. The fluoride concentrations observed in these initial experiments exceeded those found in the Chalk porewaters, indicating a mineralogical source within the Chalk. Although there seems to be a mechanism controlling the amount of fluoride in solution, understanding whether this occurs due to fluorite solubility or via sorption or mineral precipitation is an significant factor in comprehending the dynamics of the groundwater flow system. Combined with a greater understanding of the fluoride source, this information will be important in making assessments of the potential for deterioration of water quality in the NLARS abstracted water, particularly as a result of artificial recharge using reclaimed water. Conversely, the use of reactive reclaimed water for artificial recharge may provide secondary benefits by dissolution of the Chalk. This may increase aquifer permeability, thereby enhancing borehole development and increasing operational efficiency. Owing to the importance of the water-rock interactions, further experiments were undertaken and are reported here.

METHODS

Experimental Investigations

Simulated Reclaimed Water

These experiments were conducted using pebbles of Chalk material combined with double deionised water to act as a simulation of the reclaimed water product from the advanced treatment plant. 90g of Chalk pebbles containing in situ porewater were put into a Nalgene container to which 103ml of double deionised water (of known pH) was added and the lid tightly sealed. This was done using samples of Chalk from two different horizons in recovered core material, representing depths of 43.18

– 43,14 mbgl (hereafter referred to as 43 mbgl) & 42.20 – 42.16 mbgl (hereafter referred to as 42 mbgl) .

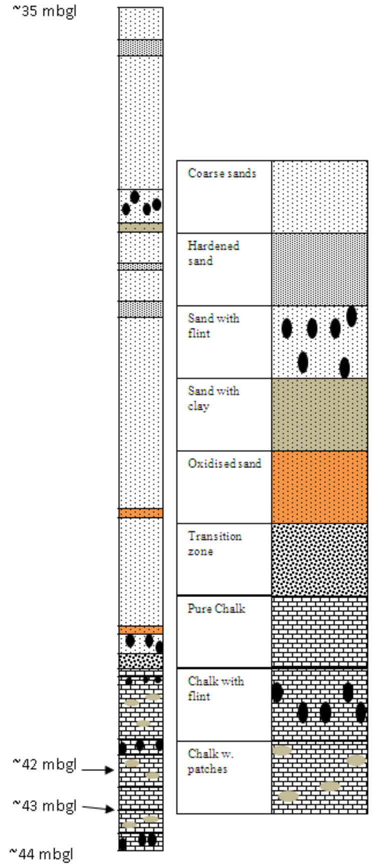

Figure 2 – Representation of core removed at Deephams site showing approx. location of experimental samples

The solutions were sampled at approximately 30 minutes, 1, 2, 4, 6, 24, 48, 72, 96 hours and then at 7, 10, 14 and 21 days by extracting 4ml of liquid with a syringe and filtering this through a 0.45µm filter tip before dividing it two sample tubes. 1ml was put in one tube for analysis with IC (Ion Chromatography) using a Dionex IC 2500, to determine levels of chloride, fluoride, sulphate and nitrate, with the remaining 3ml placed in a second tube and acidified using 50% nitric acid for analysis by ICP-AES using a Jobin Yvon Ultima 2-ICP for the cations calcium, magnesium, sodium, potassium, strontium & iron.

Acidifying the samples leads to some dilution, however as the dilution factor was the same for all samples, it would not have affected the comparison of results between samples. By removing 4ml from the container with each sampling, the volume within the container was reduced, thus increasing the concentrations within the solution. This was overcome by applying a correction to the results using the following equation:

$$Ccor = \left[(Cmes - Cprev) * \left(\frac{Vs}{Vo}\right)\right] + Cprev$$

Where: $Ccor$ = Corrected Concentration (mg/l); $Cmes$ = Measured Concentration (mg/l); $Cprev$ = Corrected concentration of previous sample (mg/l); Vs = volume in reaction vessel when sampled; Vo = Original volume in the reaction vessel

Potable Water Experiments

In this set of experiments, Chalk samples from the same depth horizons were used, i.e. 43 & 42.mbgl. These experiments were then carried out in the same manner as the previous set with 103ml of potable tap water, taken from the mains supply used to recharge NLARS, used in place of the double deionised water. The solution in the containers was sampled using the same technique with the extraction of 4ml at the same time intervals used formerly, with the samples filtered, preserved and analysed using the same techniques described above.

Mineralogical Investigations

Imaging Scanning Electron Microscope

In order to examine the textural alterations which may result from the interaction of the Chalk material with double deionised water, Scanning Electron Microscope (SEM) images were generated using a Jeol JSM-6480LV high-performance, Variable Pressure Analytical SEM with a high resolution of 3.0nm. The images generated were also studied in order to assess whether any sources with potential to generate the elevated fluoride levels observed in the experimentation could be readily viewed within the Chalk samples. Images were generated of the Chalk both prior to the experiments commencing as well as subsequent to being used for testing, to enable comparisons of the condition of the material.

Energy-Dispersive X-ray Spectroscopy

In order to identify sources of fluoride in Chalk samples, imaging SEM is quite a coarse tool and a more accurate and in depth picture can be gained by using Energy-Dispersive X-ray Spectroscopy. Thin sections of the Chalk material were created for use in the SEM with Energy Dispersive System by using either cut sections from the Chalk material or crushed samples mounted in resin onto slides. Using this technique, a full elemental analysis of the material was conducted, showing in detail the presence of any accessory minerals within the Chalk and characterising the elements present within them. Preparation of these samples was complicated by the naturally powdery character of the Chalk material when being cut and polished for the sections, however this was overcome in part by making the thin sections much thicker than standard thin sections.

RESULTS AND DISCUSSION

Experimental Results

Reclaimed Water Simulations

In this set of reclaimed water simulations,the main result highlighted was the marked differences between the chemical compositions of the waters which had been reacting with the Chalk taken from the different depths within the borehole. In the shallower sample taken from 42 mbgl, the calcium, magnesium and strontium were all elevated compared to those found following reaction of the sample from the deeper 43 mbgl. In all of these measured chemical components there was a significant difference in concentrations from the two horizons, but this was most pronounced for the calcium and sulphate concentrations. The calcium and sulphate concentrations increased significantly at the shallower level (Figure 3), with concentrations being around 5-7 times greater in samples taken from the reaction of the material from this shallower 42 mbgl level

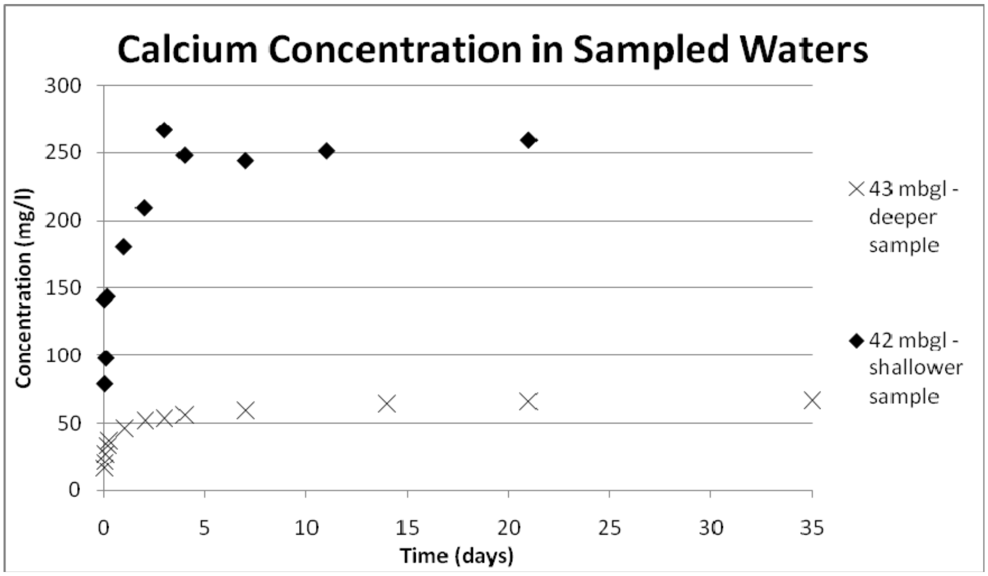

Figure 3 – Measured Concentrations of calcium in reaction waters in experiments with ultra pure water

During this set of experiments. there was also a large increase in sodium concentration followed by rapid decline during the reactions of the sample from 42 mbgl, whilst the measured potassium levels showed no discernable pattern or difference between horizons Elevated fluoride concentrations were also observed in these experiments, with concentrations increasing rapidly to maximum concentrations of ~5mg/l before reducing. Although maximum concentrations of ~9mg/l were measured in the initial experiments noted earlier, both sets of experiments produced concentrations in excess of the porewater concentration indicting a mineralogical source.

The main differences observed during the simulated reclaimed water experiments occurred using Chalk samples from the two different depth horizons used in the experiments. These differences are likely to result mainly from differences in porewater compositions of the two samples, where concentrations of calcium, magnesium, strontium, sulphate and sodium are all higher in the sample taken from 42 mbgl. The biggest differences are observed for sulphate and calcium which can be seen to be directly related to the porewater composition, as these are both highly elevated in the shallower 42 mbgl sample, whereas other components are only moderately increased at this depth.

Potable Water Experimentation

The results of the experiments conducted using potable tap water showed that between both Chalk horizons the concentrations of fluoride, chloride, magnesium, sodium and strontium were all quite similar, while there are moderate differences observed in the potassium and nitrate concentrations between both horizons. However the calcium and sulphate levels differ significantly between the two samples, with the samples from around 42 mbgl having almost twice the concentration of each of these components compared to the sample taken from 43 mbgl. This is likely to be the result of the porewater chemistries affecting the experimental results as previously described for the double deionised water reactions. The fluoride concentrations observed were also elevated but not as much as noted in the initial experiments conducted, although again both sets of concentrations exceeded the porewater value.

Comparison of water types

By examining the differences between reactions with potable tap water and simulated reclaimed water, it was hoped to be able to observe how hydrogeochemical reactions during artificial recharge may change if NLARS recharge were to be augmented by reclaimed water.

The main results revealed when comparing the reactions of these different water types were that the reactions with potable water generated greater amounts of potassium, magnesium, sodium, chloride and nitrate. This indicates that the major contributing factor in these experiments was the source water, which would be enriched with these chemicals compared to the simulated reclaimed water, and that geochemical reactions with the Chalk for these chemical components are not as significant as the source water quality.

Some of the other results of comparing the potable water and simulated reclaimed pure water experiments were less clear. Looking at the levels of sulphate and calcium in the reaction waters, it can be seen that in the deeper sample from 43 mbgl both were increased when the reactions were conducted using potable water, however for the shallower sample, the concentrations of both ions were higher using the simulated reclaimed water. This indicates that the biggest control on sulphate and calcium levels may be the porewater composition at that horizon, as was previously hypothesised.

The results for fluoride concentrations in the reaction waters were perhaps the most unexpected result from these experiments however, with both the experiments conducted with potable water and simulated reclaimed water showing similar results with the concentration increasing rapidly to a moderate to high level but far lower than those observed in the initial stage of experiments run with the Chalk material. Further work is needed in order to understand and explain these observed results.

Mineralogical Analysis

Imaging

The images produced by processing samples using standard SEM imaging techniques showed the structure of the Chalk in clear detail both before and after the experiments had been run. From these images, it was clear to see that the Chalk samples were very pure calcium carbonate. It was also possible to see the occurrence of some accessory minerals, but with imaging SEM it was not possible to identify them. Comparisons of Chalk samples imaged prior to testing as well as images of post-experimental samples showed that there was no readily identifiable alteration to the Chalk structure or texture during the experimentation (Figure 3).

(a) **(b)**

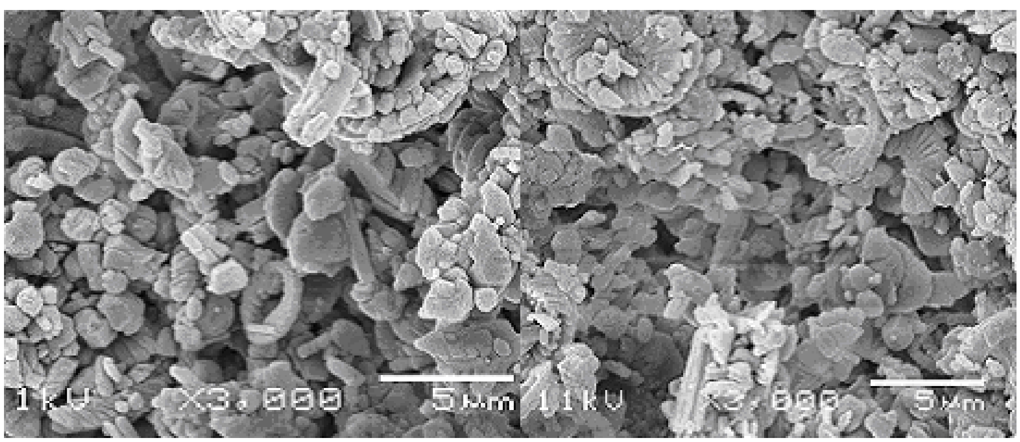

Figure 3 – Chalk SEM images (a) Chalk sample before experimentation; (b) Chalk sample following experiments

Elemental Mapping

The results of the elemental mapping are likely to prove to be useful basis for further examination and interpretation, however due to the complications involved in producing the necessary slides using Chalk material, the plots of the chemical composition of the Chalk using this technique have yet to be completed.

CONCLUSIONS

Overall, the experimental results showed that the main controlling factor on the reactions of introduced water with the chalk core was the porewater composition of the chalk core, suggesting that the geochemical reactions within the experiments are dominated by diffusion rather than dissolution. However, from the elevated fluoride levels observed, it was clear that there is some dissolution occurring during the experiments, although the concentrations observed in this set of experiments was much lower than that recorded previously in Phase 1 of the work. In additional the reasons for the subsequent decline in measured fluoride in the reaction waters is as yet unclear and further work will need to be undertaken in order to thoroughly explore and further assess this occurrence. The concentration of fluoride in the porewater samples from both depths is approximately equal and is significantly below that detected in the reaction waters and therefore seems to have had little influence on the measured final concentration in the reaction waters. Nevertheless the level of fluoride observed in the reaction waters in these experiments is still approximately 3 times the World Health Organisation recommended limit for drinking water of 1.5 mg/l and is therefore still of concern. The occurrence of fluoride in the Chalk needs to be thoroughly evaluated. The results of the Energy-Dispersive X-ray Spectroscopy should provide guidance in the appraisal of the accessory mineral assemblage of the Chalk and the effects this will have during water-rock interactions.

The marked difference seen between the different horizons evaluated in the experiments may indicate that the experimental technique used may not be the most suitable method for evaluating the geochemical reactions of injecting water into the Chalk. The samples used in the experiments were taken from slices of aquifer material of just 4cm thickness; however water entering the borehole for aquifer recharge purposes at the site where this material was collected would not be targeted at a specific horizon within the aquifer. This may mean that the focus of the experiments conducted so far has been too narrow and that further modifications may be required. An improved experimental technique may be possible to formulate if the reactions are conducted using Chalk material coming from throughout the depth of the borehole. The progression of this work is likely to be guided in part by the results of the elemental mapping which may indicate some of the causes of the elevated fluoride observed, as well as the results of modelling which is currently taking place. Modelling should indicate the likely movement of the injected water, as well as simulate the effects of diffusion, which can then be compared with the experimental outcomes.

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CLOUDBREAK: THE CRITICAL ROLE OF MAR IN A MAJOR PILBARA MINE WATER MANAGEMENT SCHEME

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Abstract

The Cloudbreak iron ore mine in Western Australia's Pilbara region is hosted by the Marra Mamba Formation aquifer, which contains a lens of fresh to brackish groundwater underlain by a hypersaline groundwater system. This groundwater system extends beneath the Fortescue Marsh – an internally-draining wetland of national significance. Ninety percent of the orebody is below water table (BWT) and the dewatering yields are very high and far exceed site requirements. The setting and scale of the Cloudbreak project mean that dewatering and effective water management are integral to the success of the operation. Managed aquifer recharge (MAR) has been adopted as a primary water management tool since mid-2008 to bank fresher waters for future use and to maintain ecological water requirements.

A set of water management principles has been developed for Cloudbreak and its sister site Christmas Creek, which adjoins Cloudbreak and will ultimately be of similar scale. The waters of the two operations are managed together through the Papa Warringka water management scheme, a term in the language of the Nyiyaparli traditional owners that means ''water in the ground'.

The scale and complexity of the Cloudbreak MAR scheme are such that there are unique operational requirements, including an extensive trigger level system to ensure ecological values are maintained; separate water management systems for different water types; and long-term design features that facilitate future intra- and inter- mine water conveyance. The paper presents an outline of the Cloudbreak MAR scheme including some of the learnings made during it's graduation from hasty beginnings.

The Cloudbreak scheme showcases how MAR can be used to meet a range of objectives in a complex environment. MAR is an ideal water management tool for the Pilbara region, which is characterised by high-yielding and often fresh aquifers, low rainfall, and a proliferation of major BWT mining developments. However MAR has application many environments, and is likely to be used increasingly at minesites around the world facing varied and difficult water management challenges.

Key words: MAR, injection, Fortescue Marsh, Cloudbreak, mining, Papa Warringka, saline

INTRODUCTION

Mining and Water Management

The Cloudbreak iron ore mine lies along the southern flanks of the low-lying, east-west trending Chichester Range in the Pilbara region of Western Australia (see Figure 1). The mine is bound on its southern side by the Fortescue Marsh, an internally draining, ephemeral wetland of national significance that is underlain by a hypersaline groundwater body. The ultimate Cloudbreak mining area will extend over a strikelength of 40km with a width of one to three km. Mining progress requires substantial dewatering of the alluvial/colluvial overburden and the orebody aquifer, 90% of which lies below the water table (BWT) in a brackish aquifer overlying a saline wedge emanating from the Marsh. The setting and scale of the Cloudbreak project mean that dewatering and effective water management are integral to the success of the operation. Aquifer injection has been adopted as a primary water management tool and has thus far proven to be practical and effective.

Dewatering for (BWT) operation and reinjection of the excess water began in August 2008 and the dewatering abstraction is currently 25 GL per year. The site requires 5GL per year for ore processing and dust suppression, and the remaining 20GL (80% of dewatering) is returned to hydrochemically-similar aquifers through injection bores. Brackish water is injected laterally along the orebody into future mining areas, and saline water is injected south of the operations into a highly permeable calcrete aquifer with natural salinity of 40,000 to 150,000 mg/L. Current dewatering is primarily brackish but this will change in the future as mining extends further south and water is increasingly drawn from the Fortescue Marsh saline wedge. Saline water will be injected to aquifers with higher background salinity and will require a higher level of monitoring and control during both surface conveyance and injection.

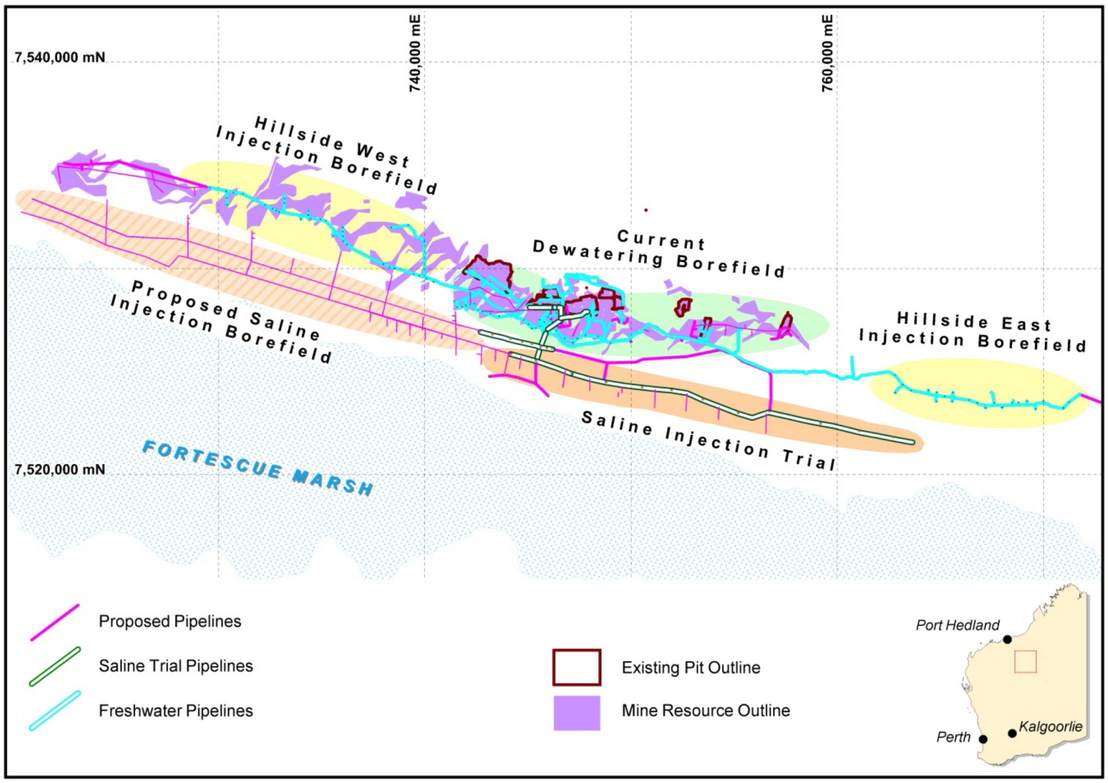

Figure 1: Cloudbreak Water Management System

There are a number of drivers for injection of the excess dewatering draw. The first is the preservation of the brackish water resource for ore processing over the life of the mine, without which Cloudbreak would have to establish a large external water source relatively early in the mine life. The second benefit of injection is to minimise the drawdown footprint of the dewatering operation, particularly toward the Fortescue Marsh where ecological functioning is poorly understood but ecological values are known to be high. A third and important driver is to prevent environmental and cultural concerns associated with the surface discharge of excess water. Prolonged discharge in an area which is dry most of the year round may lead to dependency of vegetation and associated fauna, and is an issue of note for Pilbara mining operations. Surface discharge is also discouraged by the traditional owners of the Cloudbreak project area, the Nyiyaparli people, who believe it disrupts the natural way of the land.

Dewatering at Cloudbreak will increase and is expected to reach around 50GL per year, of which 80% will be reinjected. Fortescue Metals Group also operates the Christmas Creek iron ore mine on the eastern side of Cloudbreak. Christmas Creek is yet to mine below the water table, but mining and dewatering will ultimately be of similar scale to Cloudbreak. The same water management principles will be applied and the waters of the two operations will be managed together through the Papa Warringka water management scheme, a Nyiyaparli term for ‘water in the ground’.

Natural Setting

The Pilbara is a semi-arid region of very hot summers and mild winters. Average rainfall at the nearby town of Newman is 310 mm per year. Rainfall is generally associated with cyclonic activity and occurs mostly as intense events during the summer ‘wet season’ (October to April).

The major regional drainage is the ephemeral Fortescue River. The Goodiadarrie Hills, 50 km west from Cloudbreak, effectively cut the Fortescue River into two separate river systems. West from the Goodiadarrie Hills, the Lower Fortescue River catchment drains to the coast, whereas east of the hills the Fortescue Marsh receives its runoff from the Upper Fortescue River catchment. The Fortescue Marsh forms an extensive intermittent wetland occupying an area about 100 km long by typically 10 km wide, at an elevation of around 400-405 m above sea level (mASL). To the north, the Chichester Range rises to over 500 mASL, and to the south the Hamersley Range rises to over 1,000 mASL. Following significant rainfall events, runoff from the Upper Fortescue River catchment drains to the Fortescue Marsh and creates isolated pools or may flood the entire Marsh area.

The Fortescue Marsh is listed as “Nationally Important” in the Directory of Important Wetlands in Australia. It holds ecological significance as an important site for flora and fauna populations including samphire shrub, Australian Pelican, Black Swan, Bilby, Northern Quoll and Night Parrot. It has been nominated for RAMSAR listing.

The Cloudbreak deposits lie within the Hamersley Basin where granitoid rocks of the Pilbara Craton are overlain by Archaean and Proterozoic sedimentary groups that include the Marra Mamba Formation orebody. The Fortescue Marsh is underlain by a flat-lying but complex sequence of Quaternary and Tertiary alluvial, colluvial and lacustrine sediments. The alluvial deposits increase in thickness down gradient towards the Fortescue Marsh, where a maximum thickness of approximately 70 m has been recorded and as seen in the schematic section shown in Figure 2.

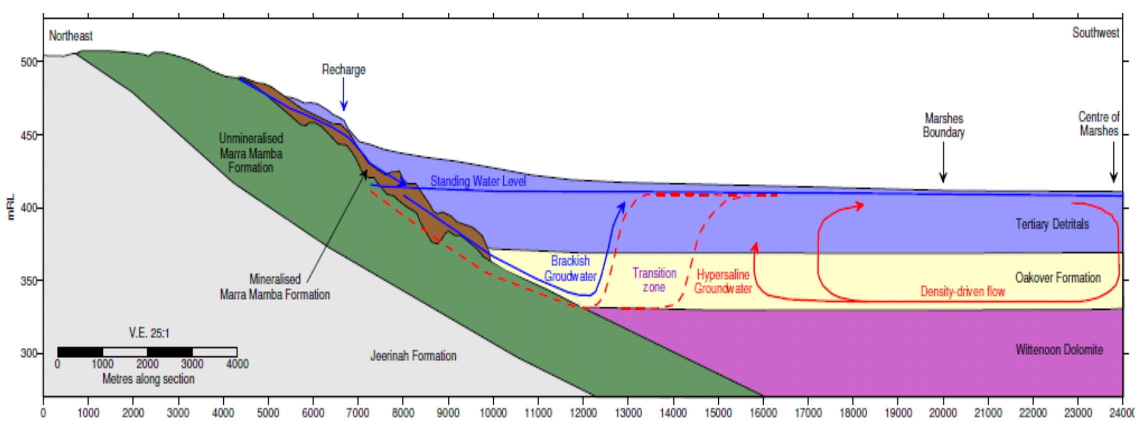

Figure 2: Cloudbreak and Fortescue Marsh Schematic Section

The most permeable hydrogeological units are the orebody and the calcretes of the Oakover Formation. The orebody is developed through supergene alteration of the Marra Mamba Formation, a banded iron formation (BIF) unit. The mineralisation process enhances permeability and the Marra Mamba ranges from highly permeable where mineralised to low permeability in the unmineralised BIF. The calcrete unit onlaps the Marra Mamba Formation, variably onto mineralised orebody and unmineralised BIF, and is thought to extend beneath the Fortescue Marsh. As such it is the chief conduit of saline water from the Marsh, and primary connector of water level change in the orebody to the Marsh. The Tertiary sediments overlying the Proterozoic Marra Mamba are generally of low permeability and contain effective aquiclude clay lenses.

There are two contrasting groundwater flow systems in the Cloudbreak project area. The first is topographically-driven flow of rainfall recharge on the Chichester Range and immediate flanks to the lower topography of the Fortescue Valley. Most groundwater stored in the orebody aquifer is derived from Chichester recharge and salinity ranges from fresh (less than 1000 mg/L) in localised lenses to about 6,000 mg/L. The other flow system is driven by the groundwaters beneath the Fortescue Valley, where salinity can exceed 130,000 mg/L. These waters evolved through evapo-concentration and migrated downwards due to the high density of the resulting brines. The resulting saline wedge meets the fresher lens in a transition zone near the southerly extent of the Cloudbreak deposits, as shown in Figure 2. The shape, size and positioning of the wedge are controlled by a number of factors including the magnitude of the density contrast, the hydraulic and dispersive properties of the aquifer, stratigraphy, structures, the groundwater head in the freshwater regime and hydraulic head from the Fortescue Marsh during periods of flooding.

CLOUDBREAK MAR SYSTEM

Groundwater Management Strategy

Aquifer injection is an integral part of a broader groundwater management strategy employed at Cloudbreak. Key points in the strategy are:

1. *Inclusion of water management as a key parameter in a 'total' mine planning process.*
   Water management principles are given high priority in planning pit sequencing and scheduling so that dewatering abstraction and saline water extraction can be minimised.

2. *Aquifer injection as the principal excess water management method*.
   Aquifer injection is a key tool for groundwater management.

3. *Separate water management streams for brackish and saline water*
   The different use and disposal characteristics of brackish and saline water are accomodated through separate conveyance, storage and disposal infrastructure.

4. *'Banking' of brackish groundwater for future recovery*
   Cloudbreak's brackish water requirements for ore processing over the life of the mining are thought to be similar to the amount of brackish water held in aquifer storage within the mining area, hence brackish dewatering is 'banked' by reinjection for future use.

5. *Targeted injection of excess water*
   Injection of saline water into saline aquifers to the south of mining serves two purposes: prevention of environmental harm caused by saline water discharge, and minimisation of the drawdown footprint toward the Fortescue Marsh to the south.

Groundwater Management System

The water extraction, distribution and injection system at Cloudbreak is extensive and growing. It currently includes over 200 abstraction bores (60 operational at any time), ten in-pit sumps, 200 km of pipeline, six major transfer or settlement ponds, 100 brackish injection bores (50 operational, others for standy or future use) and 20 saline injection bores (currently operating five at a time on a trial basis). The progressive nature of the mining operation drives continual expansion of the water management system to deal with new mining areas, and redundant bores and sumps are regularly mined through.

Mining currently occurs in the central area of the lease and dewatering is achieved through a network of abstraction bores and sumps. The system is equipped to manage four separate water quality streams: being brackish bore water; brackish sump water; saline bore water and saline sump water. Currently no saline sump water is being produced.

The network comprises a mix of open and closed pathways to injection. Most bore discharge water is pumped to open transfer ponds which serve as both pressure breakers and storage buffers. The pressure breaker function is necessary as pumping routes are commonly over 10 km and can be as long as 30 km. Bore discharge water from one pit (Brampton) flows direct to the Hillside West injection borefield without daylighting to interim ponds. Sump water is routed to settlement dams to reduce sediment loading before the water is transferred for injection or use in ore processing.

Water is classified as brackish for water management purposes if total dissolved solids (TDS) is less than 6,000 mg/L and saline if it is higher. 6,000 mg/L was adopted as it is the approximate lower end of the transition zone (roughly 6,000 mg/L to 20,000 mg/L) in which salinity changes rapidly, and is also a reasonable upper limit for beneficial use by vegetation or cattle, although salinity tolerance for some vegetation may be higher.

Brackish water is currently injected to the Hillside West and Hillside East borefields, which lie along-strike and tap the mineralised orebody in future mining areas. Each borefield is approximately 20 km long. There is no filtration of water before injection.

Only small amounts of saline water are currently produced and hypersaline water is only pumped from a deep-set structurally-controlled feature in a single open pit (Hook). This water is pumped to a lined saline transfer pond and was previously used for in-pit dust suppression or shandied with fresher waters for injection to the brackish borefields. Since October 2009 the saline water has been injected into a saline aquifer for the Saline Injection Trial as described further below.

Different styles of downhole valves have been trialled on-site to prevent cascading into injection bores and tests are still continuing. Most injection bores are equipped with a simple device comprising a poyethylene plug inside a slotted sleeve made of ABS at the base of the flexible hose downtube. The plug is attached by a chain to a hand-driven winch at the surface. Lowering the plug exposes more outlet area in the ABS sleeve which reduces the resistance to flow, allowing the backpressure to be adapted to changes in flow rate.

More advanced downhole valves are being trialled through the Saline Injection Trial. These valves employ hydraulic pilot valves to control a hydraulic ram to drive the downhole plug up and down. The plug moves automatically to achieve a set pressure (say 50 kPa) at the surface.

Maintaining constant flow rates to each injection bore is an operating aim of the system. This has to be balanced against the dynamic dewatering needs and the effects this has on the total flow for injection and the pressures in the system, which will alter individual injection rates. The current system is manually operated (other than the saline injection trial) and requires a high level of management. At least some parts of the system are likely to move toward remote automated controls.

Saline Injection Trial

Injection of saline water is being conducted at a pilot scale to prepare for the large scale saline water management that will come at Cloudbreak and Christmas Creek. Saline water requires a higher degree of control in conveyance and injection due to the potential for environmental damage from spills, and also produces challenges for equipment relating to corrosion and chemical precipitation. The saline trial was conducted in part to better understand the infrastructure and management requirements before rolling out to full-scale operation. It also serves as a low-impact aquifer investigation: saline water can't be discharged in an uncontrolled fashion, so pumping tests are limited to a few hours duration by the capacity of the water containment dams. The saline injection trial allows a comprehensive aquifer investigation without discharge. The trial was also used to test new equipment for more general use on site, notably telemeterised SCADA systems to control flows and collect real-time monitoring data.

Saline source water for the trial comes from a localised deep fractured zone of elevated salinity in Hook Pit and is pumped to a lined dam. Water is blended in the injection trunk line with fresher waters to lower the salinity to the lowest salinity observed in the injection bores. Another method of injection regulation under review would remove the injectant salinity limit and instead mandate that the salinity of aquifers with higher beneficial use (eg fresher overlying aquifers) are not impacted.

Twenty saline injection bores are employed in the saline trial on a rotating basis and three to five bores are active at any one time. Water is injected into the permeable calcrete aquifer of the Oakover Formation, which is overlain by a 20 m thick clay lens that effectively separates the lower saline aquifer from the fresh to brackish water found in the upper detritals.

A number of automated and manual controls are in place to ensure environmental protection and compliance with license conditions. Automatic controls include shut down of injection to a bore if the water level rises to within three metres of the surface; and cessation of saline water pumping if leakage is indicated by an automatically-calculated water balance. Manual controls include regular visual inspections, water level and salinity triggers in monitoring bores, and vegetation monitoring.

SYSTEM PERFORMANCE

The initial Cloudbreak injection system was constructed rapidly to meet an urgent need for operational injection capability. The early performance of the injection program was below expectation, which prompted review and a number of improvements carried out during 2009. Whilst developments and learnings continue, the system is now effective in meeting the water management requirements of the site.

The first injection bores at Cloudbreak were constructed with slotted PVC casing. This led to low initial well efficiency that was rapidly decreased by material clogging of the slots. The clogging was enhanced by poor scouring practice in the first, rapidly constructed, pipelines. A bore design with open (uncased) holes across the Marra Mamba target aquifer was trialled and the bores held open and delivered markedly better hydraulic performance. The site's standard injection bore design since then has included open holes across the target aquifer and several earlier bores were redrilled. Sustainable flow rates to some of the redrilled bores increased tenfold. Pipeline scouring and commissioning approval procedures were modified to ensure a clean network before start of injection.

Individual bore injection capacity has been highly variable, ranging from less than 5 L/s to 70 L/s. This reflects the nature of the mineralised Marra Mamba aquifer and the calcrete aquifer and is consistent with the range in pumping yields in the dewatering bores.

The plot in Figure 3 compares the specific capacity of individual bores from the saline trial under pumping and under injection, both measured after 48 hours. The first few bores were constructed without downhole valves and it can be seen that aeration from cascading pushed the bore water level up and gave a poor representation of the aquifer level. The cascading also caused vigorous vibration of the headworks that was enough to dislodge flow control valves in some cases. Downhole valves were installed in subsequent saline injection bores and retrofitted in the earlier set.

The three bores initially fitted with valves show that specific capacity during pumping is a reasonable predictor of specific capacity during injection in the saline trial area. The injection yield is about two thirds the pumping yield, which may be a result of aquifer pressurisation during injection as the aquifer is semi-confined. Injection capacity of bores in the orebody aquifer is similarly predictable from pumping performance.

Bore and aquifer clogging have to date proven to be manageable, particularly since the removal of slotted sections in the injection bores. In the early months of the Hillside West injection borefield, when injection capacity was a limiting factor on the ability of the site to manage water, bores were redeveloped by airlifting when the injection capacity fell by around 30 to 50%. Airlifting was generally effective in restoring the full capacity of the bore. This full restoration and the coarse detritus found lodged in flowmeters and caked on the downhole valves indicated that the main source of clogging was plugging by materials introduced in the pipes or the transfer ponds. Most bores needed airlift redevelopment within three to six months of operation. However since greater attention has been given to keeping detritus out of the system the loss of bore capacity has been much slower. Combined with the construction of additional injection bores and only a slight increase in the overall dewatering rate, the need for redevelopment has dropped back.

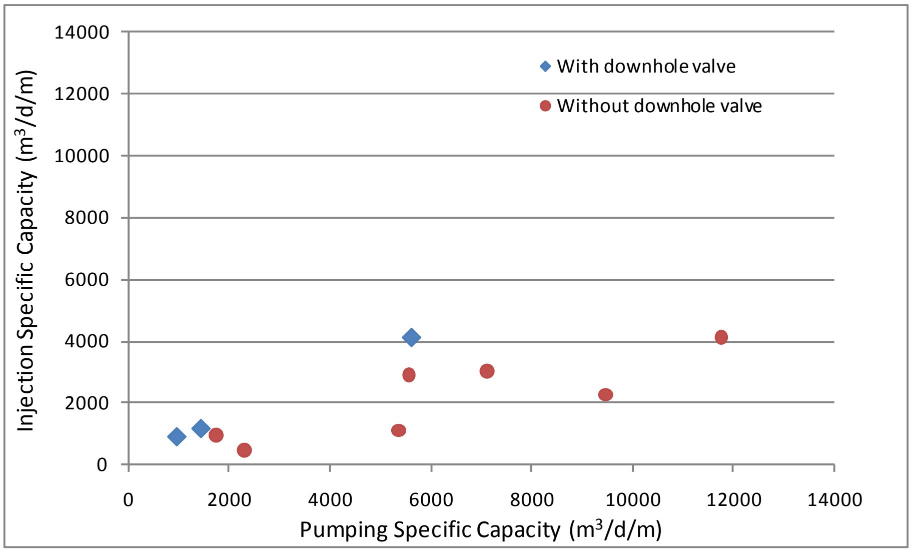

Figure 3: Specific Capacity of Bores During Pumping and Injection

Geochemical modelling was carried out to assess the potential for mineral precipitates to form and plug the boreholes during saline injection. Water chemistry and groundwater hydrogeological data were used to define a series of solution mixtures that would result from injection of abstraction water into the subsurface aquifer. The potential for mineral precipitation (e.g., calcite, silica, gypsum, etc.) for the solution mixtures was assessed with the PHREEQC geochemical model. The Pitzer thermodynamic database (pitzer.dat) was used for solution speciation calculations of major ion chemistry because of the high ionic strength of the groundwater. The assessment found the chemical compositions of groundwater from abstraction bores were very similar to the receiving waters in the injection bores. Both the abstraction and injection zone groundwater compositions are saline and dominated by Na and Cl. Water from brackish abstraction wells is less saline and has more Ca, Ma, and SO_4 relative to the abstraction and injection zone groundwater. The degrees of saturation with respect to potential mineral precipitates are approximately the same for both the abstraction and injection zone groundwater.

The geochemical assessment found that significant mineral precipitation was unlikely despite the high saline content of both the injected and host waters, essentially due to the chemical similarity. The operation of the saline injection trial has supported this finding and very little clogging has been observed in the saline injection bores. Given the saline injection bores have only operated for eight months at time of writing it may be that persistent precipitate or biological scales requiring more aggressive bore remediation will develop, however the record to date suggest these processes are unlikely to be a major burden to operations.

The Cloudbreak numerical groundwater model is a key planning tool for dewatering and injection. The model is constructed in the FEFLOW (5.4) finite element code, selected due to the importance of density-driven flow to the regional hydrogeological setting, and due to the need for higher resolution in a number of discrete areas around the site. Modelling has been used to predict the future recirculation of injected water back to dewatering will be roughly 10 to 15%. Operation of the Hillside West borefield broadly validated the numerical model in this area whilst identifying that permeability and storage in the mineralised Marra Mamba were somewhat higher than previously thought. As a result the drawup cone from injection is flatter and broader; drawup at individual bores is lower than expected and the drawup propagates further from the borefield.

The performance of the Hillside West borefield highlights a milestone reached in the operation of the Cloudbreak injection borefield, where it is now understood that constraints on regional water level, flow or quality are likely to place stronger limits on injection capacity than the individual bore capacity. The challenges of hydrogeological characterisation and management are still present, but early questions about the feasibility of reinjection as a mine water management tool on this scale have been settled.

CONCLUSION

The Cloudbreak MAR scheme has quite rapidly graduated from a system with teething difficulties to a well-functioning operation. The feasibility and practicality of a large-scale aquifer injection system for management of the mine's waters has been demonstrated. The total dewatering abstraction at Cloudbreak and neighbouring Christmas Creek may ultimately exceed 100 GL per year and the array of injection bores and pipework will continue to grow to service the site's evolving dewatering needs. Along with this growth, the focus for injection management at Cloudbreak and Christmas Creek will be characterisation and optimisation of the system to reduce the numbers and maintenance of bores, and the ongoing investigation of the regional hydrogeology in this highly complex natural and managed environment.

The Cloudbreak scheme showcases how MAR can be used to meet a range of objectives in a complex environment. MAR is an ideal water management tool for the Pilbara region, which is characterised by high-yielding and often fresh aquifers, low rainfall, and a proliferation of major BWT mining developments. However MAR has application in many environments, and is likely to be used increasingly at minesites around the world facing varied and difficult water management challenges.

ACKNOWLEDGEMENTS

Simon Windsor, Stephen Collett and Ian Brandes De Roos all made strong contributions to the work described, and the Dewatering team at Cloudbreak has been outstanding in the construction and operation of the Cloudbreak MAR system.

Mine voids as aquifer – Mine water reuse

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Abstract

Mining sector is traditionally associated with a potential impact on water resources. The advances developed for the prevention of mine water pollution and mine water decontamination have opened new horizons for mine water reuse, which can be a complement to the primary water resources. During the closure phase of the mine and after pumping water suspension, water in mine voids can be considered a pseudo-karstic aquifer acting as underground dams. Mine water stored in underground mine works constitute real underground dams similar to the surface dams, but they do not suffer evaporation losses and they are available for its use in drought periods as a complement to the conventional water resources. There are many areas of the world with serious shortage of water resources; on the other side, the increases of traditional operations that are water consumers, such as agriculture, generate a great pressure to the hydric systems. Then, the search for alternative water resources has been promoted, and the use of mine waters has become especially relevant in countries with historic mine districts. In the same way water reuse has explored a great development on the last years, studies on the reutilization of mine waters have developed new fields of application which are mainly focused in three sectors: low enthalpy geothermal energy, electrical energy generation and supply for industrial areas. Thus, mine waters can be considered not only a waste, but also a potential water resource that can be used for a great range of uses in the industrial and environmental field. A large number of projects in the world are focused on mine water reuse, some examples are considered in this paper.

INTRODUCTION

Mining and Environment have always been associated in a negative sense. There are many historical mining districts in the world where acidic mine drainages have produced severe pollution episodes in soils and waters downstream the mine operations. Currently, advances in mine water decontamination have opened new horizons and potential for mine water reuse in different fields, complementing the primary water resources. The potential for reuse of acidic mine water that have been treated is high (Jovanovic et al, 1998; Du Plessis, 1983), such as in Mpumalanga province, in South Africa, where mine water from carboniferous deposits are being reused for irrigation of fields (Jovanovic et al, 1998), or in Jordan, where mine water is reused for irrigation of golf fields after desalinization (Rimawi and Jiries, 2009).

The availability of groundwater resources in some areas is conditioned by the economy and the lack of access to technology for the exploitation of these resources. It is quite usual in mining to reach depths that are higher than thousand meters blocking aquifers that imply a problem but which can be used for recharging shallow aquifers. An example of this methodology can be found in the regions of Illawarra and Wollondilly (Australia) where Illawarra Coal's water filtration plant reinjects 2 mega litres per day of mine water (Australia Government, 2008).

Mine activities create voids and new fracture systems that alter the natural hydrogeological system of the area. When mining activities cease and the pumping stops, the underground mine works constitutes a water reservoir in which the water quality is dependent on several factors including the oxygen status of the void, pH, hydrogeological flow system, composition of wall rock, concentration through evaporation, biological activity and hydrothermal inputs. In some cases, surface and underground mines where water quality needs to perform a first stage of treatment can also be used as water reservoirs suitable for industry and environmental activities. Water drains from mine works are commonly associated with environmental threats; however, not all mine waters are polluted. Historical documents show that the use of mine water has been considered in the past, the roman historian Plinius, mentioned yet in some of his treatises the use of the mine waters for the supplying of populations near the mines (Rackham,1952). With the industrial revolution, there are references about the existence in XIX Century in Matlock (England) of a modest spa for the people of the area where the water came from mines in Lower Carboniferous limestone (Naylor, 1983). There are more references of the use of mine waters such as in Carrhouse in England (Younger, 1996) or Linares or Caceres in Spain (Linares Council, Caceres Council, 2007) where the waters from the mines have been used by the near populations for different daily uses.

In many areas of the world, the rainfall reduction, together with the increase of evapotranspiration affect the grade of recharge of the aquifers in summer periods, and mine waters can play an important role. Mine water can be a source of water for activities such as agriculture. The correct management of the underground storages suppose not only a new added value to the agriculture of the areas around the mine site, but an influence that can be extended to more distant areas through the use of the extracted flows to make contributions to the surface hydric systems, increasing its flow and allowing the use of this water in more distant areas without new infrastructures.

Recent studies on the reutilization of mine waters have developed new fields of application focused in three main sectors: low enthalpy geothermal energy, power generation and supply for industrial areas and agriculture.

MINE WATER REUSE IN INDUSTRIAL AND ENVIRONMENTAL FIELDS

The reuse on site for reduction of the fresh water demand in the mineral processing plants is a potential main use for mine waters. COMSOL Energy Inc. will have installed this year a new water treatment system in one of the largest coal mines of EEUU (Buchanan coal mine). This mine located in Oakwood, Virginia, will use GE technology that allows reusing 99 percent of the water at the plant for the preparation facility (GE Energy, 2010), This will allow the company to solve economical and environmental operative problems by the reduction of water demand and the volume of wastes.

On the other side, the use of groundwater as a source of heat has been known from ancient times (Cataldi, 1993), however, it is in the XX century when geothermal energy began to be used effectively. Today, the use of this technology is widely use in the world, as evidenced by the existence of small geothermal exploitation for individual uses in different projects in Germany, United States, Scotland or Spain. The most important in the world is the Mine Water Project in Heerlen (Netherlands). It constitutes the first power station in the world using exclusively mine water in the process; the project is currently extending to Germany and Belgium (COST, 2009). A Scheme of geothermal use of mine water is shown in figure 1, in which mine water is used as a heating system in the area of the closing mine.

The second important field is the electrical energy generation, which is a big consumer of water resources; as an example, Exelon Generation Company LLC in South Africa approximately consumes 1,5% of the water resources of the country, some 280 million cubic meters per year for the production of 95% of the whole electrical energy of the country. In a general way, the power stations are located close to coal basins, which have been the suppliers of fossil fuels for electricity generation; this localization gives rise to the mine water reuse, either as direct reuse and subsequent environmental benefit with effluent discharge, as direct reuse and water recirculation in the refrigeration system, or as direct reuse and water reinjection in mine works. Thus, the NMRC (National Mineland Reclamation Center) at West Virginia University (USA) is studying the possibility of using the nearly 250 billions of gallons (estimated value) stored in more than 10.000 underground abandoned mines estimates by PADEP (Pennsylvania Department of Environmental Protection) in Appalachian coal basin in Pennsylvania and West Virginia for supplying thermal power plants in the area, (Veil et al, 2003). It can be mentioned too the Wadesville and Still Creek Demonstration Project, where Exelon Generation Company LLC is studying from 2003 the substitution of a part of the flow used for refrigeration of the Exelon’s Limerick Generating Station (LGS) by water from Wadesville Mine Pool (Exelon GC, 2009).

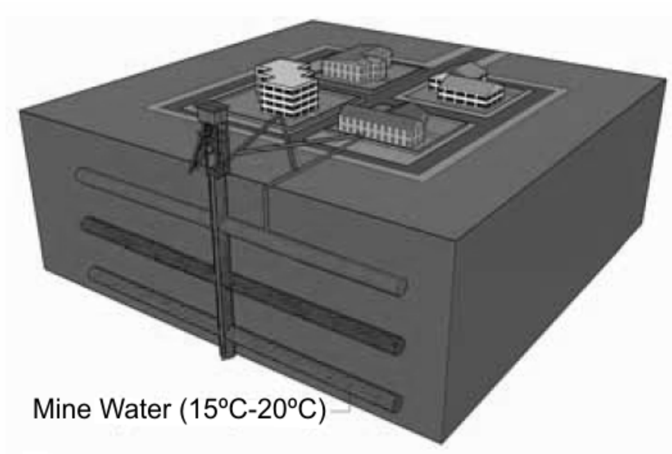

Figure 1.- Scheme of low enthalpy geothermal utilization of mine water (Modify from SEPI, 2008)

Another important aspect in historical mine sites where the reutilization of the abandoned mine sites has turned into a challenge for the society, is the application for water supply for its use in industrial areas. The consumption of water in these areas can exceed the consumption of water by population, producing an unbalancing of the available water resources and showing a big problem for municipalities. In these cases mine water reuse for refrigeration processes and other industrial applications can suppose an interesting option in order to reduce the consumption of primary water resources. At a small scale it has been studied the potential of AMD (Acid Mine Drainages) for electrical energy generation using the technology of combustible piles (Cheng et al, 2007).

The constant temperature of mine water along the year generates the optimal conditions for the growth of fish species in farm, as an example in West Virginia and Mid Appalachian states (USA), the Maryland Department of Natural Resources and the MMC (Mettiki Coal Corporation) study the technical and economical use of the average hundreds of thousands meters (estimated) of groundwater collected in abandoned underground coal mines, for treated mine water to raise brown trout, rainbow trout and cutthroat trout (Viadero et al, 2004).

Finally the reinjection of mine water in aquifers is an extended technique in the field of water regeneration. The realization of artificial recharge of aquifers has been a technique applied since decades to decrease the affection of the mining operations on them. Many populations are closed to the coast and water supplying for them is in many cases combined with surface Waters and groundwater, this methodology together with the increase of population has lead to the extraction from the upper aquifers flow water higher than the natural recharge living to marine intrusion. The use of mine water from close mine can suppose a solution to the problem. The reinjection close to the coast will allow the creation of fresh water barriers avoiding the marine intrusion and allowing the continuity of the use of groundwater by population. A similar system with regenerated water from the water-treatment plants can be found in Orange Country (USA) where the recycled water is injected by deep drills in the coast to avoid the salinization of aquifers.

POTENTIAL OF MINE VOIDS AS UNDERGROUND WATER RESERVOIRS

The coal mining in Asturias (Northern Spain) has been a prosperous industry from the eighteenth century and the traditional motor of regional economy. From the different Asturian coal basins the central coal basin is the most important despite it is suffering a progressive closure of mines; currently, only seven mines are in production in this basin. In other small basin in the municipality of Gijón La Camocha, that has been abandoned in 2008, Figure 2 shows the central area of Asturias, which lies 52% of the population of the region, and it is located more than 90% of the industry (Loredo and Pendas, 2010)..

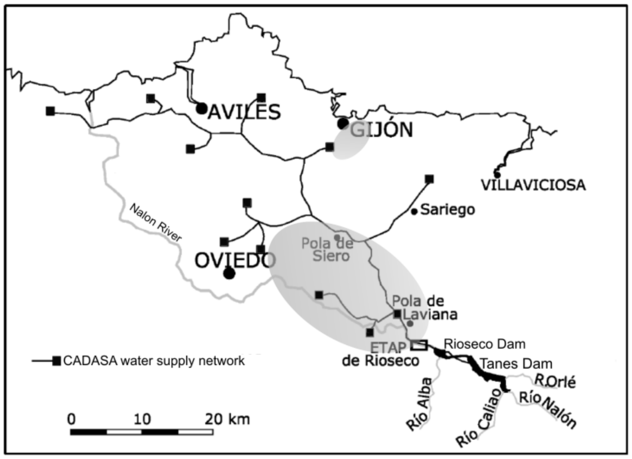

Figure 2.- Asturias central area, with CADASA water supply network and in circles the coal mining areas with closures coal mines with a high potential mine void aquifers.

Water demand for different uses for the central area of Asturias is presented in table 1, where it is possible to observe that the main user of the water resource is the industry with 51,2 Hm^3/year. The industrial areas are included in five municipalities (Gijón, Avilés, Oviedo, La Felguera and Mieres), and there are underground coal mines in three of them. Then, it could be possible to avoid to remove water from the network water supply of Water Consortium of Asturias Principality (CADASA), or from water courses or dams, increasing consequently the water resources for environmental uses. Furthermore, considering that according to the evolution of meteorological data, a decreasing of precipitation in a 17% is expected in this scenary of climatic change, the volume of water stored in underground mines of Central Coal Basin, evaluated in 40 Hm^3/year (Loredo and Pendas, 2010), are equivalent to the capacity of the two biggest dams of central area of Asturias -Tanes and Rioseco- (figure 2). Then the abandoned mines can have an interesting use as potential underground reservoirs.

Table 1.- Water provisions of the central area of Asturias for different uses.

		TOTAL (Hm^3/year)	TOTAL(l/person/day)
Resources	Surface	299,3	866,1
	Groundwater	45,0	130,2
Demand	Fixed population	26,4	76,5
	Industry	51,2	148,2
	Power Generation	25,0	72,4
	Irrigation	1,1	3,0
	Cattle	7,6	21,9
	Golf Courses	0,9	2,7
	Environmental	52,6	152,2
Surplus	**TOTAL**	**134,5**	**389,2**

Although a great part of the mines are abandoned. it is necessary to maintain water dumping in many of them because there are underground connexions with mines in operation and in other cases, to avoid flooding events in buildings. Volumes of pumped water and water quality of the coal mines are summarized in table 2. The pumping means an important economical contribution for the company and it also supposes an important loss of potential water resources in the basin that could be regulated and used for better water management in the basin.

Then, it is necessary a change in the concept of water management in Asturias. Due to the fact that Asturias, in comparison with other Spanish regions, is a region with high rainfall precipitation and abundant water resources (Loredo and Pendas, 2010), the concept of alternative sources of water resources is not enough considered into the water management organizations; then, in spite of the potential of mine voids as underground water reservoir has not currently any use.

Table 2.- Some mines in Asturias coal basin with flows and water qualities of mine water. Names in bold indicate that the mine is in operation actually.

Mine	Flows (m^3/year)	Water Quality (R.D. 140/2003)
La Camocha	693.500	A3
Santiago	1.817.689	A1
Montsacro	167.584	A1
San Jorge	396.840	
San Antonio	3.321.100	A1
San José	3.929.166	A3
Barredo	1.716.993	A1
Sta. Bárbara	1.391.727	
Polio	1.950.604	A1
Tres Amigos	834.723	A1
Samuño	2.571.875	A1
San Mames	666.211	A1
Figaredo	2.946.325	
Mosquitera	2.493.622	A1
Candín II	85.188	A1
Candín I	656.283	A1
Fondón	661.349	A1
Mª Luisa	1.579.780	A1
Cerezal	1.128.126	A1
Sotón	3.088.539	A1
Carrio	3.289.201	A1
Mariana	900.000	A3
San Nicolás	2.096.054	A1
TOTAL	**38.382.479**	

La Camocha mine has been abandoned in 2008. Neither of the studies associated with mine closure consider the possibility of maintaining the pumping in the mine, but there is no water control during the mine flooding so the economic costs could be higher (Ider, 2008; Sadim, 2008). A possibility could be the consideration of the mine water not as a waste but as a new potential resource, and this is especially important in times of climatic change and the search for a sustainable management of the available water resources (Loredo et al, 2008).
Based on the mine work plans and according to a preliminary valuation of the secondary porosity in the post exploitation works, the volume of the underground dam can be estimated in 2.8 to 3.5 Hm^3, using as calculus base a section of gallery of 10 m^2 and a correction factor caused by subsidence and a filling of 45 to 55 %. The mine water drainage together with other options of supplementary water supply could suppose for the municipality the maintenance of a good environmental quality in a river of the area and the water supply to industrial areas in the periphery of the city. The town currently consumes more than 70,000 m^3 of water per day in low touristic seasons, whereas the water consumption in summer season is in the order of 101.000 m^3 per day (Loredo et al, 2008).

The water supply to Gijón (Figure 4) comes from an area distant from the municipality and the nearby aquifers. The city receives the water from the springs of Llantones, Arrudos and Perancho, and also from the drills on the limestones and dolomites of the liasic of Villaviciosa system. These contributions suppose a 50% of the supply, which is complemented with water resources from Tanes and Rioseco dams, managed by CADASA (Consorcio de Aguas de Principado de Asturias).

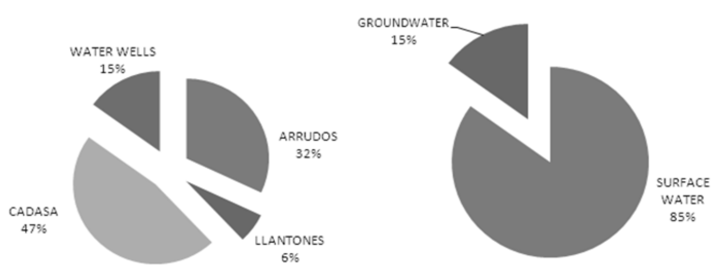

Figure 4. *Gijon water supply*

In years in which the drought affects the dams of the north Basin, the water supply to Gijón does not has problems due to the coordinated Management of surface and groundwater, but the flow level of some rivers in the area is resented; for example, the Peñafrancia River has been dried for a month in 2009, due to the pumping in the aquifers to supply water to the city, with the environmental cost that this implies. At this moment, the potential use of mine water with environmental aims is possible, and it allows maintaining the flow level of the small rivers which fed the aquifers used for the water supply to the city in drought periods (Pendas, 2005).

The mine water quality is good for its use in different purposes. According to hydrochemical data, it is bicarbonated water with low content of sulphates (Figure 5). There is not necessary any type of treatment for environmental uses or irrigation, and it is dependent of crops or receiver ecosystem. Furthermore, the analysis of other mine waters in the central coal basin indicate that there are waters included in the category of A1 treatment according to the 75/440/CEE Directive, allowing even its use for human consumption (García-Carro et al, 2008). Table 3 shows mine water quality for potential irrigation use.

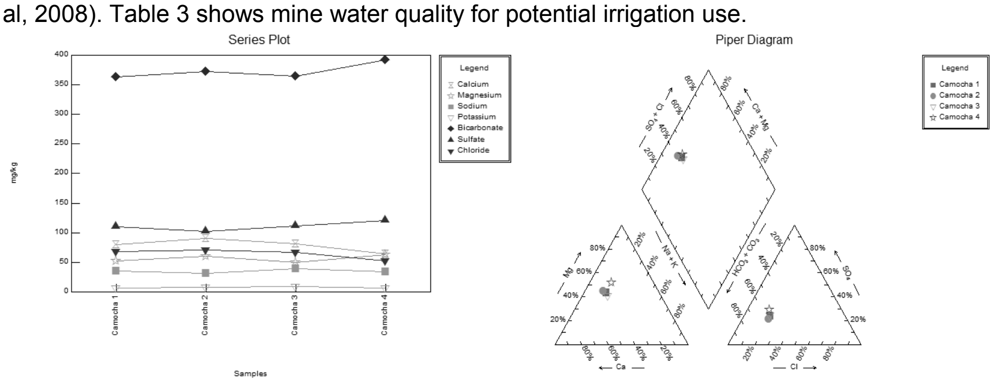

Figure 5. *Series and Piper diagrams for Camocha´s mine water*

Table 3. *Mine water characteristics.*

WATER SAMPLES	
CAMOCHA 1	**CAMOCHA 2**
Fluid Properties	Fluid Properties
Water Type Mg-HCO3	Water Type Mg-HCO3
Dissolved Solids 717 mg/kg 715.26 mg/L	Dissolved Solids 735 mg/kg 733.23 mg/L
Density 0.99757 g/cm3 Conductivity 880 µmho/cm	Density 0.99759 g/cm3 Conductivity 924.71 µmho/cm
Hardness (as CaCO3)	Hardness (as CaCO3)
Total 422.05 mg/kg 421.02 mg/L	Total 432.08 mg/kg 431.04 mg/L
Carbonate 422.05 421.02	Carbonate 432.08 431.04
Non-Carbonate 0.0 0.0	Non-Carbonate 0.0 0.0
Irrigation Waters	Irrigation Waters
Salinity Hazard High	Salinity Hazard High
Sodium Adsorption Ratio[1] 765×10-3	Sodium Adsorption Ratio[1] 619×10-3
Exchangeable Sodium Ratio 0.187	Exchangeable Sodium Ratio 0.142
Magnesium Hazard[1] 52.2	Magnesium Hazard[1] 52.1
CAMOCHA 3	**CAMOCHA 4**
Fluid Properties	Fluid Properties
Water Type Mg-HCO3	Water Type Mg-HCO3
Dissolved Solids 723 mg/kg 721.25 mg/L	Dissolved Solids 732 mg/kg 730.23 mg/L
Density 0.99758 g/cm3	Density 0.99758 g/cm3 Conductivity 842.17 µmho/cm
Conductivity 880.12 µmho/cm	
Hardness (as CaCO3)	Hardness (as CaCO3)
Total 413 mg/kg 412 mg/L Measured	Total 472.28 mg/kg 471.14 mg/L
Carbonate 413 412	Carbonate 472.28 471.14
Non-Carbonate 0.0 0.0	Non-Carbonate 0.0 0.0
Irrigation Waters	Irrigation Waters

Salinity Hazard High Sodium Adsorption Ratio[1] 858×10-3 Exchangeable Sodium Ratio 0.212 Magnesium Hazard[1] 50.1	Salinity Hazard High Sodium Adsorption Ratio[1] 722×10-3 Exchangeable Sodium Ratio 0.177 Magnesium Hazard[1] 61.9

[1] Sodium Adsorption Ratio and Magnesium Hazard were calculated with the expressions

$$SAR = \frac{[Na^+]}{\sqrt{\frac{[Ca^{2+}]+[Mg^{2+}]}{2}}} \quad MH = \frac{[Mg^{2+}]}{[Ca^{2+}]+[Mg^{2+}]}\times 100$$

Using the Rockware software AQQA

Moreover, the quality of waters can also be shown the theoretical volume stored in the underground mine Works, which are estimated between 3,8 and 2,3 Hm^3, using as calculus base a gallery section of $10m^2$, and a coefficient of the exploitation by filling and collapse of 45-55% for maximum and minimum estimations.
The use of the resource does not present big costs, as the transport distances of water from the mine are small (4 km maximum). A supply by gravity from the mine, located at 101 meters above sea level, to the areas of interest. Eight football fields, and two golf fields located in the proximities of the mine, which have a daily consumption of 6 l/m^2 (Loredo et al, 2008) could be included among the potential areas to supply with mine water, in this case, the salinity of the waters would not be an obstacle since it is an alternative source in drought seasons and not for routine use, on recreational herbaceous crops and not agricultural uses.

CONCLUSIONS

Mine activities in the Asturian coal basin create voids and new fracture systems that alter the natural hydrogeological system of the area, resulting in new reservoirs like karstic carbonate aquifers to triple porosity.
Thus, the management of mine water stored in abandoned deep mine works allow to recuperate an important part of water resources increasing the availability of them; the new aquifer formed after pumping water suspension during the closure phase of the mine behaves as a pseudo-karstic aquifer where the mine voids originated during the exploitation phase of the mine act as underground dams. Then, mine water stored in underground mine works constitute real underground dams similar to the surface dams; they do not suffer evaporation losses and they are available for its use in drought periods as a complement to conventional water resources.

The reuse of mine waters can serve as a starting point for restoration of mining/industrial depressed areas by its use for the industry and the agriculture as well as by the creation of a new environmental asset by the conversion of mining passives into a new source of resources in the field of tourism and environmental protection, minimizing the hydric footprint and offering new alternatives of the use to the mine water resources.

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Field and laboratory experiments to investigate infiltration processes and clogging effects from a ponding recharge system at Ban Nong Na, Bangrakum District, Phitsanulok Province, Lower Yom River Basin, Thailand

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Abstract

A small-scale field experiment consisting of surface infiltration tests was conducted at Ban Nong Na, located in Bangrakum District, Phitsanulok Province, situated in the Lower Yom River Basin. The shallow groundwater in this area has been heavily pumped for growing rice all year round, and within the past decade static water levels within the gravel, sand and silt aquifers have decreased to a depth of up to ten meters below the ground surface. Research is currently being conducted to investigate the feasibility of managed aquifer recharge by surface ponding methods in the Lower Yom River Basin. This study, which forms one component of the broader project, aims to assess infiltration processes and clogging effects at the laboratory (column) scale and at the small scale (25 m^2) in the field. The laboratory experiment consisted of two main components: 1) physical, chemical and biological analyses of raw water and the ambient groundwater and 2) soil column testing under constant head conditions over a period of 100 hours. The field experiment consisted of three main components: 1) characterization of the physical and hydraulic properties of the unsaturated and saturated media 2) pretreatment design considering levels of turbidity removal using synthetic poly and sand filter, and 3) infiltration testing under constant head conditions over a period of 30 hours. These works are intended to provide the design criteria for establishing a larger scale (1,600 m^2) pilot recharge system at the study site. Average infiltration rates for the laboratory experiment for source waters with mean turbidities of 0.5 and 100 NTU were found to be 3.27 and 0.15 m/d respectively. The infiltration rate from the field experiment with an average turbidity 50 NTU was 2.53 m/d; a magnitude commensurate with the lab study. Since infiltration rates in excess of 1 m/day are desirable for the pilot trial, the turbidity of the raw canal water used for recharge will be controlled to be less than 50 NTU. Whilst both the laboratory and field experiments were brief and longer test periods needed, more extensive investigations will be performed over the 2010 monsoon season during the full-scale pilot trial.

Keywords: infiltration, clogging, experiment, ponding recharge, Lower Yom River Basin

INTRODUCTION

A small-scale field experiment consisting of infiltration tests via ponding systems was conducted at Ban Nong Na, located in Bangrakum District, Phitsanulok Province. The study area is situated in the Lower Yom River Basin where the shallow groundwater has been heavily pumped for growing rice all year round. Within the past ten years or so, static water levels of the gravel, sand and silt of the alluvial fan deposits have decreased from about one to ten meters below the ground surface, placing constraints on the lift irrigation systems employed by farmers. Thus, the objective of the study is to determine infiltration processes and clogging effects of ponding recharge system or managed aquifer recharge in the laboratory and in the field at the small-scale (25 square meters). This study is part of a broader project investigating the feasibility of managed aquifer recharge by ponding-based methods in the lower Yom River Basin, Thailand.

Clogging is one of the most important factors effecting infiltration performance (Pavelic, et al., 2007; Wu et al., 2007) and the technical and economic viability of MAR. Clogging may be defined as the reduction of the available pore volume of a porous media due to a combination of physical, biological and chemical processes. Consequently, the immediate effect of clogging is to reduce the intrinsic permeability of a system, leading to a drop in infiltration rates in the case of surface ponding (Perez-Paricio and Carrera 1999). The maximum infiltration rate is generally observed at the beginning of an infiltration test and decreases over time. The poorer the source water quality, the faster and more extensive the reduction in infiltration rate, as illustrated in Figure 1. The challenge here is define the appropriate quality of source water to achieve viable rates of infiltration over the long term.

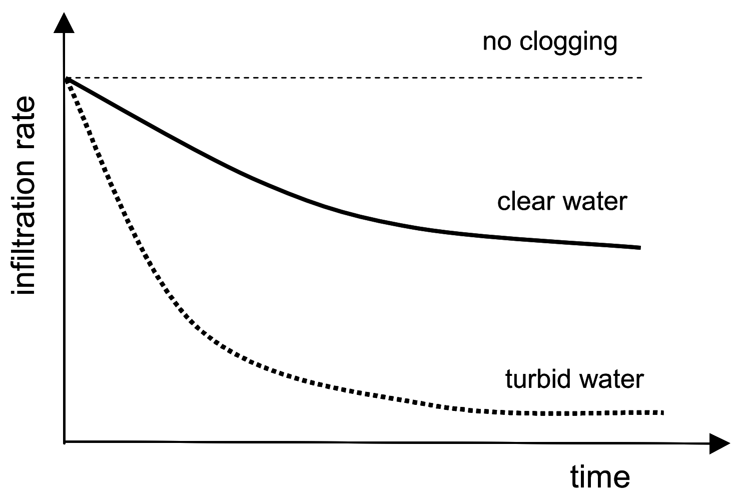


Figure 1. Conceptual illustration of infiltration and clogging with clear and turbid source water quality

MATERIALS AND METHODS

Study area

The experimental site is located at Ban Nong Na, Bangrakum District, Phitsanulok Province, Lower Yom River Basin, Thailand (Figure 2). Climate data of the study area collected from Phitsanulok Province station during year 2000 – 2010 indicate a mean temperature of 28 ^{o}C, relative humidity of 70 %, wind speed of 1.37 knot and annual pan evaporation of 1,504 mm. Annual rainfall analyzed from four rainfall stations at Bangrakham, Sai Ngam, Lan Krabue and Sam Ngam Districts determined from the Thiessen method is 1,286 mm.

The lithology of the Ban Nong Na site consists of clay (0.0-1.0 meters Below Ground Surface (m BGS)), sandy clay (1.0-3.5 m BGS), upper sand (3.5-7.5 m BGS), lower sand (7.5-12.0 m BGS), gravel (12.0-14.0 m BGS) and clay (14.0-15.0 m BGS) as shown in Figure 3. Physical properties of two main aquifers (upper and lower sands) are distinguishable from their different grain size distributions. The effective grain size (d_{50}) and coefficient of uniformity (C_u) of upper sand and lower sands are 0.34 and 0.49 mm and 2.14 and 2.19 for the upper sand and lower sands respectively. Hydraulic properties of the shallow aquifers determined by pumping tests indicate values of transitivity between 2,050 to 3,600 m^2/day and specific storage between $8.4x10^{-4}$ to $1.3x10^{-2}$.

An experimental pond with a width, length and depth of 5.0, 5.0 and 3.5 meters respectively and side-wall slope of 2.8:1 (v:h) was constructed at the site. The base of the pond was filled with a 0.5 m thick layer of fine sand and overlaid with poly material to aid removal of turbidity. The d_{50} value of the sand filter is 0.33 mm and the synthetic poly has an effective opening size 0.10 mm.

Water for recharge was sourced from the canal adjacent to the experimental pond. Inflows of water into the pond were determined from a v-notch weir (water inflow), and infiltration rates read from a staff gauge fitted in the pond.

Four piezometers, completed with 1 inch diameter PVC pipe, were constructed within the pond at various depths within the unsaturated zone (PP1 - PP4). Five piezometers (2 inch diameter PVC pipe) were constructed close to the pond to depths of 12-13 meters (GW59 - GW63) to monitor the saturated zone Three piezometers at greater distances from the pond (10-20 meters away) to depths of 12-14 meters were additionally constructed (GW34, GW51 and GW66P). Further afield, 50-100 meters from the pond, four piezometers to depths of 12-19 meters were constructed (GW44, GW50, GW55, GW58 and GW65P). Piezometer locations are indicated in Figure 2 and 3.

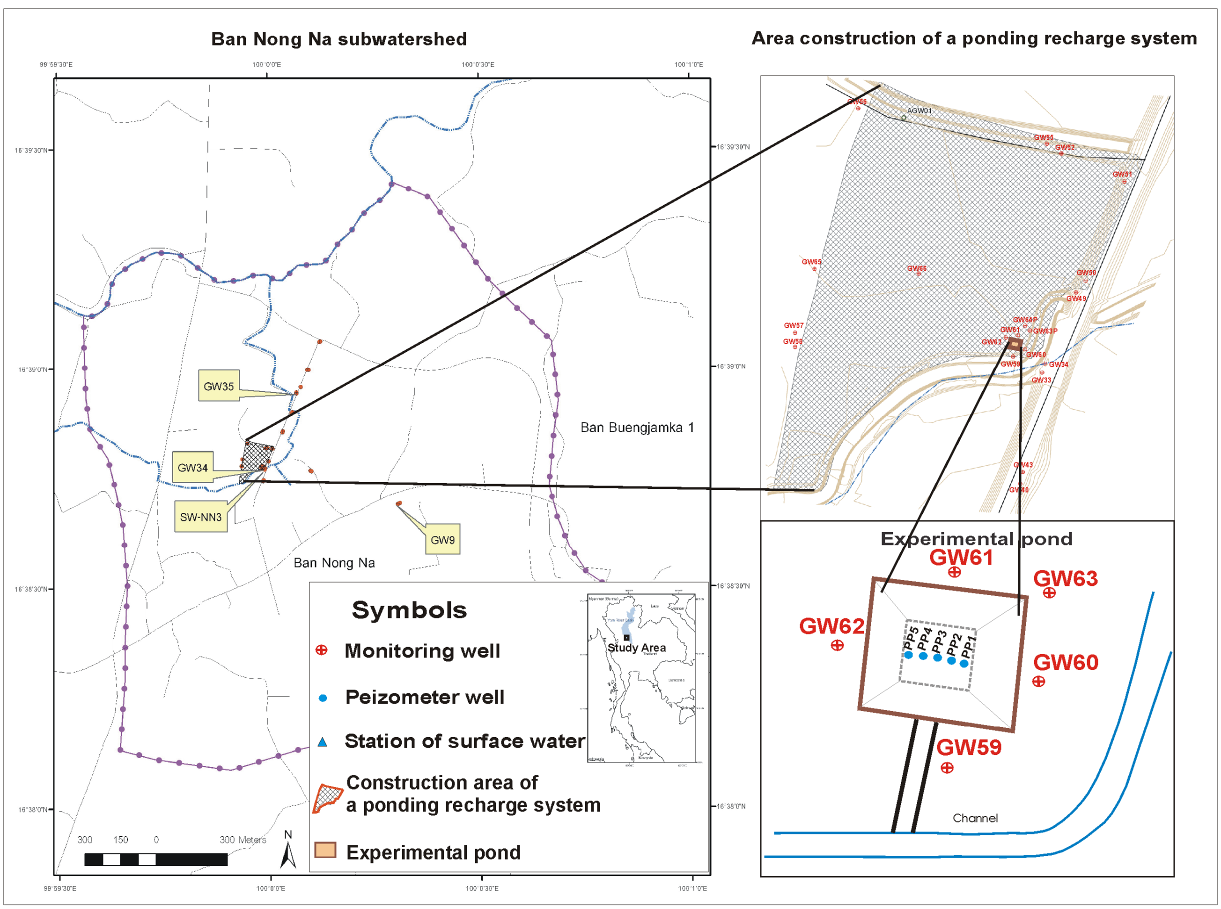


Figure 2. Location of the study area and water sampling stations

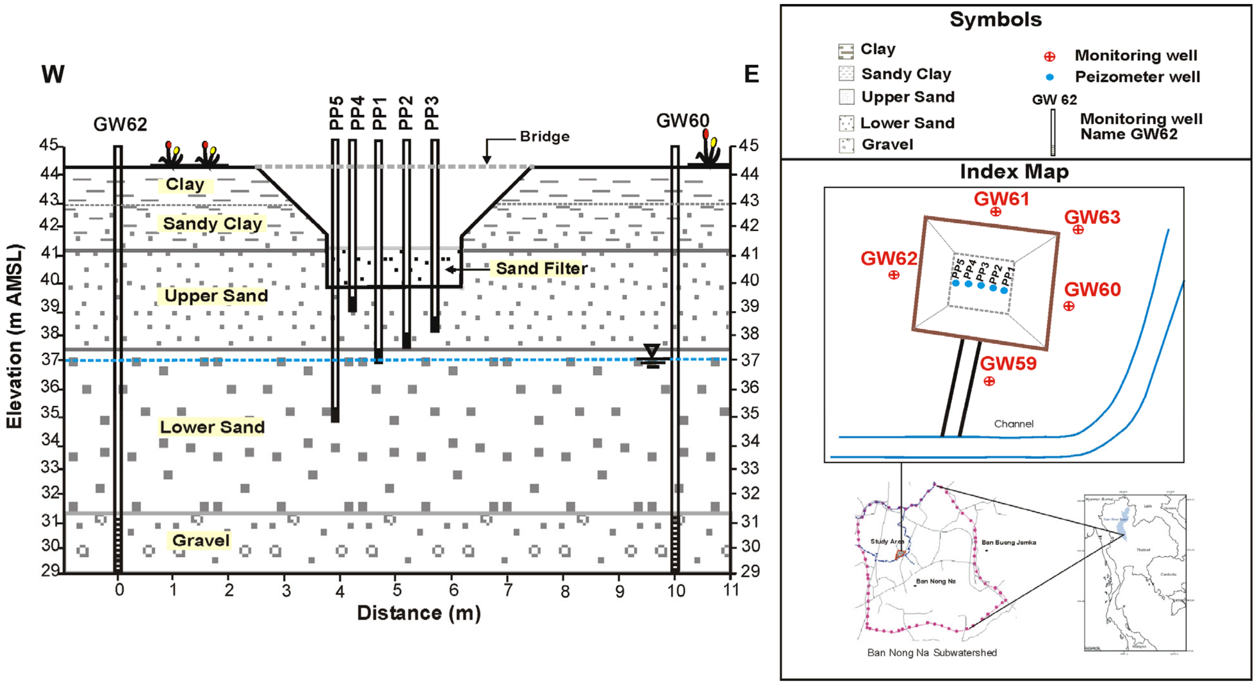


Figure 3. Hydrogeological cross section along the transect W-E

Laboratory experiment

The laboratory experiment consisted of two main components: 1) physical, chemical and biological analyses of raw water and the ambient groundwater and 2) soil column testing under 0.1 meter constant head conditions for a duration of 100 hours.

The water samples from surface water and groundwater were analyzed for physical, chemical and biological constituents (parameters listed in Table 1). The surface water was sampled at the site (SW-NN3). Groundwater quality was determined at three locations: the natural recharge area (GW8), at the study site (GW33) and at the discharge area (GW35), as shown in Figure 2.

The laboratory experiments consist of 2 distinct water types - clean water (0.5 NTU) and more turbid water (100 NTU). A total of 6 soil columns, 10 cm in diameter and 65 cm in length, made from clear plastic pipe were each packed with one of three types of media: upper sand, lower sand and sand filter. Each column contains 10 cm of the media from the bottom.

Each of the tests were performed in two distinct phases. In the first phase, the experimental duration was 30 hours, whilst for the second phase infiltration extended for 100 hours. The media was undisturbed between the first and second phases, and all other experimental conditions remained unchanged.

The mean infiltration rate (q_{avg}) for each column test over the experimental period was calculated from Harmonic mean (Freeze and Cherry, 1979) as given in equation 1:

$$q_{avg} = \frac{d}{\sum_{i=1}^{n} \frac{d_i}{q_i}} \qquad 1$$

where q_i is basic infiltration rate, d_i is thickness of aquifer and d is sum of aquifer thickness.

Rates of clogging for each laboratory and field test were quantified with the aid of a simple exponential decay model. The infiltration rate changes over time (q(t)) may be described by the following equation 2:

$$q(t) = q_0 e^{-kt} \tag{2}$$

where q_0 is the observed initial infiltration rate, k is the fitted clogging coefficient and t is the elapsed time.

Field experiment

The field experiment consists of three main components: 1) measurement of physical and hydraulic properties of the porous media in both unsaturated and saturated zones, 2) pretreatment design considering levels of turbidity removal using synthetic poly and sand filter and 3) infiltration testing under 2 meter constant head conditions for 30 hours with a recharge water turbidity that ranged from 43 to 59 NTU (average of 51 NTU)**.**

RESULTS AND DISCUSSIONS

Water quality

Surface water and groundwater samples were collected on December 24, 2009. Physical, chemical and biological characteristics of the water samples are given in Table 1. Since the sampling was performed during the dry season, the values for surface water quality are unlikely to be representative of wet season conditions. Values for turbidity, for example, are probably four-fold higher than anticipated during the wet season. Groundwater quality conditions, on the other hand, are expected to be more stable. The TDS data suggests that the surface water in the canal originates from enriched groundwater as a result of return flows from the adjacent paddy fields. The data suggests that clogging is likely to be dominated by particulate depositions, but other forms of clogging cannot be discounted.

Table 1. Surface water and groundwater quality at the study site

Parameters	Unit	Surface water SW-NN3	Shallow groundwater		
			GW9	GW34	GW36
Physical quality					
Turbidity	NTU	882	174	43	25
Total Suspended Solids	mg/L	1,707	251	61	25
Chemical quality					
pH	-	6.68	5.63	6.09	6.48
Calcium	mg/L	19.15	11.67	23.48	20.42
Magnesium	mg/L	8.42	3.84	4.16	3.98
Total Dissolved Solid	mg/L	680	132	255	171
Biological quality					
Total Organic Carbon	mg/L	11.09	< 2	< 2	< 2

Laboratory column experiment

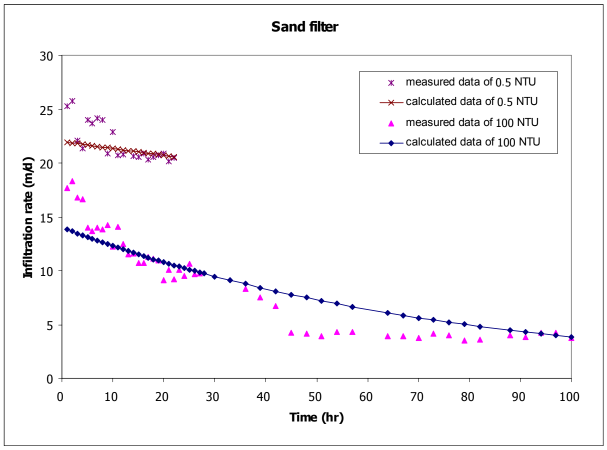
The laboratory column experiments were carried out during March 2-7, 2010. Basic infiltration rate from the laboratory experiment (Figure 4), were analyzed by harmonic mean for source water turbidities of 0.5 NTU and 100 NTU were 3.27 and 0.15 m/d as shown in Table 2. Infiltration rates for the longer second phase tests are consistently lower than for the first phase tests. Infiltration rate data were fitted with exponential decay model given in equation 2. Initial infiltration rates (q_0) for 0.5 NTU and 100 NTU are 0.5 – 22 m/d and 0.35 – 14 m/d respectively. The coefficient k for 0.5 and 100 NTU are 0.003 – 0.01 and 0.013 – 0.022 respectively as shown in Table 3.

Table 2 Basic infiltration rate of laboratory experiment (unit: m/d)

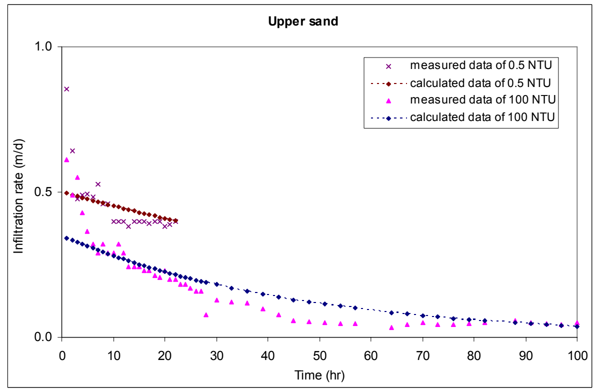
Water quality / Media	Sand filter	Upper sand	Lower sand	Mean
1. Clear water (0.5 NTU)	18.0	1.3	10.8	3.27
2. Turbid water (100 NTU)	4.8	0.05	2.4	0.15

Table 3 Fitted exponential decay rates (k) for the laboratory experiments

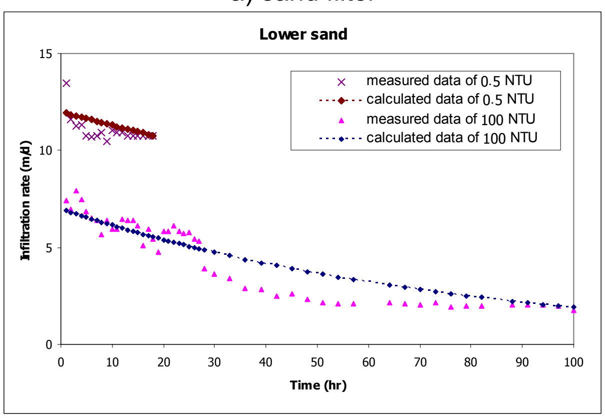
Water quality / Media	Sand filter		Upper sand		Lower sand	
	q_0	k	q_0	K	q_0	k
1. Clear water (0.5 NTU)	22	0.003	0.50	0.010	12	0.006
2. Turbid water (100 NTU)	14	0.013	0.35	0.022	7	0.013



a) sand filter



b) upper sand



c) lower sand

Figure 4. Infiltration rates from laboratory experiment

Field experiment

The field experiment was carried out during February 22-23, 2010. The initial infiltration rate (q_0) was 5.2 m/d and harmonic mean of 2.53 m/d as shown in Figure 5. The total volume of water infiltrated during the experiment was 52 m^3. The fitted value of k was 0.017.

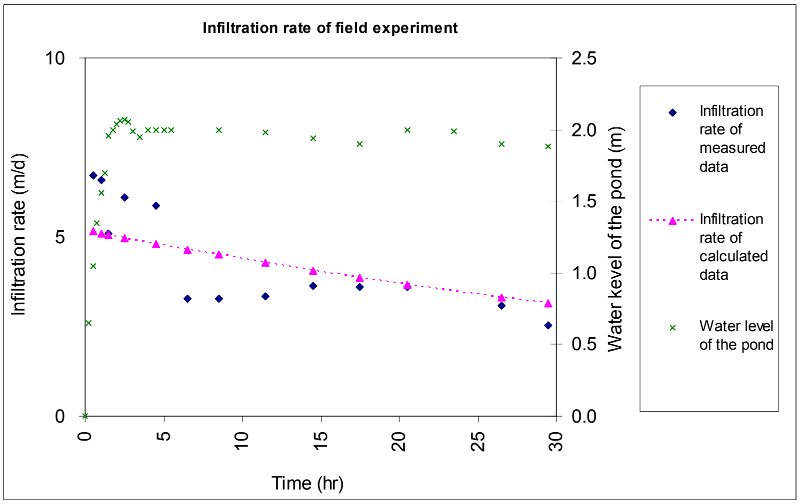


Figure 5. Infiltration rate of field experiment

CONCLUSIONS AND RECOMMENDATIONS

Infiltration rates of the laboratory of 0.5 and 100 NTU ranges from 3.27 to 0.15 and Infiltration rate of the field experiment of 50 NTU is 2.53 m/d. Clogging effects of field and laboratory experiments had yet to stabilize fully as the experimental time was limited. The ranges of turbidity are relatively large therefore; the raw water turbidity for the upscale pilot trial experiment will be maintained at less than 50 NTU in order to achieve target infiltration rates of about 1 m/d.

Both the laboratory and field experiments should be tested over longer periods, until quasi steady state conditions are observed, which did not occur in this study. The full scale pilot test that will be conducted during July to October 2010 should enable this to occur.

ACKNOWLEDGMENTS

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Integrative technologies for safely managed groundwater recharge using reclaimed water in Zhengzhou, China

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Abstract

It is significant to safely manage groundwater recharge using reclaimed water for recovering aquifer and relieving water shortage and environmental pollution. We present here an ongoing project on integrative technologies for safely managed groundwater recharge using reclaimed water in Zhengzhou, North China, which includes wastewater treatment, artificial wetland, infiltration basin, monitoring system and safety assessment of water quality. The wastewater treatment system is an integration of coagulation, filtration, adsorption and disinfection, including a recently developed flocculant of modified Bentonite by microwave plus ferrous sulfate. Municipal wastewater is reclaimed by artificial wetland and the wastewater treatment system and conducts into infiltration basins for recharge. The water quality of outflow from the wastewater treatment system generally satisfies the limit of Water Quality Standard for groundwater recharge. The recharged groundwater using wastewater reclaimed by the integrative technologies in the Zhengzhou site can generally satisfy criterion of grade III of the quality standard for groundwater GB/T14848-1993 and can be used for fishery, industry and agriculture. The recharge in the early period leached a lot of ammoniac nitrogen and calcium sulphate from vadose zone and made high TDS and ammoniac-N in waters below the leaching zones which decrease with continuation of leaching.

Keywords: reclaimed water, soil aquifer treatment, groundwater recharge, water safety, Zhengzhou

INTRODUCTION

The growing conflicts between supply and demand of water quantity and quality plus wastewater discharge are serious problems to restrict the social development in North China. Thus it is significant to safely manage groundwater recharge using reclaimed water for recovering aquifer and relieving water shortage and environmental pollution. The authors present here an ongoing project on integrative technologies for safely managed groundwater recharge using reclaimed water in Zhengzhou, North China. The integrative technologies for safely managed groundwater recharge mainly include~~s~~ wastewater treatment, artificial wetland and infiltration basin (Bouwer, 2002) for recharge, monitoring system covering different depth of vadose zone and aquifer, and safety assessment of water quality.

Background of the recharge site

Zhengzhou is located in the lower reaches of the Yellow River, where groundwater is the most important water resource and supplies for 70% of water use. Extensive overexploitation of groundwater has resulted in significant decline of groundwater level in the past decades (Sun et al., 2009). Surface water has been serious polluted which threatens groundwater quality. The recharge site is located in the northeast part of Zhengzhou, 1.5km west to National Road 107, the north side of Hongbao road and about 350 m away from Jialu River in the northeast (Fig.1). Main water in the Jialu River is from municipal wastewater treatment plants, generally being or worse than grade V of environmental quality standard for surface water (GB 3838-2002) (GAEP, 2002) (Table 1).

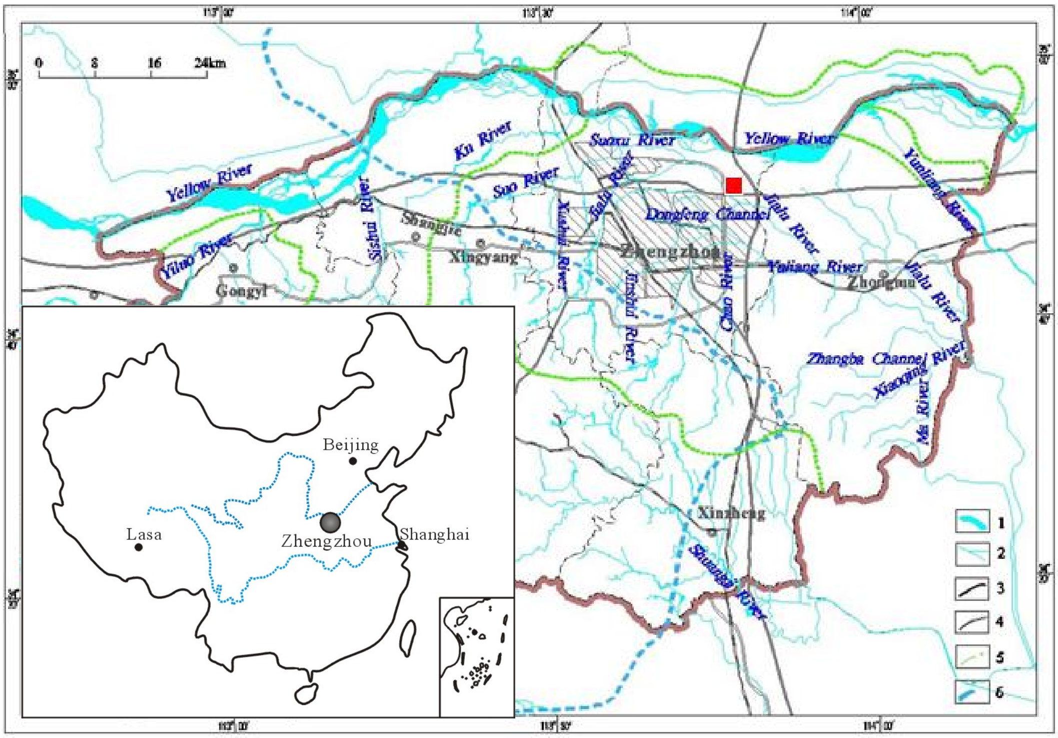

Fig.1 Location of recharge site and surface water in Zhengzhou

1. Reservoir; 2. River; 3. Highway; 4. Railroad; 5. Borderline of catchment; 6. South-to -north water transfer project; Red square is the recharge site.

A borehole showed that materials from land surface to depth of 9.1m are interbed of sandy loam, silt loam, silt and sandy clay, in which top 3 m is artificial filling; and fine and medium sand from 9.1 m to 30.3 m which is the main aquifer of shallow groundwater. Hydraulic conductivities of top soils are in the order of 0.096 to 1.344 m/d measured by double ring infiltrating. Hydraulic conductivities for the silt loam and sandy clay are 5.88×10^{-5} -1.08×10^{-4} m/d measured by the falling-head permeability tests using undisturbed soil cores. Water table depth in the recharge site now is 6.3-7.4 m but was near 2 m in the previous discharge area of the regional groundwater flow.

Water quality of the shallow groundwater (Table 1) basically belongs to grade III of Quality standard for groundwater GB 14848-1993 (AQS. 1993), but NH_4^+-N, permanganate index, and Fe over the limit of grade III and fully satisfy the limit of grade IV, *i.e.* it does not satisfy the requirement of standards for drinking water GB 5749-2006 (MOH, 2006). Total dissolved solids (TDS) of the shallow groundwater are about 1 g/L. Some organic contaminants, like phthalate esters, polycyclic aromatic hydrocarbons and organochlorine pesticides were detected in the shallow groundwater but not over the limit of grade III of Quality standard for groundwater GB 14848-93 (AQS. 1993). Most of the shallow groundwater near the recharge sites is pumped for replenishment of the fishpond.

Table 1 Background of water quality in different waters (mg/L except pH)

Date	Types	pH	permanganate index	TN	TP	F^-	Cl^-	NO_2^-	NO_3^-	SO_4^{2-}
2009-3-1	Jialu River	7.82	29.0	19.0	2.0	0.4	214.1	-	1.3	169.4
2009-3-1	Fish Pond	8.08	22.9	2.8	0.4	0.4	179.2	-	0.4	110.9
2009-3-1	Shallow Groundwater	7.95	3.7	1.8	0.2	0.4	174.3	-	1.7	89.6
2009-5-1	Shallow GW(20m)	-	-	-	-	0.4	146.4	0.07	0.2	71.6
Date	Types		NH_4^+-N	As	Ca	Fe	K	Mg	Mn	Na
2009-3-1	Jialu River		15.9	0.018	67.5	2.2	16.0	33.9	0.1	110.5
2009-3-1	Fish Pond		1.1	0.036	66.5	1.6	6.7	46.4	0.1	75.4
2009-3-1	Shallow Groundwater		0.9	0.048	86.7	3.7	5.2	44.8	0.5	74.2
2009-5-1	Shallow GW(20m)		-	<0.001	102.6	1.4	1.9	64.1	1.4	98.9

METHODS AND MATERIALS

Wastewater from the Jialu River is pumped to the artificial wetland at a rate of 1000 m^3/d for decontamination. The outflow of the artificial wetland is reclaimed by the wastewater treatment system and then conducted into an infiltration basin for further purification using soil aquifer treatment (SAT) and groundwater recharge. To against the biological clogging in SAT, four infiltration basins are used for alternative leaching and drying.

Wastewater treatment

A low cost high efficient flocculant for wastewater treatment was developed using modified Bentonite by microwave plus ferrous sulfate ($FeSO_4$). An optimum water treatment system integrating of coagulation using the modified Bentonite, filtration using sand and fiber ball, adsorption using MWB activated carbon, and ozonic disinfection was developed.

Artificial wetland

The artificial wetland is on the northeast side of the Jialu River and its area for the first phase is about 1.5 ha. Plants planted in the wetland include hydrophyte ~~include~~ *Canna*, water spinach or water convolvulus, *cattail* (*Typha*), etc.

Monitoring systems

Water quality of inflow and outflow of the water treatment system are analyzed daily, including pH, electric conductivity (EC), turbidity, total phosphorus (TP), total nitrogen (TN), nitrate nitrogen, nitrite nigtrogen, ammonia nitrogen, and permanganate index, etc. Other composition in the reclaimed water and groundwater, such as macro-composition, heavy metals and organic pollutants are analyzed monthly or seasonally.

Observation and sampling in the recharge basin include water content for the top 2.5 m vadose zone measured by Time Domain Reflectometry (TDR) at viable depths in the early infiltration and dry periods; soil water collected at depths of 1m, 2 m, 3 m, 4 m and 5 m; and water level measured and water sampled at depths of 7.5 m, 10 m, 15 m and 20 m in the saturated zone (Fig. 2).

Assessment of water quality safety

Water quality at different steps is assessed according to the following standards of China: The reuse of urban recycling water: Water quality standard for groundwater recharge GB/T 19772-2005 (AQSIQ, 2005); quality standard for groundwater GB/T14848-1993 (AQS, 1993); and standards for drinking water quality GB 5749-2006 (MOH, 2006). Before conducting into infiltration basin, reclaimed water, the outflow of the integrated treatment system must satisfy the requirement of the water quality standard for groundwater recharge GB/T 19772-2005. Water quality in the recharged aquifer should satisfy the limit of grade III of quality standard for groundwater GB/T14848-1993 or be better than that quality of original groundwater.

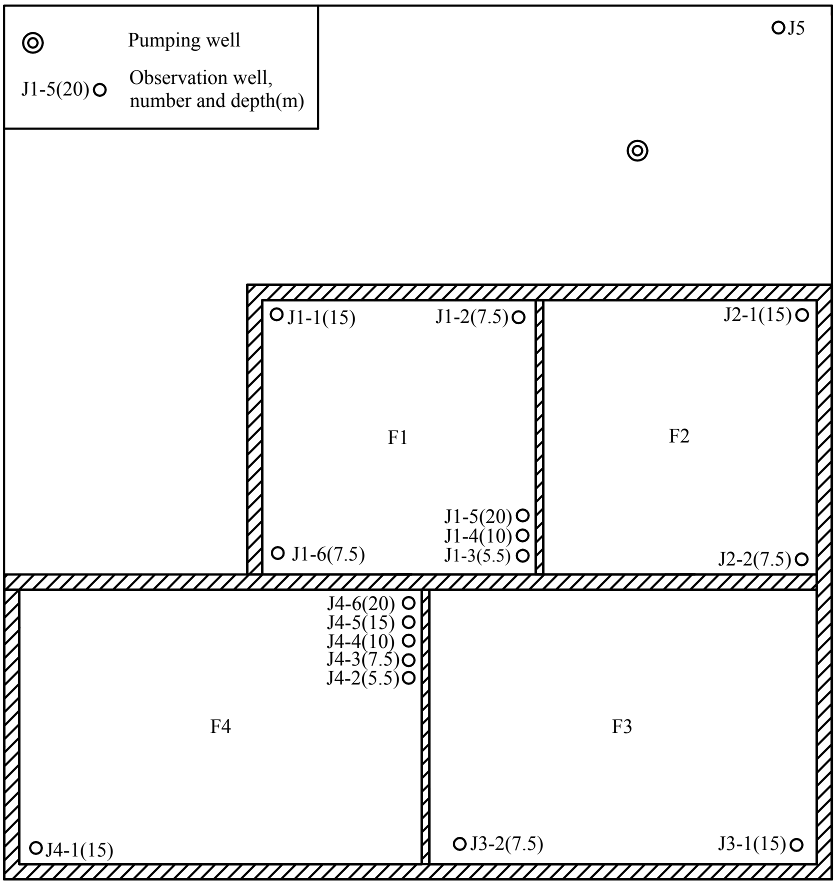

Fig. 2 Monitoring and sampling wells at different depths in the Zhengzhou recharge site

RESULTS AND DISCUSSION

Based on laboratory experiment of modified bentonite for processing wastewater by roasting activation, acid activation, modification of Al-pillared and modification of micro-Ferrous sulfate, we found that the micro-Ferrous sulfate modified bentonite has strong capability to remove total phosphorus, because micro modification of Ferrous sulfate plus bentonite can effectively expand the interlayer spacing of bentonite and the iron ions may balance the negative charges on silico-oxygen tetrahedron, which make cations from interlayer more exchangeable.

The optimum integrated wastewater treatment system can effectively remove suspended solids, organic components and total phosphorus of wastewater, and the removal rates are all higher than 80% (Table 2). The water quality of outflow generally satisfies the limit of the water quality standard for groundwater recharge GB/T 19772-2005 (AQSIQ, 2005) (Table 3 and 4). The operating cost is 0.7 RMB per cubic meter of wastewater.

Table 2 Removal rates for selected components (Pan, 2010)

	Turbidity (NTU)	COD_{Mn} (mg/L)	TP (mg/L)	TN (mg/L)	Turbidity (NTU)	COD_{Mn} (mg/L)	TP (mg/L)	TN (mg/L)
Before treatment	66.4	31.8	2.86	27.87	102.4	32.2	2.98	30.10
After treatment	4.7	6.0	0.34	21.36	8.1	6.3	0.36	27.76
Removal rate(%)	93	81	88	23	92	81	88	8
Before treatment	92.3	36.8	3.35	28.30	112.5	53.5	3.55	29.12
After treatment	3.3	5.3	0.31	24.46	2.4	5.5	0.36	24.82
Removal rate(%)	96	85	91	14	98	90	90	15

Note: COD_{Mn} denotes permanganate index.

Table 3 Macro ions of different waters in the Zhengzhou recharge site (mg/L)

Date	Number	Depth (m)	K^+	Na^+	Ca^{2+}	Mg^{2+}	SO_4^{2-}	Cl^-	HCO_3^-	TDS
2009-10-19	Jialu river		13.3	141.6	59.6	27.5	136.9	171.6	369.1	737.5
2010-4-21	Jialu river		13.8	89.9	69.1	22.7	126.3	112.0	298.2	586.7
2009-10-1	Before treatment		13.9	116.3	74.7	31.5	119.2	110.5	332.8	643.9
2009-10-1	After treatment		16.3	113.1	80.1	30.5	114.0	132.2	299.9	648.1
2009-10-29	T4-1	1	11.1	165.0	159.7	32.9	141.7	195.4	270.2	848.9
2009-10-29	T4-2	2	14.8	117.2	219.9	59.0	205.3	169.9	487.7	1037.7
2009-10-29	T4-3	3	21.7	169.9	857.5	223.2	1807.3	115.7	593.1	3508.5
2009-10-29	T4-4	4	42.4	179.2	784.0	247.8	1888.6	113.5	362.5	3453.2
2009-10-29	T4-5	5	37.1	180.4	543.5	246.5	2614.2	161.2	692.0	4139.3
2009-10-19	J4-3	7.5	15.6	129.2	292.2	139.9	1174.9	153.5	823.8	2323.4
2009-10-19	J4-4	10	11.7	131.3	244.6	129.4	922.3	153.5	757.9	1978.3
2009-10-19	J4-5	15	10.6	115.6	171.6	81.4	405.8	151.3	724.9	1306.9
2009-11-14	J4-6	20	4.9	107.1	119.4	61.6	110.9	141.2	757.9	930.1
2010-4-21	J4-3	7.5	7.9	115.8	227.8	128.5	965.0	151.1	491.2	1860.1
2010-4-21	J4-4	10	6.3	104.8	210.6	132.7	742.0	142.1	614.0	1659.8
2010-4-21	J4-1	15	3.4	87.2	112.2	73.2	45.2	150.0	666.6	812.0
2010-4-21	J4-5	15	6.0	98.6	119.5	66.9	71.4	150.1	684.2	859.5
2010-4-21	J4-6	20	4.3	98.9	110.1	65.8	240.8	153.6	456.1	907.1
2010-4-21	Pumping well	20	4.5	103.2	134.4	84.7	252.1	158.0	631.6	1059.0

Note: Sample number with T is soil water sampled from vadose zone.

The monitored water quality– at~~in~~ different depth of vadose zone and saturated zone shows the reclaimed water infiltration in the early period leached a lot of ammoniac nitrogen and calcium sulphate from the vadose zone (Table 3 and 4). TDS in soil water of vadose zone and shallow saturated zone are in the order of 2-4 g/L and ammoniac nitrogen ranges from 1-20 mg/L in the fall of 2009 and tend to decrease with continuation of leaching, compared to the concentration in June 2010.

The executive results of the integrative technologies shows that the recharged groundwater using the reclaimed water can generally reach criterion of grade III of the quality standard for groundwater of China (GB/T14848-1993) or not worse than background quality of groundwater. The recharged groundwater in the Zhengzhou recharge site can be used for fishery, industry and agriculture.

Table 4 Comparison of selected components of different waters in the Zhengzhou recharge site (mg/L)

Date	Types (depth)	pH	TP	TN	NH_4^+-N	NO_3^--N	NO_2^--N	COD	COD_{Mn}
2009-10-1	Jialu River		1.33	17.92	9.22	2.19	0.680	49.6	
2009-10-1	Wetland outflow		0.02	9.09	0.62	2.12	0.027	30.9	
2009-10-1	Before treatment	7.52	0.29	7.98	0.22	3.18	0.380		7.4
2009-10-1	After treatment	7.61	0.05	7.87	0.10	5.64	0.140		4.0
2009-11-9	After treatment		0.34	11.39	0.717	5.24	0.260		5.7
2009-11-9	T4-1(1m)		0.01	12.82	0.467	6.18	<0.003		2.8
2009-11-9	T4-2(2m)		0.02	10.18	2.736	4.72	0.382		10.4
2009-11-9	T4-3(3m)		-	3.77	2.309	0.10	<0.003		11.5
2009-11-9	T4-4(4m)		0.02	20.36	13.098	<0.03	0.442		5.5
2009-11-9	T4-5(5m)		-	19.64	19.316	0.30	<0.003		6.5
2010-6-8	Jialu River		2.00	12.15	9.57	-	-	54.5	
2010-6-8	Wetland outflow		0.99	4.85	3.46	-	-	39.7	
2010-6-8	Before treatment	7.96	0.72	4.50	2.44	0.99	0.362		11.6
2010-6-8	After treatment	7.92	0.06	3.89	1.59	1.81	<0.003		4.2
2010-6-8	Groundwater J4-3(7.5m)	6.99	0.16	9.29	7.57	0.87	0.011		4.7
2010-6-8	Groundwater J4-4(10m)	7.04	0.02	9.01	7.54	0.95	0.024		4.6
2010-6-8	Groundwater J4-1(15m)	7.39	0.17	1.84	1.24	0.64	0.002		3.1
2010-6-8	Groundwater J4-6(20m)	7.36	0.15	2.26	1.85	0.72	<0.003		5.8
2010-6-12	Before treatment	8.09	0.84	3.23	1.52	0.92	0.180		11.0
2010-6-12	After treatment	8.08	0.04	2.96	1.05	1.91	<0.003		4.4
2010-6-12	Groundwater J4-3(7.5m)	7.10	0.07	9.50	7.71	0.80	0.019		4.0
2010-6-12	Groundwater J4-4(10m)	7.16	0.01	11.39	9.35	0.92	0.220		3.4
2010-6-12	Groundwater J4-1(15m)	7.58	0.15	1.49	1.47	0.76	0.009		2.6
2010-6-12	Groundwater J4-6(20m)	7.37	0.15	3.47	2.54	0.70	<0.003		5.9

Note: COD is chemical oxygen demand analyzed by $K_2Cr_2O_7$; COD_{Mn} denotes permanganate index.

CONCLUSIONS

The optimum integrated wastewater treatment system including modified Bentonite can effectively remove suspended solids, organic components and total phosphorus of wastewater, and the removal rates are all higher than 80%. The water quality of outflow generally satisfies the limit of the water quality standard for groundwater recharge GB/T 19772-2005.

The recharged groundwater using municipal wastewater reclaimed by the integrative technologies in the Zhengzhou site can generally reach criterion of grade III of the quality standard for groundwater GB/T14848-1993 or not worse than background of groundwater quality and can be used for fishery, industry and agriculture.

The reclaimed water infiltration in the early period may leach a lot of ammoniac nitrogen and calcium sulphate from the vadose zone and make high concentrations in TDS of 2-4 g/L and ammoniac nitrogen of 1-20 mg/L in water from vadose zone and shallow saturated zone and tend to decrease with continuation of leaching.

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Soil Aquifer Treatment Using Advanced Primary Effluent

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Abstract
Soil aquifer treatment (SAT) using primary effluent (PE) is an attractive option for wastewater treatment and reuse in many developing countries with no or minimal wastewater treatment. One of the main limitations of SAT of PE is rapid clogging of the infiltration basin due to high suspended solid concentrations. Some pre-treatment of PE before infiltration is likely to reduce this limitation, improve performance of SAT and help to implement this technology effectively. The effects of three pre-treatment options namely sedimentation (SED), coagulation (COAG) and horizontal roughing filtration (HRF) on SAT were analyzed by conducting laboratory-scale batch and soil column experiments. The sedimentation and coagulation pre-treatments led to less head loss development and reduction of clogging effect. The head loss development in soil column using PE+COAG and PE+SED was reduced by 85% and 72%, respectively, compared to PE alone without any pre-treatment. The overall dissolved organic carbon (DOC) removals of pre-treatments and soil column collectively were 34%, 44%, 51% and 43.5% for PE without any pre-treatment, PE+SED, PE+COAG and PE+HRF respectively. Coagulation pre-treatment of PE was found to be the most effective option in terms of suspended solids, DOC and nitrogen removal. Sedimentation pre-treatment of PE could be attractive where land is relatively less expensive for the construction of sedimentation basins.

Keywords
Soil aquifer treatment, primary effluent, pre-treatment, coagulation, sedimentation, bulk organic matter,

INTRODUCTION

Soil Aquifer Treatment (SAT) is a managed aquifer recharge (MAR) technology, which in combination with other available wastewater treatment technologies can produce effluent of acceptable quality for indirect potable reuse. It is a low cost and appropriate option for wastewater reclamation that ensures sustainability of both surface water and groundwater sources within the context of integrated water resources management. It is equally attractive for developed as well as developing countries as it is robust, removes multiple contaminants, is environment friendly, and minimizes the use of chemicals and energy. During SAT treated wastewater effluents applied to the ground are infiltrated and further purified through physical, chemical and biological mechanisms taking place in unsaturated (vadose) and saturated (aquifer) zones during soil passage. The performance of a SAT system, however, depends on quality of the effluent, hydrogeological conditions at the site and process conditions (hydraulic loading rate, pre and post treatment, wetting and drying cycle) applied (Amy and Drewes, 2007; Sharma et al., 2008).

SAT has been applied for further treatment and reuse of primary, secondary and tertiary effluents from wastewater treatment plants (Wilson et al., 1995; Nema et al., 2001; Crites et al., 2006). SAT using primary effluent (PE) could be an attractive option in many developing countries where there is no or minimal wastewater treatment and where there is a need to increase the existing water resources to meet the increasing water demand for different water uses. However, a SAT system using PE has a limitation of clogging (reducing infiltration rate) (Lance et al., 1980; Carlson et al., 1982; Rice and Bouwer, 1984). A proper pre-treatment of PE before applying it to the infiltration basin for SAT is likely to reduce clogging, increase the bulk organic matter removal efficiency and improve overall performance of SAT. Therefore, this study investigated the effect of different advanced pre-treatment of PE in order to increase the infiltration rate as well as removal efficiency of bulk organic matter during SAT.

MATERIALS AND METHODS

Characteristics of the primary effluent used

The wastewater primary effluents used in this study were collected from a full-scale conventional wastewater treatment plant in the Netherlands. The average wastewater quality parameters of the primary effluent are presented in Table 1.

Table 1 Typical quality characteristics of primary effluent

Parameters	Unit	Values
DOC*	mg/L	35 ± 3
DOC	mg/L	25 ± 6
pH		7.7 ± 0.4
Temperature	˚C	16 ± 2.5
Dissolved oxygen	mg/L	0.8 ± 0.2
Dissolved oxygen (after aeration)**	mg/L	8.4 ± 1.6
EC	µs /cm	1055 ± 45
NO_3-N	mg/L	0.37 ± 0.28
NH_4+-N	mg/L	37.2 ± 0.8
TSS *	mg/L	135 ± 10
TSS	mg/L	85 ± 5
UVA_{254} *	1/cm	1.0 ± 0.1
UVA_{254}	1/cm	0.89 ± 0.1
SUVA *	L/mg-m	3.04
SUVA	L/mg-m	2.89 ± 0.2

* Before 1 hr of settling

** After aeration of the primary effluent in the laboratory, before feeding to soil columns

Experimental Setups

The experimental process during this research consisted of two main parts: (i) pre-treatment of primary effluent and (ii) laboratory-scale soil column experiments simulating SAT

Pre-treatment: Three different pre-treatment options for primary effluent analyzed during this study were (i) sedimentation (SED), (ii) coagulation (COAG) and (iii) horizontal roughing filtration (HRF). Sedimentation pre-treatment of primary effluent was achieved by storing it in 60 L containers for 3 days and the supernatant was used for soil column studies. Jar test experiments were conducted using ferric chloride to determine the optimum conditions for coagulation of PE with respect to suspended solids removal. The coagulation pre-treatment of PE was done at coagulant dose of 10 mg Fe/L with rapid mixing at 150 rpm for 1 minute, slowing mixing at 15 rpm for 20 minutes followed by sedimentation for 30 minutes. The horizontal roughing filtration (HRF) pre-treatment set up consisted of 100 mm diameter PVC pipe of 1 m length with two compartments of 50 cm each; the fist compartment was filled with gravel of 12-18 mm size and the second compartment was filled with gravel of 4-12 mm size. The hydraulic loading rate for HRF was 0.3 m/h.

Soil column experiments: Soil column experiments were conducted at different process conditions using primary effluent (PE) with and without pre-treatment. The laboratory-scale SAT simulation system consisted of two sets of soil column experimental setups, each made of 2 PVC columns of 54 mm internal diameter and 2.5 m length each connected in series Silica sand (size 0.8 to 1.25 mm) was used as the filter media. The hydraulic loading rate (HLR) was maintained at 1.25±0.05 m/day. Aerobic conditions were maintained in the soil columns by aeration of wastewater treatment plant primary effluents before feeding them to the soil columns. Furthermore, aerobic conditions were confirmed by measurement of oxygen profile along the depth of the soil columns. The soil columns were also provided with the manometers to monitor the head loss development along the depth of the column.

For pre-treated effluents, suspended solids (SS) and DOC concentrations were measured. For soil column studies, DOC, UVA_{254} (UV absorbance at 254 nm), specific UV absorbance (SUVA), oxygen and nitrogen (ammonia and nitrate) profiles along the depth of the soil column were monitored. Furthermore, fluorescence excitation-emission matrix (F-EEM) of influent and effluents of soil columns was also analyzed using a spectrofluorometer (FluoroMax-3, HORIBA Jobin Yvon, Inc., USA) and the change in intensity of three characteristic organic matter peaks were calculated (Baker and Lamont-Black, 2001; Leenheer and Croué, 2003). Further details of the experimental setups, water quality and process conditions applied, and analytical methods used are presented in Hussen (2009).

Acclimation of the soil columns: At first, the soil columns were biologically acclimated (ripened) using settled primary effluent under aerobic conditions. Pre-settling of primary effluent for 1 hour was done in order to reduce the total suspended solids (TSS) that were expected to reduce the length of experimental run due to clogging. The average influent DOC concentration during this ripening period was 35± 3 mg/L. The effluent DOC concentration decreased with time and it took about 30 days to reach steady state with respect to DOC removal. This can be attributed to the development of biofilm on the filter media. About 34% DOC removal was achieved after 30 days of soil column ripening. Each time after changing the process conditions and feed water, soil columns were again allowed to attain steady state with respect to DOC removal before taking samples to study the effect of a particular parameter.

RESULTS AND DISCUSSION

Effect of Pre-treatment on SS and DOC Removal

Table 2 summarizes the average removal efficiency of SS and DOC from primary effluent for different pre-treatment options tested. Advanced pre-treatment options provided significant removal or reduction of suspended solids of primary effluent. The removal of SS during sedimentation and coagulation pre-treatment were higher (>85%) and better than that with HRF. The removals of DOC in all three pre-treatment options were comparable.

Table 2 Average SS and DOC removals with different pre-treatment of primary effluent

Pre-treatment	Suspended solids (SS) mg/L			DOC (mg/L)		
	Influent	Effluent	Removal %	Influent	Effluent	Removal %
Coagulation followed by sedimentation (PE+COAG)	141	20	85.5	29.6	25.5	13.8
Sedimentation for 3 days (PE+SED)	140	19	86.1	27.8	25.6	9.3
Horizontal roughing filtration (PE+HRF)	80	30	62.5	27.1	24.0	11.5

DOC and UVA_{254} Removal

After ripening of the of the soil columns, DOC, UVA_{254} and SS concentrations along the depth of column were monitored. Figure 1 presents the removals of DOC along the depth of soil column fed with primary effluent, with and without pre-treatment. It was observed that, in all cases, most of the DOC removal was taking place in the top 1 m of the soil column. The average DOC removals in the column were found to 34%, 34%, 37% and 32% for PE without any pre-treatment, PE+SED, PE+COAG and PE+HRF respectively. Table 3 summarizes the overall DOC removal with and without different pre-treatments. It can be seen that PE+COAG gave the highest overall DOC removal while the effects of PE+SED and PE+HRF on DOC removal were comparable.

Table 3 Summary of DOC removal during pre-treatment and soil column experiments

Influent applied	DOC removal (%)		
	Pre-treatment	Soil column	Overall
PE	-	34	34
PE+SED	9.3	34	43
PE+COAG	13.8	37	50.8
PE+HRF	11.5	32	43.5

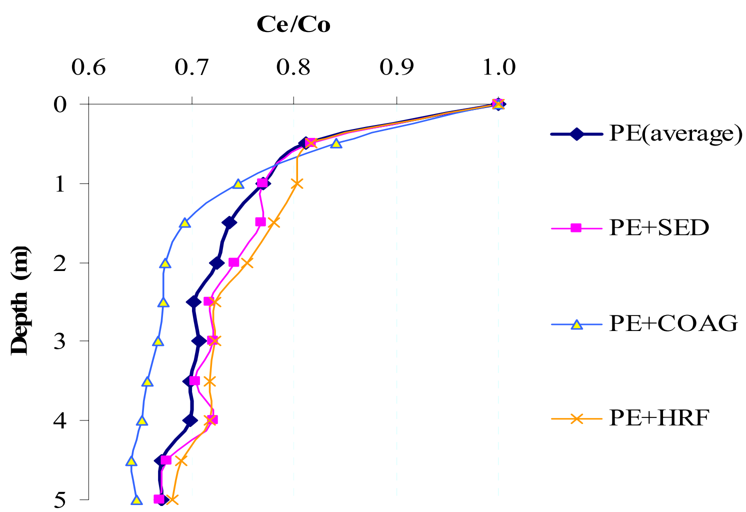

Figure 1 Removals of DOC along the depth of soil column for primary effluent with different pre-treatment (HLR = 1.25 m/d, media size =0.8-1.25 mm, depth of soil column = 5 m).

Figure 2 show the UVA_{254} and SUVA profiles along the depth of the soil column under different process conditions. For all effluents, it was observed that UVA_{254} decreased and SUVA values increased along the depth of the soil column, indicating that SAT preferentially removes non-humic (more biodegradable) fractions of the organic matter.

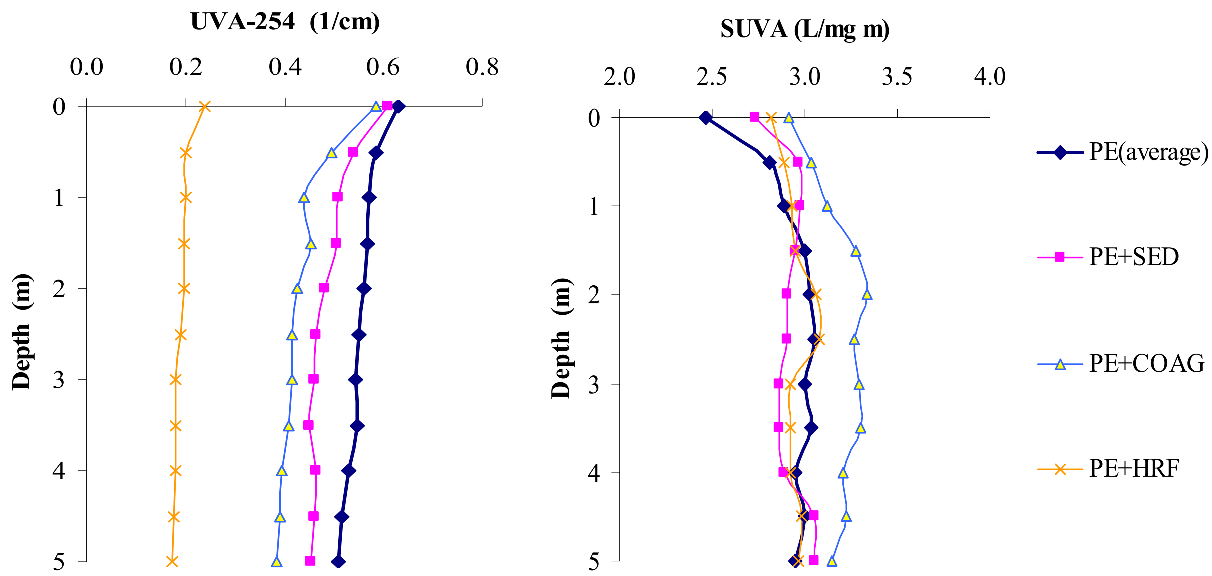

Figure 2 UVA-254 and SUVA profiles along the depth of soil column for primary effluent with different pre-treatment (HLR = 1.25 m/d, media size =0.8-1.25 mm, depth of soil column = 5 m).

F-EEM analysis of the soil column influents and effluents

Analysis of the F-EEM spectra of different types of influents and effluents from soil column revealed that all of the three characteristic organic matter fractions (peaks) namely humic/fulvic-like, humic-like and protein-like, were present in all of the samples. Table 4 presents the reduction in intensity of peaks of three organic matter fractions for primary effluent with different pre-treatment. In most cases it was observed that SAT preferentially removes protein-like (more biodegradable) organic matter fractions while removals of other organic matter fractions were limited.

Table 4 Reduction in intensity (%) of characteristic organic matter peaks during soil passage for primary effluent with different pre-treatment

Influent applied	Reduction of intensity of peaks (%)		
	Humic/Fulvic-like[1]	Humic-like[2]	Protein-like[3]
PE	5.1	1.9	44.5
PE+SED	12.7	10.0	23.1
PE+COAG	4.6	17.8	47.7
PE+HRF***	6.8	6.2	2.2

1. Humic/fulvic like peak at excitation = 330-350 nm and emission = 420-480 nm
2. Humic-like peak at at excitation = 250-260 nm and emission = 380-480 nm
3. Protein-like peak at excitation = 250-280 nm and emission = 280–350 nm

*** The quality of PE during PE+HRF study was diluted by 70% at WWTP due to rainfall.

Nitrogen Removal

Figure 3 shows the ammonium and nitrate profiles along the depth of soil columns for different pre-treatments of primary effluent. It was observed that some nitrification and denitrification can be achieved during soil passage. Ammonium concentrations were decreasing and nitrate concentrations were increasing in the top 1 m of the soil column because of the nitrification. Nitrate concentrations decreased again in the lower part of the soil column because of denitrification. Table 5 summarizes ammonium and total nitrogen removals during pre-treatment, during soil passage and overall removal for different pre-treatments applied.

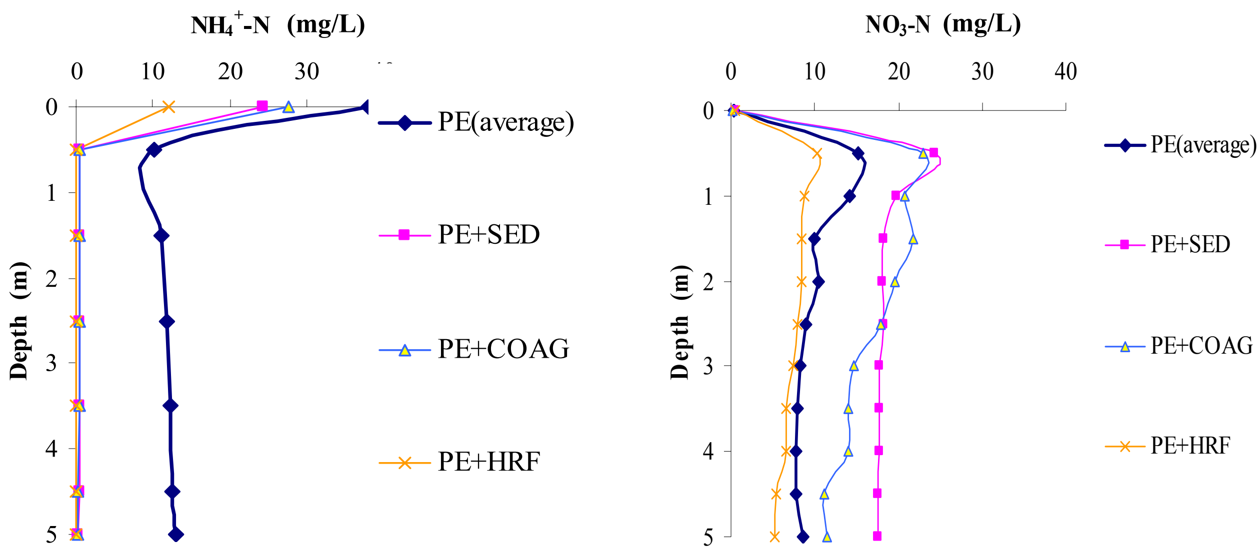

Figure 3 NH_4^+ and NO_3^- profiles along the depth of soil columns for primary effluent with different pre-treatments (HLR = 1.25 m/day)

Table 5 Ammonium and total nitrogen removal during SAT for primary effluent with different pre-treatments

Influent applied	**Ammonium Removal (%)**			**Total Nitrogen Removal (%)**		
	Pre-treatment	Soil column	Overall	Pre-treatment	Soil column	Overall
PE alone	0	66	66	0	44	44
PE+SED	35	63.3	98.3	34.5	28.6	63.1
PE+COAG	27.4	71.8	99.2	26.8	42.1	70.9

It was observed that PE+COAG and PE+SED resulted in very high removals of ammonium (>98%) whereas PE+COAG was the best option in terms of total nitrogen removal. It is expected that in deeper soil column (as in field sites), some zones will have anoxic conditions which will further promote denitrification.

Effect of Pre-treatment on Clogging/Head Loss Development

The average suspended solids concentrations of the soil column effluents were 0.55 mg/L, 0.60 mg/L, 0.95 mg/L and 1.66 mg/L for PE+COAG, PE+SED, PE+HRF and PE alone, respectively. This shows that regardless of the type of the feed water or pre-treatment, SAT effectively removes suspended solids from wastewater treatment plant effluents within the depth of 5 m. The main consequence of suspended solids removal during soil passage is clogging of the basin and head loss development. The head loss development along the depth of the soil column was monitored by taking manometer readings with time (Figure 4). It was observed that most of the head loss development was taking place in the first 0.5 m of the soil column. The head loss increased with time for PE alone and PE+SED whereas no significant increase in head loss was observed in the case of PE with coagulation pre-treatment. The average increase in head loss development with time was 0.95 cm/hour for PE without any pre-treatment. For PE +SED the increase in head loss with time in the soil column was only 0.3 cm/ hour.

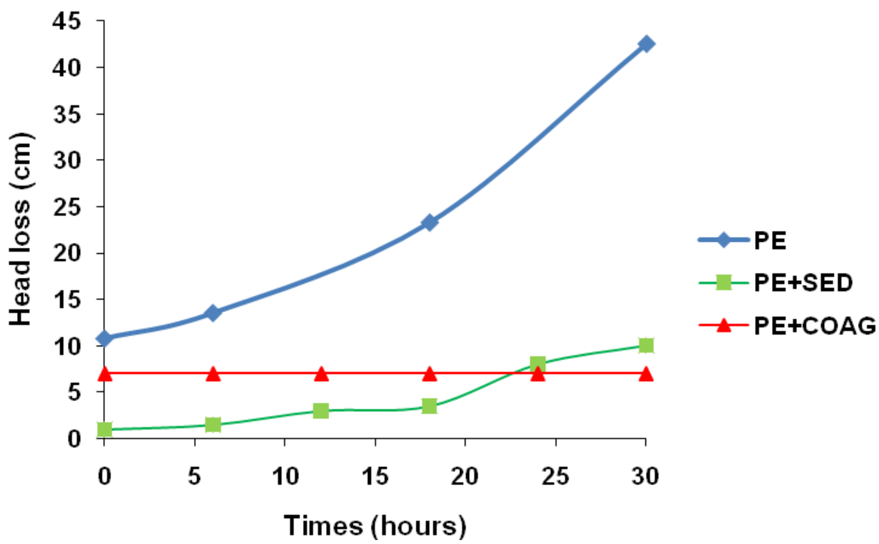

Figure 4 Head loss development with time during soil column experiment using PE and pre-treated PE (at 0.5 m depth of soil column)

The advanced pre-treatment options of sedimentation and coagulation showed less head loss development and reduction of clogging effects. Initially after 1 hour of operation, the total head loss for soil columns fed with PE+COAG and PE+SED was 50% and 60%, respectively, of the corresponding head loss for column fed with PE without any treatment. After 30 hours of operation, the increase in head loss in the soil column fed with PE+COAG and PE+SED was reduced by 85% and 72%, respectively, of the head loss in the soil column fed with PE alone without any pre-treatment.

The pre-treatment of PE before SAT showed a significant improvement in SS removal and decrease in head loss in the soil column. Therefore, the clogging effect or the reduction of infiltration rate of SAT can be improved as well by pre-treatment of PE before SAT. The advantage of improvement of infiltration rate is to minimize the frequency of cleaning of the infiltration basin (raking or scrapping of the clogging layer) and higher or longer wet/dry cycle could be applied. Sedimentation and coagulation have a comparable improvement in terms of SS removal and reduction in head loss. Coagulation followed by sedimentation is an appropriate pre-treatment option with respect to removal of DOC and nitrogen transformation, where there is a scarcity of land, where there is an easily accessible skilled operator and where the recovery of cost of operation and maintenance is high. On the other hand, a sedimentation pond is an attractive option as a pre-treatment of PE before SAT for developing countries where the land is available to construct a pre-treatment pond of considerable size. This option produces less (chemical) sludge, requires less maintenance and operation cost and does not require highly skilled operators.

CONCLUSIONS

- The suspended solids removal with coagulation and sedimentation pre-treatment of PE before SAT was more than 85% whereas it was about 60% for horizontal roughing filter pre-treatment. Additionally, DOC removals of 13.8%, 11.5% and 9.3% were observed during coagulation, horizontal roughing filtration and sedimentation pre-treatment, respectively.
- The DOC removals during soil column experiments using PE pre-treated by different options and PE without any pre-treatment were comparable. DOC removals in the soil columns using PE +COAG, PE+SED, PE+HRF and PE were found to be 37%, 34%, 32% and 34% of the influent, respectively.
- Overall DOC removals of PE+COAG, PE+SED and PE+HRF were about 51%, 43% and 44%, respectively.
- It was observed that some nitrification and denitrification can be achieved with soil columns fed with primary effluents with or without any pre-treatment. Nitrogen transformation (nitrification) in the soil column using pre-treated PE was higher than that using PE without any pre-treatment. About 98% of the NH_4^+-N was converted to NO_3^--N or removed during SAT using PE pre-treated by coagulation and sedimentation, whereas about 66% of NH_4^+-N was converted to NO_3^- - N. Additionally about 35% and 30% of NH_4^+-N was removed by coagulation and sedimentation, respectively, during pre-treatment of PE.
- The effect of clogging on SAT basin operation by PE, reducing the infiltration rate, could be minimized by pre-treatment of PE. After 30 hours of operation of soil column, the increase in head loss was reduced by 85% and 72% of that for PE alone using PE+COAG and PE+SED, respectively.
- Coagulation pre-treatment of PE was found to be the most effective option in terms of suspended solids, DOC and nitrogen removal. Advanced primary effluent with coagulation followed by SAT could be attractive option for wastewater treatment and reuse in developing countries. Alternatively, extended sedimentation pre-treatment of PE followed by SAT could be attractive where land is relatively less expensive for the construction of sedimentation basins or ponds.

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Urban Stormwater Recycled via Aquifer Storage Transfer and Recovery in Adelaide, South Australia

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Abstract

The Aquifer Storage Transfer and Recovery (ASTR) research project was established in 2005 to assess whether reedbed-treated urban stormwater could be stored in an initially brackish limestone aquifer and recovered at a quality which meets Australian Drinking Water Guidelines. An innovative configuration was used with separate injection and recovery wells providing a minimum residence time of 6 months to investigate the effectiveness of passive treatment of the injected water in the aquifer. The research work conducted between 2005 and 2009 involved a broad range of disciplines with an extensive field program focused on exploring the fate of water quality hazards through the reedbed and aquifer.

The ASTR research project provided opportunities for scientists to participate in leading-edge research and develop national approaches to managing risks in water recycling via aquifers. With the broad range of organisations involved, the ASTR project was able to successfully foster widespread networking and collaboration between government agencies, industry and researchers. The ASTR research project has also provided valuable advances for the water industry by developing methods for measuring pathogen and trace organics attenuation.

The results of the research project show that with effective management and identified preventive measures water is able to be produced at the site that meets all of the health related drinking water quality requirements, but not all of the aesthetic requirements. This paper will highlight the major drivers and partners behind the development of the ASTR project and outline the concept, objectives and outcomes of the research undertaken with particular focus on the aquifer related components.

Keywords: attenuation, collaboration, contaminants, groundwater, stormwater, risk management

INTRODUCTION

The City of Salisbury, an expanding urban area on the Northern Adelaide Plains in South Australia, is an Australian leader in the development of urban stormwater harvesting and reuse systems. Originally, wetlands were constructed within this Council area for improving the quality of stormwater discharging to sea through coastal mangroves, for flood mitigation and for urban amenity value. Subsequently, valuable fit for purpose water quality supplies were produced by harvesting the treated stormwater from the wetlands and storing it in aquifers.

Recent severe urban water restrictions imposed by drought across Australian cities, together with a drying climate, highlighted that research was needed to investigate opportunities for enhanced utilisation of stormwater. The Aquifer Storage Transfer and Recovery (ASTR) research project was therefore established with the following objectives:

- Demonstrate that stormwater can be treated using passive methods and stored in aquifers to be recovered at a quality which meets the Australian Drinking Water Guidelines; and
- Demonstrate that the framework used for managing water quality and matching treatment processes with subsurface water quality improvement are robust, sustainable and transferable.

To achieve these objectives, the ASTR research project has involved assessing urban stormwater quality, the design and operation of a specialised subsurface storage and recovery system, and assessing reedbed and aquifer treatment effectiveness. This paper provides an overview of the scheme, outlines the research undertaken, summarises the Managed Aquifer Research (MAR) related outcomes, and the benefits of broad collaboration.

SCHEME OVERVIEW AND OPERATION

Located at Parafield Airport in the City of Salisbury, the ASTR research project uses an established stormwater treatment system to collect water from an urban catchment prior to passage through a stormwater harvesting system and storage in the aquifer (see Figure 1). The outflow from the reedbed is pumped to a well-field which encompasses six production wells spaced at 50 m in a quadrilateral configuration (see Figure 2). In routine operation the four outer wells are used to inject treated stormwater (i.e. IW1), and the two inner wells (i.e. RW1) are used to recover stored water from the aquifer. The innovative configuration provides a minimum of 6 months storage and was designed to investigate the effectiveness of passive treatment of the injected water in the anoxic aquifer (see Figure 3).

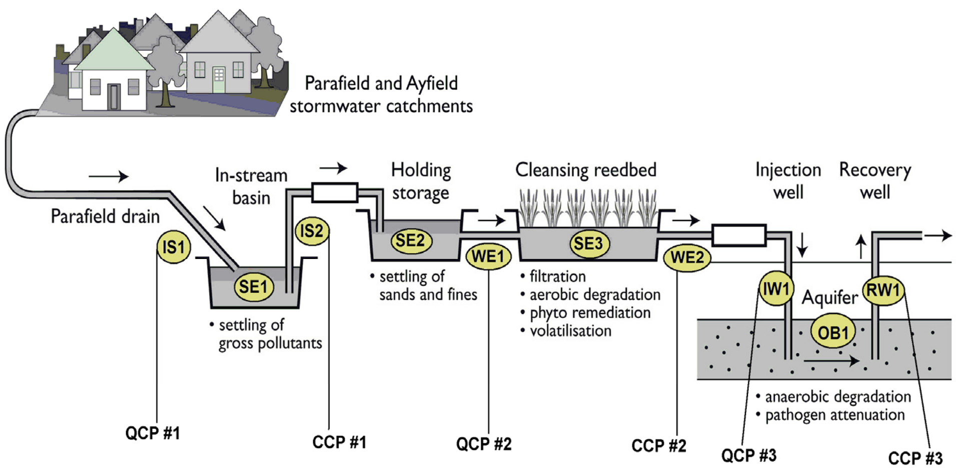

Figure 1: The ASTR scheme in the City of Salisbury, showing critical control points (CCPs), quality control points (QCPs), water and sediment quality sampling points (after Page et al. 2010)

The injection and recovery wells have an open interval at a depth of 165 to 185 m in a relatively uniform sequence of limestone and sand (known locally as the T2 aquifer) which has a low-moderate permeability (hydraulic conductivity approximately 2.5 m/d). The T2 aquifer is overlain by a clay layer that provides an aquitard which prevents: (i) migration of injected water into the overlying aquifer; and (ii) any migration of pollutants into the T2 aquifer from the above unconfined aquifer. The aquifer is in a nitrate-reducing redox state, is at an ambient temperature of 23°C and is low in oxygen. The ASTR system, including the well configuration and the groundwater modelling undertaken, is described in detail by Kremer et al. (2010).

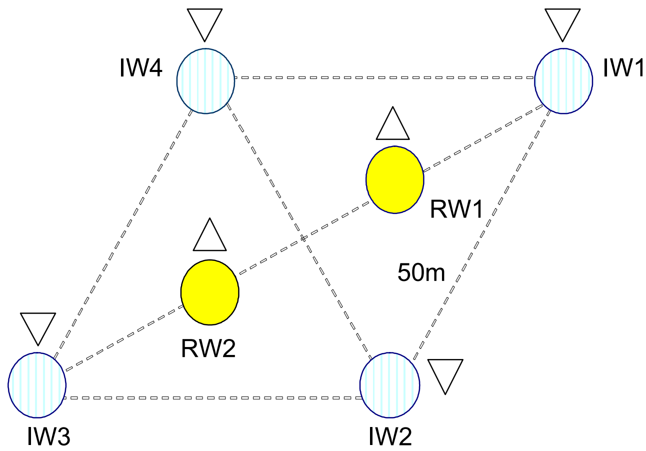

Figure 2: Schematic of the ASTR Well-field Configuration

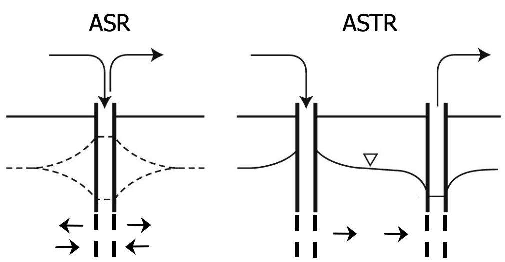

Figure 3: Schematic highlighting the difference between ASR and ASTR

The confined aquifer is naturally brackish with an ambient groundwater salinity of 2,000 mg/L (electrical conductivity (EC) of approximately 3,600 µS/cm). An initial flushing phase was therefore needed to freshen the storage zone within the aquifer and between September 2006 and June 2008, 373,000 m^3 was injected into the two inner wells. During this time, periodic groundwater sampling and down-hole water quality profiling was performed and EC data collected showed a clear breakthrough of the injected water across the study zone with concentrations dropping from 3,620 µS/cm in June 2006 to 591 µS/cm in August 2008. Subsequently, injection shifted to the outer wells in September 2008 and 30,000 m^3 was recharged by June 2009. Injection rates ranged between 3 - 5 L/s at the injection wells. 106,000 m^3 was recovered from the inner wells between February and April 2009 with constant extraction rates of 10 L/s at both recovery wells.

RESEARCH UNDERTAKEN AND OUTCOMES

The role of the aquifer in the treatment train has not been considered with the same rigor as other engineered treatment components such as filtration or disinfection (Dillon et al. 2008; Page et al. 2010; in press). Therefore, one of the key objectives of the research undertaken was to determine whether utilising separate injection and recovery wells provided more reliable inactivation of pathogens and biodegradation of trace organics through a longer and more uniform residence time in the aquifer. The research conducted between 2005 and 2009 which relates to this objective is summarised in Table 1 and the outcomes from a number of these aspects are summarised below. Further information can be obtained from the references provided.

Table 1: Research Undertaken to Assess the Effectiveness of ASTR

Research Aspect	Research Undertaken	References
Decay of Enteric Pathogens	Studies of pathogen attenuation in the aquifer	– Toze et al. (2009) – Page et al. (2010) – Page et al. (in press)
Attenuation of Organic Chemicals	Studies of herbicide removal in stormwater harvesting wetlands	– Page et al. (submitted)
Geochemical Modelling	Estimate likelihood of release of inorganic constituents from the aquifer matrix into groundwater recovered water	– Page et al (2009) – Vanderzalm et al. (in press).
Groundwater Flow and Solute Transport Modelling	- Modelling the residence time and salinity of water recovered from the aquifer under a range of operational scenarios using FEFLOW (2D & 3D models) - Prediction of the performance of the scheme	– Pavelic et al. (2004) – Kremer et al. (2008) – Kremer et al. (2010)
MAR Risk Assessment	Assess the safety of the use of recovered water as a drinking water supply	– Page et al. (2008) – Page et al. (2009) – Page et al. (in press)

Decay of Pathogens

The redox status of the aquifer has a major influence on the rate of degradation of pathogens, organic chemicals and inorganics introduced into the aquifer (Toze et al. 2010). Although no human enteric pathogens were found in the source water, the rate of natural die-off of pathogens in the aquifer was measured using the *in-situ* diffusion chamber method (Toze et al. 2010) for a range of representative pathogens including bacteria, viruses and protozoa. The die-off rate of bacterial pathogens was fast (i.e. in the order of days) but protozoa and viruses were more variable and generally slower. The results of the quantitative microbial risk assessment showed that the aquifer ranked less than UV and chlorination disinfection but higher than the reedbed in the reduction of the risk from rotavirus, while the aquifer ranked in between UV disinfection and chlorination for *Cryptosporidium*. The aquifer was therefore identified as a potential barrier, which can exhibit a high capacity for water treatment when combined with other engineered barriers (Page et al. 2010).

Attenuation of Herbicides

An extensive suite of organic chemicals was analysed in the reedbed and the aquifer. Simazine was the most frequently detected organic chemical within the outlet of the cleansing reedbed over a three year period. However, simazine was not detected in samples taken from the injection and recovery wells, suggesting sorption or biodegradation during aquifer storage (Page et al. 2009). Simazine is known to have an average half-life of 60 days under aerobic conditions, but there is no data on simazine degradation rates in anaerobic aquifers. Monitoring using novel passive samplers indicated that the reedbed treatment barrier was effective in removing approximately 50% of these herbicides. Future research aims to also characterise the aquifer treatment barrier with respect to organic chemicals (Page et al. submitted).

Geochemical Modelling

The chemistry of water stored in an aquifer is affected by the quality of the source water, the conditions within the aquifer and chemical reactions between the source water and the aquifer material or the ambient groundwater. Geochemical modelling of aquifer processes was therefore undertaken to:

- Determine the longevity of specific facets of aquifer treatment at the ASTR site; and
- Consider any potential hazard sources in the system.

The analysis found that:

- Subsurface reactions could lead to increased concentrations of arsenic, iron, manganese, trace species (eg cadmium, chromium, lead) or hydrogen sulphide in the recovered water (Page et al, 2009).
- Nutrient removal is occurring along the injection flow-path. However, there was insufficient groundwater quality data to validate nutrient removal processes (Page et al, 2009).
- Recovered water did not reveal mobilisation of iron or arsenic in the aquifer. However, recovered water did contain iron near Australian Drinking Water Guideline levels and on occasions exceeded the aesthetic limit for colour (which can cause iron staining). It should be noted that the native groundwater and stormwater both contain iron concentrations in excess of aesthetic Australian Drinking Water Guideline values at the ASTR site.

Groundwater Flow and Solute Transport Modelling

Groundwater flow and solute transport modelling of the ASTR site has involved both 2D and 3D modelling and was undertaken using the FEFELOW software (Kremer et al. 2008; 2010). Simulations of the mixing fraction of brackish groundwater with fresh stormwater indicated that the salinity of the recovered water would remain below the Australian Drinking Water Guidelines limit (500 mg/L TDS) based on the assumed operating conditions. The results from the modelling therefore support the expected viability of ASTR operations even under stressed (i.e. low rainfall) conditions and recommends that the recovery efficiency should be maintained at approximately 80% to enhance the buffer zone. The validated modelling provides a predictive tool which is able to simulate the fate of injected water within the aquifer to assess the viability of the ASTR scheme under various operational scenarios.

Risk Assessments

The ASTR project has assisted in the development of the Australian Guidelines for Water Recycling: Augmentation of Drinking Water Supplies, Stormwater Harvesting and Reuse and Managed Aquifer Recharge (MAR) (EPHC–NHMRC–NRMMC, 2008, 2009a, 2009b) which provides a risk management framework to assist the protection of human health and the environment. These guidelines provide a consistent scientific approach to regulation and environmental issues associated with MAR and have now been adopted nationally across Australia. The ASTR research project has been able to provide data, develop methods, test the risk assessment approach, and provide example risk assessments for other practitioners to follow (see Page et al, 2009).

COLLABORATION AND KNOWLEDGE TRANSFER

The ASTR research project brought together seven South Australian organisations (as summarised in Table 2) with a commitment to conduct research into alternative water supplies in the City of Salisbury through innovation, advancing science and building knowledge and confidence. The ASTR research project was financially supported by the Government of South Australia and the Australian Government. However, the success of the project has also been reliant on the expertise and the cash and in-kind contributions of project partners and stakeholders. The ASTR project, through the widespread collaboration, has extended the knowledge of the respective organisations involved (see Table 2) and has led to the development of more MAR projects in South Australia.

In addition to local collaboration, a relationship was forged between the ASTR project team and Reclaim Water (www.reclaim-water.org) which is a specific targeted research project funded by the European Commission involving a total of 19 international partners including research institutes, universities and private organisations from western and eastern Europe, China, Singapore, Israel, Mexico and South Africa. Participation in Reclaim Water has led to extensive knowledge exchange in the field of water reuse including MAR, hydraulic / chemical processes modelling, risk assessment and analytical methods associated with chemical and pathogen contamination.

Table 2: South Australian Project Partners, Roles, Contributions and Key Benefits

Organisation	Type	Role	Key Benefits
CSIRO	Research Organisation (National)	ASTR research, hydrogeology, water quality & hazard assessment expertise, surface & groundwater sampling, modelling of aquifer hydraulics, membership of steering committee & technical committee	Learnt more about industry needs including operational requirements of MAR schemes
City of Salisbury	Local Government	ASTR & wetland operations, provision of land & stormwater, pump & pipeline network, water quality monitoring, in-kind support, membership of steering committee & technical committee	Learnt more about technical aspects of hydrogeology with the assistance of CSIRO
SA Water	State Government Agency	Water quality, analytical & infrastructure support, regulation advice, pipeline network, membership of steering committee	Built their organisational capacity & understanding of the issues in MAR to assess its feasibility as a water supply technique
Department of Water, Land and Biodiversity Conservation	State Government Agency	Regulation advice, water resource planning / strategy, hydrogeology, field technical investigations (geophysical logging, pump tests), membership of steering committee	Developed knowledge to assist in governance of MAR
United Water	Industry	Project management, water treatment quality advice, cash and in-kind support, website platform, knowledge transfer, membership of steering & technical committee	Built their organisational capacity & understanding of the issues in MAR with transfer or knowledge to other MAR sites
Department of Health, South Australia	State Government Agency	Regulation advice, membership of technical committee	Information to assist in refining the approvals processes for other MAR schemes
Environment Protection Authority, South Australia	State Government Agency	Regulation advice, membership of technical committee	Information to assist in refining the approvals processes for other MAR schemes

CONCLUSIONS

The research conducted between 2005 and 2009 has demonstrated that water recovered from the wells at the ASTR site generally meets the Australian Drinking Water Guidelines without further treatment. However, the results suggest that relying on stormwater via aquifers as an ongoing supply will require inclusion of disinfection (ultraviolet light and chlorination) and iron removal (aeration) treatment to achieve an acceptable low level of risk and meet the aesthetic quality requirements of the Australian Drinking Water Guidelines.

The subsurface storage and transport aspect of the ASTR scheme is not advanced enough to fully evaluate the benefits or risks as the evaluation of hydrogeochemical processes was largely based on the operation of the ASTR site in the flushing mode. Further research is therefore needed to obtain more water quality data, test the robustness of the concept and to engage with the community concerning the desirability of this source of water for various uses, to explore options for harvesting and use of stormwater (alone and in combination with other waters) and to assess impacts of its use on water distribution systems.

The ASTR project grew from an idea, to a concept, to a research project involving multiple organisations and attracting research and capital funding. The project succeeded in meeting its objectives and project milestones within the specified timeframes. The project also demonstrated the benefit of collaboration between industry, research organisations and local and state government agencies, including regulators, from the outset of a project and has established a model which is now being utilised in subsequent MAR projects.

ACKNOWLEDGEMENTS

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Reclaimed Water ASR in a Barrier Island Shallow Aquifer, Destin, Florida.

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ABSTRACT

Management of reclaimed water in coastal communities is an increasing challenge because of growing populations and associated reclaimed water flows, environmental concerns, and usually great land acquisition costs. Destin Water Users, Inc. (DWU) implemented an aquifer storage and recovery (ASR) program that will provide cost-effective and environmentally sound additional peak “disposal” capacity and additional water to meet reuse demands during peak irrigation periods, which will reduce demands on freshwater resources. Tertiary-treated wastewater is stored in the shallow Sand-and-Gravel Aquifer that contains freshwater, but has poor quality. The DWU ASR system is designed and permitted to consist of seven ASR wells with a total capacity of 8,040 m^3/d. Initial testing and solute-transport modeling results indicate that the storage zone has a low dispersivity and high effective porosity, which results in low degrees of mixing and migration of the injected water. Despite its apparent clean quartz sand storage zone, arsenic leaching has occurred in the recovered water with a maximum reported value of 41 µg/L. Concentrations have subsequently decreased suggesting depletion of a limited leachable arsenic supply. The DWU ASR project is the first of its kind in Florida and may serve as a prototype for similar systems on barrier islands elsewhere in the world.

INTRODUCTION

Water resources management in many coastal areas of the world has become a great challenge because of a combination of limited freshwater resources, susceptibility of groundwater resources to contamination from saline-water intrusion, and increasing demands associated with population growth that is often concentrated in coastal communities. Barrier island communities are particularly vulnerable because of the paucity of fresh surface water and shallow, locally recharged groundwater.

Safe disposal of wastewater can also be a major challenge because of the environmental sensitivity of coastal areas and very limited available, and often prohibitively expensive, undeveloped land for surface application methods. It is now widely recognized that reclaimed water (also referred to as treated sewage effluent) is a valuable water resource rather than a disposal problem. Reuse of reclaimed water is an important element of water conservation in that it reserves high-quality freshwater resources for greater values, particularly potable water supply. Reuse also eliminates many or all of the environmental impacts and costs associated with reclaimed water disposal.

Communities vary in the annual percentage of their reclaimed water flows that are reused. In some instances, the existing reuse infrastructure and demands may not be adequate to accept the entire reclaimed water flows. In areas with substantial seasonal variation in precipitation, and thus irrigation requirements, there may be relatively little demand for reclaimed water during wet periods. Greater reuse could occur if excess water can be stored during periods of excess supply for later use during high demand periods. Such seasonal, large-volume storage of reclaimed water is possible using aquifer storage and recovery (ASR) technology. The storage of reclaimed water possible with ASR can also increase the demand for reclaimed water by increasing the reliability of the supply. Irrigators are often reluctant to commit to reuse systems unless they are guaranteed water during both seasonal dry periods and droughts, when they need the reclaimed water the most. Reliable supply thus results in increased demand.

Barrier island and other coastal areas are often underlain by the shallow aquifers that are not suitable for potable water use. Either water quality is too poor for potable use or sustainable yields are inadequate for significant long-term pumping because of low recharge rates. However, these shallow aquifers may be best used as the storage zone for reclaimed water ASR systems. The storage provided by the ASR system can increase the reuse rate of reclaimed water, and the balancing of injection and recovery will allow the operation of the system without inducing long-term saline water intrusion. Destin Water Users, Inc. (DWU), which serves the City of Destin located on a barrier island in northwestern Florida (Figure 1), is developing an ASR system in which tertiary-treated wastewater is being stored in a shallow, semi-confined, quartz-sand aquifer.

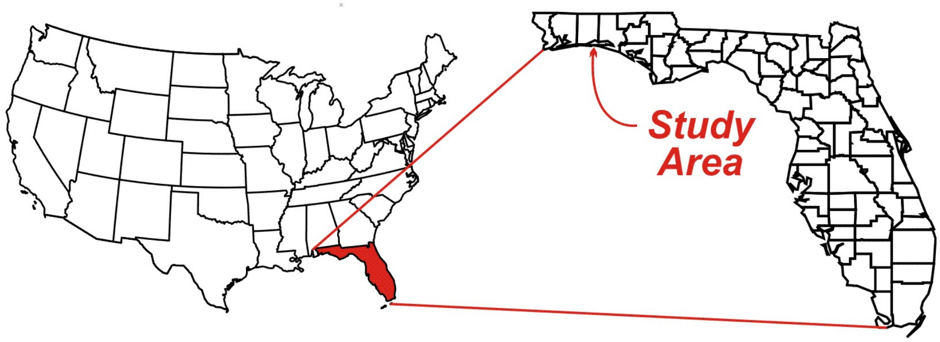

Figure 1. Location map.

DESTIN WATER USERS WATER MANAGEMENT CHALLENGES

The potable water source for the City of Destin is the Upper Floridan Aquifer of the Floridan Aquifer System (Figure 2), which is a confined aquifer that is recharged on the mainland. Further withdrawals from the Upper Floridan Aquifer are limited by concerns over saline-water intrusion. The Sand-and-Gravel Aquifer contains freshwater, but is not directly suitable for potable water use because of high iron and hydrogen sulfide concentrations. Use of the Sand-and-Gravel Aquifer for potable use is prohibited in the City of Destin by a local ordinance. The Sand-and-Gravel Aquifer is widely used for residential irrigation. The DWU reclaimed water flow is mostly reused for irrigation by large irrigators such as golf courses. Excess reclaimed water is disposed of by land application methods and thus recharges the Surficial Zone of the Sand-and-Gravel Aquifer.

Like many other growing coastal communities, Destin needs additional peak flow wastewater disposal capacity. As a resort area, environmental protection is a major concern, and limited undeveloped land and high land costs effectively preclude expansion of the land application system. An off-shore discharge would be expensive to permit and construct and would likely elicit strong public opposition. Expansion of the reuse system is constrained by the lack of a reliable additional supply during peak demand periods.

DWU implemented an ASR program because the storage provided by the system would provide the following benefits:

- ASR is the most cost-effective option for providing needed peak flow "disposal" capacity,
- ASR avoids the potential environmental impacts and objections of other disposal options such as surface water outfall,
- Storage of excess reclaimed water for later use avoids the waste of a valuable resource, and
- The stored water will increase dry season supply and reliability and thus allow DWU to serve more customers reducing demand on fresh groundwater resources.

The DWU ASR system has a design capacity of 8,040 m^3/d (2.125 million gallons per day) and will consist of seven ASR wells and associated monitor wells (Figure 3). The storage zone is the Main-Producing Zone of the Sand-and-Gravel Aquifer (Figure 2), which consists predominantly of Mioceneaged medium sand to gravel-sized quartz. The native groundwater in the Main-Producing Zone is suitable for direct use for irrigation. However, expansion of production in the zone creates risk of contamination by saline-water intrusion. The ASR system will allow for long-term withdrawals from the zone without promoting saline-water intrusion.

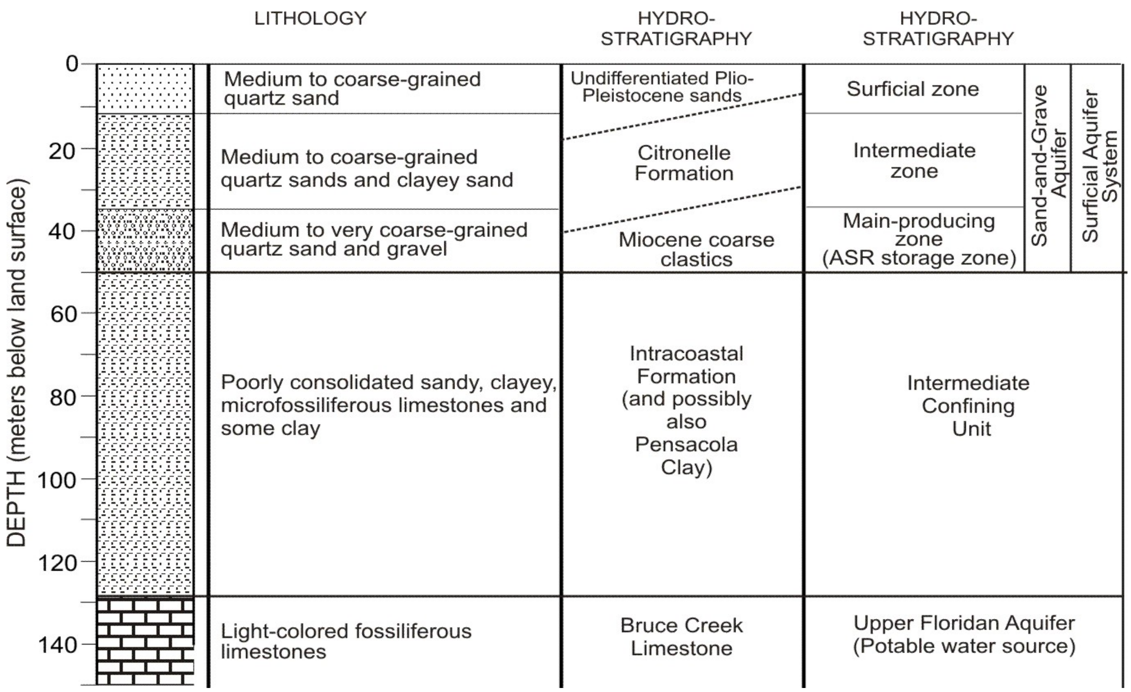

Figure 2. Geology and hydrogeology of the Destin area.

The DWU ASR system will be constructed and tested in two phases. Phase I consists of the construction and testing of one ASR well with two associated storage-zone and one shallow monitor well. Phase II will consist of the installation of the remaining six wells and additional monitor wells. The injected reclaimed water is tertiary treated and chlorinated and meets all applicable groundwater standards.

OPERATIONAL TESTING RESULTS

The initial three operational (cycle) tests for Phase I are summarized in Table 1. The injected and native waters can be differentiated based on compositional differences, particularly chloride, sodium, total dissolved solids, and fluoride concentrations. Chloride concentration is the preferred tracer because it is conservative, can be measured with high accuracy, and there are distinct differences between the injected and native waters. The percentage of injected reclaimed water in the recovered water can be estimated using binary, linear mixing equations.

A calibrated solute-transport model was performed to simulate the percentage of injected water in the recovered water. The objectives of the modeling were to develop a better understanding of the hydrogeology and mixing processes in the storage zone through the calibration process and to develop a predictive tool that would assist in the design of ASR system expansion and development of operating protocols.

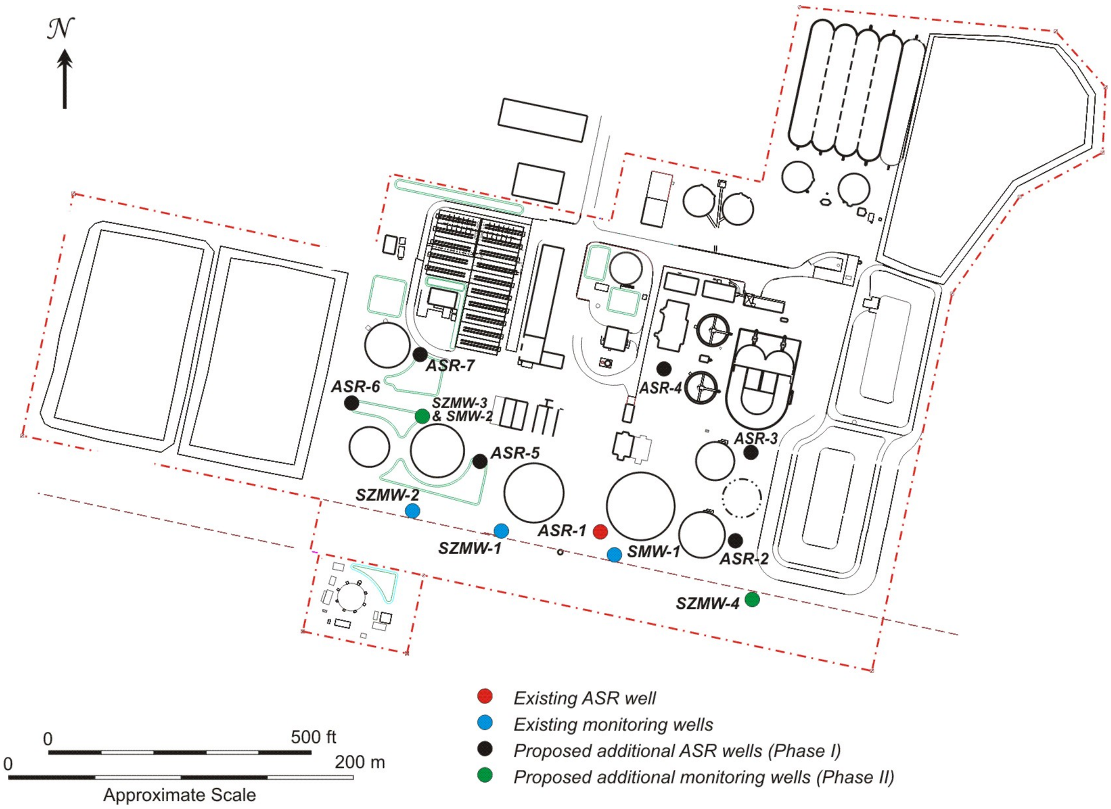

Figure 3. Site plan showing well locations.

Table 1. Summary of Cycle Tests No. 1 through 3						
Cycle Test No. 1						
Phase	**Start Date**	**Volume**		**Duration (days)**	**Average Rate**	
		m^3	**(MG)**		**m^3/day**	**(gal/day)**
Injection	June 10, 2009	9,641	2.547	10	964	254,662
Storage	-	-	N/A	0	-	-
Recovery	June 20, 2009	9,569	2.528	12	798	210,679
Cycle Test No. 2						
Injection	August 21, 2009	8,086	2.136	9	897	237,300
Storage	August 30, 2009	-	-	10	-	-
Recovery	September 9, 2009	8,017	2.118	9	896	235,300
Cycle Test No. 3						
Injection	October 21, 2009	11,610	3.067	19	611	161,424
Storage	November 8, 2009	-	-	22	-	-

The field data and simulation results indicate that the injected water remained close to the ASR wells with a relatively low degree of dispersive mixing. A small grid size (0.38 m, 1.25 ft) and low longitudinal dispersivity (0.09 m; 0.3 ft) were required to obtain a reasonably close match to the observed results. The model still tends to slightly underestimate the percentage of recovered water due to numerical dispersion. There was no suggestion that the injected water entered any of the monitor wells. The low degree of dispersive mixing is attributed to groundwater flow being intergranular in the unconsolidated sands. A low degree of aquifer heterogeneity and high effective porosity results in the injected water volume being compact and remaining close to the ASR well.

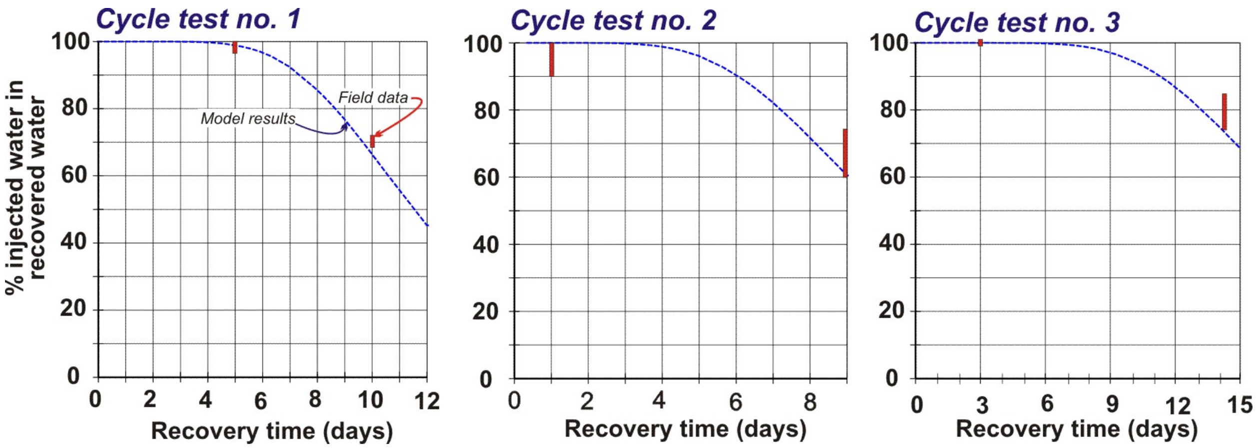

Figure 4. Simulated recovered water composition (dashed line) and field data (vertical bars).

ARESNIC LEACHING

The leaching of arsenic into water stored in ASR systems in Florida has become a major regulatory concern since the lowering of the arsenic primary drinking water maximum contaminant level (MCL), and thus groundwater standard, from 0.050 mg/L to 0.010 mg/L. The source of the arsenic is believed to be the oxidation of trace arsenic-bearing pyrite in the storage zone strata (Arthur et al., 2001, 2002, 2007; Mirecki, 2004, 2006a, 2006b).

It was expected at the onset of the DWU ASR project that the system would have a low susceptibility to arsenic leaching because of the relatively clean sand mineralogy of the storage zone. Nevertheless, arsenic leaching did occur at concentrations exceeding the 0.010 mg/L standard. Concentrations tended to progressively increase during recovery (Figure 5). Overall concentrations decreased with each successive operational test, which suggests that there is a limited amount of leachable arsenic in the sands and gravels, and it is being depleted.

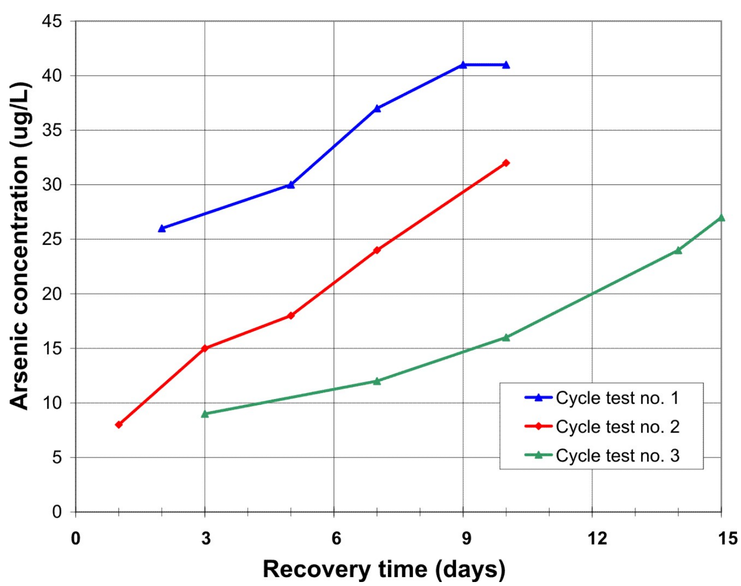

Figure 5. Arsenic concentration data in the recovered water for the first three operational cycles.

An additional geochemical issue is managing the total trihalomethanes (THMs) concentration. The injected water is required to meet the primary drinking water MCL of 80 µg/L. The disinfection system must be operated so that the injected water meets the groundwater total coliform bacteria standard of 4 cfu/100 mL, while not exceeding the total THMs MCL. The injected water from cycle tests 2 and 3 meet the THMs MCL, but the initial recovered water exceeded the standard. The data from operational test 3 are provided in Table 2.

Table 2. THM Data from the Recovery Phase of Operational Test 3

Parameter	12/3/09	12/10/09	12/24/09
Chloroform (µg/L)	51	43	42
Bromodichloromethane (µg/L)	34	26	23
Dibromochloromethane (µg/L)	23	18	15
Bromoform (µg/L)	4.3	3.6	2.7
Trihalomethane total (µg/L)	112.3	90.6	82.7
Chloride (µg/L)	117	107	95.5

The data indicate the THMs had formed in the storage zone after injection, which was followed by the onset of their degradation. Brominated THMs are preferentially removed. Both field and experiment studies indicate that the concentrations of brominated THMs are attenuated by biological processes in chemically reducing groundwater environments (e.g., Bouwer and Wright, 1988; Pyne et al., 1996; Pavelic et al., 2005a, b; Singer et al.,1993). Chloroform is more refractory and appears to require highly reducing conditions for rapid biological degradation to occur. With the of return of reducing condition in the DWU ASR system storage zone, biodegradation of the brominated THMs will occur, and THM concentrations in the injected water would be expected to drop into the 40-50 µg/L range as only significant quantities of chloroform will remain.

CONCLUSIONS

Unconsolidated quartz sand and gravels have hydrogeologic properties that are favorable for the recovery of high percentages of the injected waters. The predominance of matrix (intergranular) flow results in a low degree of dispersive mixing, and the high effective porosities and low degree of aquifer heterogeneity result in the injected water volume being compact and remaining close to the ASR well. Shallow aquifers, such as the DWU storage zone, have the advantage of comparatively low construction costs compared to deeper aquifers.

Shallow coastal aquifers in many areas either naturally have poor or marginal water quality or have degraded water quality due to saline-water intrusion from over exploitation or other anthropogenic contamination. The DWU ASR system illustrates the value of the aquifer zoning concept. The optimal use of the Sand-and-Gravel Aquifer in the City of Destin is as a natural and enhanced reservoir (through ASR) for non-potable water for relatively low value uses (e.g., residential irrigation). Storage of reclaimed water serves the environmental and water management goals of providing a safe means for managing peak wastewater flows, conserves a valuable water resource, and reduces demands of higher quality fresh groundwater resources, which can be reserved for higher-value potable use.

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Managing the Thermal Aquifer System of Caldas Novas – Brazil

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Abstract
Caldas Novas is situated in the center of Brazil and its economy depends on the hard rock thermal aquifer system of Neo-Proterozoic age which supplies about 100 swimming pools. A dramatic drawdown of the unconfined aquifer of 40 m from 1987 to 1996 was noticed after first wells were successfully drilled.

A thermal semi-confined aquifer of karstified quartzites (>60°C) is connected with a fractured schist/quartzite unconfined aquifer. The recharge of the thermal aquifer is localized in the nearby Serra de Caldas. The main discharge is west of the Serra while a little amount of the thermal water migrates into the unconfined aquifer and mixes with the local recharged groundwater (<49°C).

The responsible authority limited the thermal water abstraction in 1996. The groundwater table could recover but under the condition that licensed quantities are not abstracted yet. To manage the aquifer artificial recharge will take place. The main source will be the used pool water. This water has still a temperature of 30°C. Three variations are under test. Infiltration in dug wells with gravel filter, infiltration in left shallow wells, and direct injection in deep wells. In some areas experimental rainwater harvesting takes place. The behavior of the temperature field and both aquifers is represented in a numerical model.

Keywords: Caldas Novas, fractured aquifer, groundwater head stabilization, reuse, thermal water

INTRODUCTION

In the east of the Serra de Caldas (Fig. 1) in the state of Goias, Brazil, were found initially thermal springs, where the city of Caldas Novas is located. West of the Serra de Caldas still occur at the Pousada do Rio Quente (Caldas Velhas) multiple springs in a limited area which, together discharge about 1.5 m^3 / s forming the Rio Quente. In the east there are now only the springs at the Lagoa Pirapitinga with a total discharge of about 12 L / s. All springs have dried up in Caldas Novas, after wells were drilled and the water table was drawn down. After more and more wells had caused an extreme decrease of the water table, the competent authority (Departemento Nacional da Produção Mineral - DNPM)) required a maximum annual withdrawal amount, which is monitored monthly. Since the fluctuations in groundwater levels are still very high and the supply amount approved is not extracted yet, waters from the swimming pools will be used to recharge the aquifer in order to achieve a stabilization of the water table on a level of 655 m.

Fig. 1:Localization of Caldas Novas

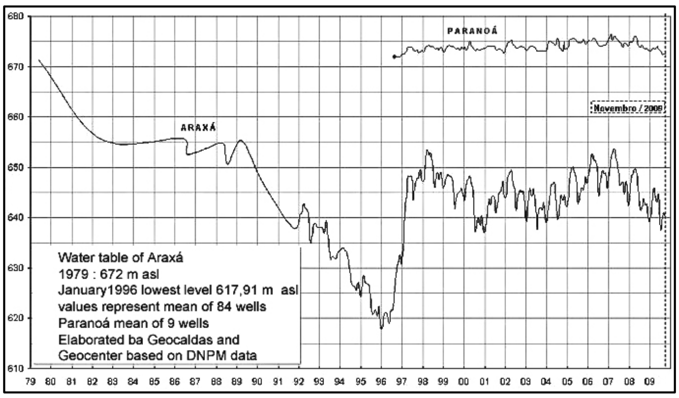

Fig. 2: development of the water tables in Caldas Novas

The development of the water table is shown in Figure 2. However, there remains a problem that affects not the water table at the moment. The DNPM has mining rights and thus granted drilling permits and the permit for thermal water abstraction, but there is a much bigger quantity approved, as is currently being pumped from the wells. Looking at the groundwater table behavior and taking in account an increase of abstraction, it will not be possible to represent a sustainable management of the aquifer. The economy of Caldas Novas, however, depends entirely on the operation of swimming pools, which are supplied with thermal water. New apartments and hotels will be added continuously and the city's population is growing strongly. The population was increasing sharply from about 10,000 residents in 1990 to 70000 in 2010. 90% of the jobs depend on the thermal water which means tourism.

The DNPM issues no new permits for well drilling, until the groundwater conditions are better known and sustainable management can be demonstrated. For this reason, the idea was born to reuse the water after use in the swimming pools, with different artificial recharge methods.

SOME BASICS ABOUT THE LEGAL ASPECTS OF THERMAL WATER IN BRAZIL

In Brazil two different authorities are responsible for the extraction of groundwater. For drinking, agricultural and industrial use of groundwater the states are the relevant authorities. If the water is bottled or is it warmer than 27°C, is the licensing authority, the DNPM, this is a federal agency. It is considered to hold its value as a mineral that also delineated "mines" has. According to one acquires mineral rights and the well owners are mining entrepreneur. As usual in any other mining operations levies are paid to the mining authority, which depend on the abstracted amount taken from thermal water. DNPM makes no difference in which kind the water is used. Even if the water is not bottled, but as in the case of Caldas Novas is only used for bathing purposes, the same rules are applied. This means the chemical and bacterial limits are applied for bottled drinking water under all circumstances.

GEOLOGY

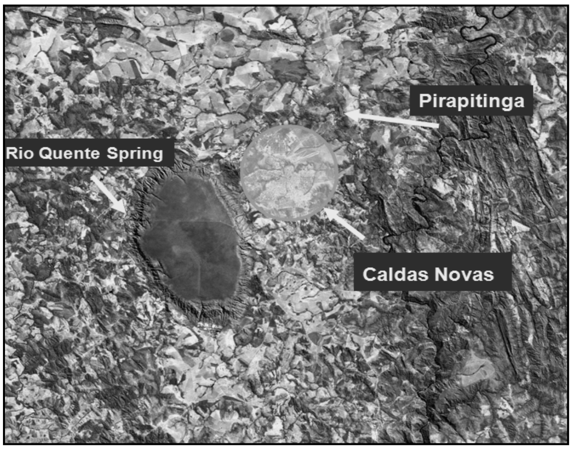

Fig. 3: Satellite image of the Serra de Caldas center, Caldas Novas and springs

Caldas Novas (Goias) is situated on a plateau west of the Serra de Caldas, a monolithically outstanding plateau, a 300 m elevated oval dome morphology (fig. 3).Two stratigraphic units of Upper Meso-Proterozoic age built up the region, Paranoá and Araxá. Paranoá is the older unit that consists of thick to thin plated banked quartzites. These rocks form the Serra de Caldas. Drake (1985) describes it as the “Serra de Caldas window”. The youngest deposits of this unit are marbles, which were deposited in lenses and don’t cover the quartzites everywhere. The overlying layers consist of an alternating shales and thin layers of quartzite. They belong to the Araxá unit and are folded strong. The youngest sediments are from the Cretaceous or Tertiary and occur on the Serra de Caldas.

TECTONIC AND STRUCTURE

The Serra de Caldas is a dome, which is constructed from deformed metamorphic sedimentary rocks. The sediments were deposited in the upper middle Proterozoic and during the Brasiliano-phase tectonic deformation (D’elRey et al., 2004;2008) could be detected as a depression in the basement of the multiple size of the dome (Lugão, P.P. et al. 2000), there was a special structure in the quartzite that led to a piling, and the strata were maintained in their horizontal position (fig. 5).

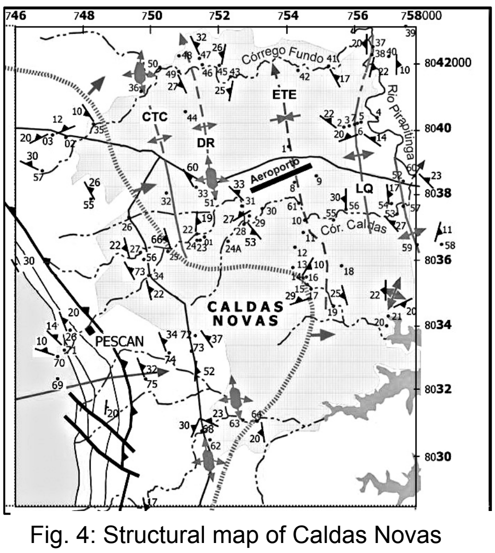

Fig. 4: Structural map of Caldas Novas

The younger mudstones have been slated and strongly folded. The thin quartzite layers were also folded with, but reacted much more brittle and show open fractures. After the Brasiliano faulting phase was followed by further compressive phases, which were not of the same magnitude. However, the restraint caused the north and south of the oval shape and the rotating strike. Big fractures and deep faults are not visible at the surface but are indicated by the morphology. They are of old origin. Lineations at the edges of the Serra de Caldas can be seen very well in the satellite image (fig. 3).

The adopted duplex structure divides along the internal folds the Serra de Caldas. The impact of deep structures can be observed on small folds as shown in Figure 4. In Figures 4 and 6 small morphologic bulges can linked to Paranoá folding in the underground. In these structures may also be correspondingly large fractures emerged in the deep. The areal distribution of the thermal waters suggests that it might be the pass way.

HYDROGEOLOGY

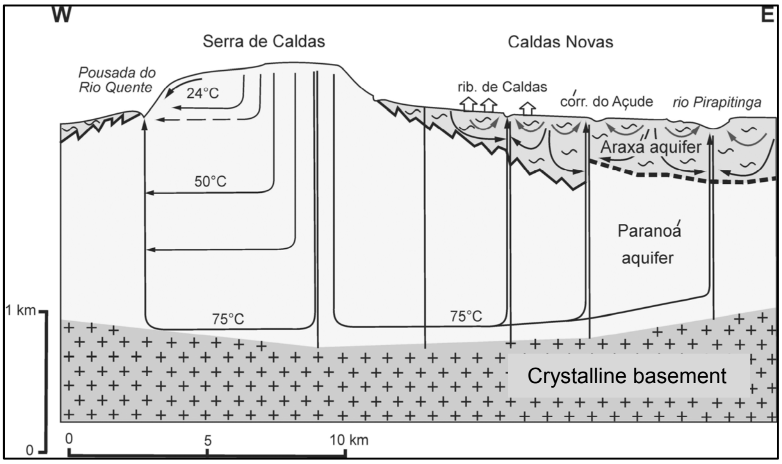

Fig. 5: scheme of thermal groundwater flow in the Serra de Caldas area

Two aquifers can be distinguished (figure 5): Araxá as an unconfined aquifer and Paranoá as semi-confined thermal aquifer. The hydraulic conductivities are at 10^{-5} - 10^{-7} m/s. Pumping tests show first the emptying of the well and then a delayed strong flow, which can be interpreted by the delayed response of large cavities. The groundwater recharge of the thermal groundwater takes place in the Serra de Caldas which could be proven by hydrochemical signatures (Tröger et al., 2004, Tröger 2007). The thermal waters from the Paranoá aquifer have a pH around 6, and low desolved mineral content. The water, which is accessed directly from that aquifer has a temperature of 59°C. The Araxá aquifer has a lower groundwater recharge, but is in the area of Caldas Novas important as the water mixes with the hot uprising groundwater from the Paranoá aquifer which enables the draw off from this aquifer. The mixing water is characterized by a pH of 7.7 and higher mineralization up to 150 µS. The temperature varies between 38°C and 48°C, where the geothermal gradient of 6°C to 15°C is less. The groundwater of the unaffected Araxá aquifer also has a neutral pH is much less mineralized than the mixed water and has temperatures of up to 33°C.

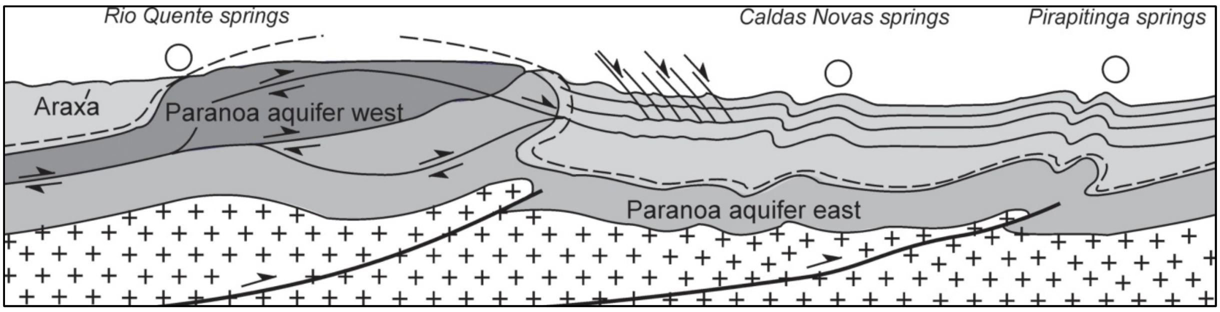

Fig. 6: W – E cross section of Rio Quente springs – Serra de Caldas dome – Caldas Novas – Pirapitinga showing the duplex structure and the division of the aquifer. The springs occur in consequence of deep faults

Araxá and Paranoá groups are extensively exposed in central Brazil. A special situation has arising in the area of the Serra de Caldas, which is due to the underground structure and the associated formation of the duplex structure. Primary fractures were created and these extend to a depth of 2500m. Groundwater must reach that depth which is necessary to heat it to about 76 ° C. Outside of this structure the total thickness of the Paranoá group is probably less than a quarter(?) and the basement rocks are reached in less depth. Moreover, there seem to be any major fractures, so that no deep groundwater circulation is adopted.

Recent findings have shown that the Serra de Caldas must be divided by a compressive fault (figure 6) in two independent aquifers east and west. 90% of groundwater recharge flow to the springs of the Rio Quente. The reservoir is smaller and the hydrograph of the Rio Quente has a variation form

900L/s to 1800 L/s. The groundwater reservoir, that is related to the Caldas Novas thermal water, is significantly greater and the variation of the groundwater table is relatively small (fig.2).

If one evaluates camera logs from the well, one notes that the fractures widen in the depth which is quartzite solution phenomena known from the quartzite karst of Venezuela. The recordings also show that the groundwater flows very quickly through the fractures, which can be observed in small particles coming by fast without pumping the well. The good passage for in deeper layers of the Araxá group is still not sufficient to explain the temperature of thermal water. The thermal water has its origin in the Paranoá group, which apparently only rises up from the semi-confined aquifer in some extension faults, which are up to exemptions under the city situated. The karstification of quartzite banks and marble lenses is tied to tectonic structures. On the flanks of the Serra de Caldas, the karstification is also observed. It can be inferred that before the erosion of the layers over the Serra de Caldas has reached its present state, the warm groundwater has solved quartzite. For the corrosion of the quartzites are the ancient structures of great importance.

DEVELOPMENT OF GROUNDWATER CONDITIONS

After the first well had started to groundwater extraction, there was a significant draw down of the water table and the hot springs dried up in the city of Caldas Novas. With the successful development of tourism increased the demand of thermal water for over 100 swimming pools. The thermal water abstraction reaches in the mean 210 L/s or 756 m^3/h although about the double is licensed. The thermal water abstraction exceeded very quickly the groundwater recharge and with a government intervention the construction of new wells was forbidden and the abstraction of existing wells was limited. At the same time the city grew and more and more areas were sealed (Fig. 7). Thus, the groundwater recharge of the Araxá aquifer was reduced. In years with low natural recharge the hydrograph shows a strong negative tendency, with not even the approved quantities are taken (fig. 2). In the future it is to expect that more wells will be drilled to pump the thermal water and the water table falls strongly again. Because of the licensed amount of thermal water extraction DNPM is not able to regulate the total draw off.

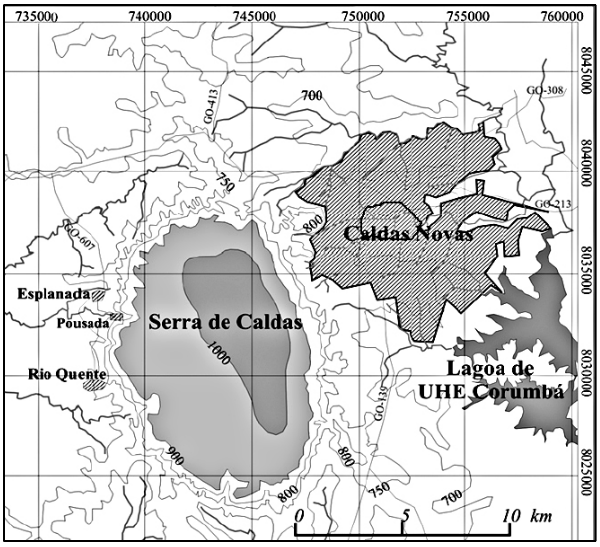

Fig. 7: The increasing from the old city core (inner circle) to today's extension of Caldas Novas

Fig. 8: Rainwater infiltration trench, left filled with gravel, right a plastic foil covers the gravel and the original soil is refiled

USE OF RAINWATER

A major emergency was used to compensate for the lost groundwater recharge through surface sealing. Figure 7 shows the spreading of the city during the last two decades. The reduction of non sealed areas will keep on as apartment buildings are in execution. The roofs of newly built houses are tentatively fitted with rain gutters and the water over slit trenches the surface. The design of the slit trenches is very simple. A 4 m deep and 20 m long trench was excavated and filled with gravel to a depth of 2 meters. Then the gravel was covered with foil to prevent the occurrence of clays. The divisions are supplied by the long side, so that air can escape. As the precipitation 1500 mm / year is, so about 1200 mm can be enriched by the roof surfaces. However, only a few roofs and use a portion of the enriched water is lost as interflow.

REUSE

The large variation of the water table, which is due to insufficient formation (recharge?), caused the mine owner, a project for artificial recharge to be initiated. The aim was to investigate the thermal water after it had been used in the swimming pools for artificial recharge. Several measures were planned and analyzed in pilot studies. Three different methods should address. The recharge of the used pool water in sink wells, a process where no treatment of the water is needed. The second method was the direct recharge of the used water carried out on old wells with good hydraulic conductivity and, finally, a theoretical study should be studied to recharge in the deep Paranoá aquifer. By now the newly created laws that study, however, was not very useful, as the mixture of groundwater from different aquifers is not allowed.

ARTIFICIAL RECHARGE WITH THE RUN OFF FROM SWMMING POOLS

From the more than 100 swimming pools permanent runoff water is available. The thermal water is led from the well directly or through cascade water fountains in the pools. Since the water is not chlorinated, it has to be changed after a short residence time and is again discharged in almost all cases into the creeks and small rivers. The water is lightly contaminated with ammonium and nitrate. Thus, also slight increase in chloride content is measured, as in the warm climate, the evaporation of salts of the bathers are rinsed in pool water. Furthermore, appears a higher ammonium concentration, which is due to urine. The values reach a maximum of 2.1 ppm. The values for nitrate are usually only slightly higher than those of thermal water. This value reaches, however, in some wells over10 ppm.

An important legal aspect must be respected when artificial groundwater recharge takes place. Under the Brazilian law may in groundwater, the value of 0.1 ppm of ammonium not be exceeded. Therefore, attempts to convert ammonium into nitrate. The limit for nitrate is 10 ppm, so that here are the procedure boundaries. However, the nitrate is probably registered geogenically and the limit should be seen as unrealistic, because it is far below the WHO listed limited values. Nitrate could not be analyzed in the thermal water from the Paranoá aquifer.

Two different methods of artificial recharge were elected for the water from the pools. In the first method, the water from the pools flows through a sedimentation tank and over a fat separator. The water is then fed into a shaft, which is filled with a gravel filter with 6 m depth. The grain size is 1 to 4 cm. The shaft is secured at the top with concrete rings, which are two meters into the ground (fig.8).

Fig. 8: sink well with gravel and observation pipe

The monitoring and setting of the maximum amount of infiltration was installed at each pilot sink well an observation pipe. The inflow was measured with water meters. An observation well was drilled in the direction of the groundwater flow. Two pilot plants have been provided, which were on different soils, which have been tested with infiltration measurements. Another criterion for the location selection was the water quality of the pool with a fully occupied hotel at the end of the year and during carnival. Since these are public holidays in the summer, it could be assumed that the urine contamination was greatest. Both locations had a total N-load of less than 1 ppm.

Since for every pilot, only two sink wells were available, there was already a priori limit set for the total. In the most permeable soil that can be mapped in Caldas Novas, infiltration capacity from 10 m^3/h per well have been achieved. For this performance of the sink well it is filled with gravel up to the top. In the measuring well downstream a rise in groundwater levels was observed. As the financial burden for the production of these sink wells is quite low, more of these facilities were recommended to build in favorable positions. A thermal change is by this measure not expected, since the infiltration takes the path of natural groundwater recharge and will heated up by the geothermal gradient.

DIRECT ARTIFICIAL RECHARGE

The pilot study for the direct groundwater recharge was carried out on old wells that are no longer in operation. To minimize the thermal impact, one well was selected where the temperature at 35°C was only slightly above the water from the swimming pools. The water is drained into a large reservoir for sediment deposition. There is also a fat separator. To comply with the Brazilian laws, the water was passed through an aeration pool in a reaction tank. Up to 0.5 ppm of ammonia could be oxidized to nitrate. Then the water flowed through a sand filter, followed by a zeolithe bed and an activated carbon filter. Then the water passed an UV - disinfection and finally was infiltrated by gravity in the well (fig. 9).

Fig. 9: Tank with sand, zeolithe, and activated carbon, left UV desinfection

The pilot plant was designed for cost reasons, for a maximum flow of $17m^3/h$. The test of the well showed an infiltration capacity 50 m^3/h. in a three-day trial, such amount of used water would be only available from several pools. A large quantity of artificial recharge would probably also bring a large specific thermal burden. Furthermore, there is no need of very much water for recharge to rise up the water table to the 650 m level.

NUMERICAL MODEL

The result is actually a three-dimensional numerical groundwater model, whereby the thermal currents are to be reviewed. Also should be set for the recharge well by the model, the locations and infiltration quantities. The model is continually evolving, as in Caldas Novas substitute drillings of new wells, refer to the thermal water from the Paranoá aquifer. This may cause a slightly change of the pressure conditions and the thermal influence in the Araxá aquifer.

CONCLUSION

A stabilization of the thermal ground water table in Araxá aquifer seems possible. The pilot study has shown that various appropriate measures can be taken, leading to an overall positive result. There are, however, necessarily long-term measurements to carry out before a final determination of useful methods can be given.

The study was realized only with funds from the AMAT.

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Effectiveness of Underground Dams in Shandong Peninsula, China

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Abstract: Six underground dams have been built to address seawater intrusion problems by increasing groundwater storage in Shandong Peninsula. This paper analyses the characteristics of six dam sites in Shandong Peninsula, including topography, geology, hydrology, hydrogeology, main groundwater source types and distribution, as well as their effectiveness and techniques that have been used to enhance recharge due to accumulation of clay and silt upstream of dams. Site selection principles and construction technology in building complete bidirectional cut-off walls are discussed. Comparing observational data before and after the construction of a cut-off wall, the changes in groundwater recharge and discharge as well as changes in groundwater quality were analysed, and a numerical model was calibrated to simulate the observed effects and help inform management of the recharge enhancement. Finally, social, environmental and economic benefits of the construction of underground dams were evaluated, problems of underground barrier were summed up; and measures of protecting groundwater sources of supplies and improving groundwater quality in the reservoir area were proposed.

Key words: coastal plain; underground reservoir; managed aquifer recharge

INTRODUCTION

Underground dams can be effective in preventing seawater intrusion and have been widely employed in coastal areas where seawater intrusion is a serious issue. Japan is comparatively advanced in building underground reservoirs with the technology of cut-off wall type. Several underground reservoirs were built in the 1970s to 1980s (SWCRI,1989; Satoshi Ishida, *et al.*,2003; Japan Green Resources Agency,2004) and have been effective in preventing sea water intrusion, but also improving ground water levels.

Shandong Peninsula is surrounded by the sea from three directions, with the topography consists of low mountains and hilly land. The lithology is mainly granite and gneiss. water system originates inland and directly discharges into the ocean, with characteristics of short source and swift current.

Groundwater is a major source of urban water supply and occurs in alluvial material formed in streams and river valleys and on both sides of rivers which are underlain by impermeable or low permeability granite. The aquifer is mainly coarse sand, gravel, etc., to a thickness less than 20m, but there are low permeability sections present. In the estuary of most rivers, while the thickness of Quaternary sediments is less than 50m, pore water exists as phreatic water, and can be formed as confined water downstream of larger rivers. After the 1980s, successive droughts occurred in Shandong Peninsula and groundwater became overexploited, leading to seawater intrusion in some coastal areas, and currently the area affected by seawater intrusion is estimated at 1000km^2.

To address water shortage in coastal plain areas and seawater intrusion six underground reservoirs were constructed(Figure 1). Based on investigation and observation, the methods of building underground reservoirs to regulate and store groundwater in riverways located in piedmont alluvial plain in hard rock areas are deeply researched and summarized in this paper.

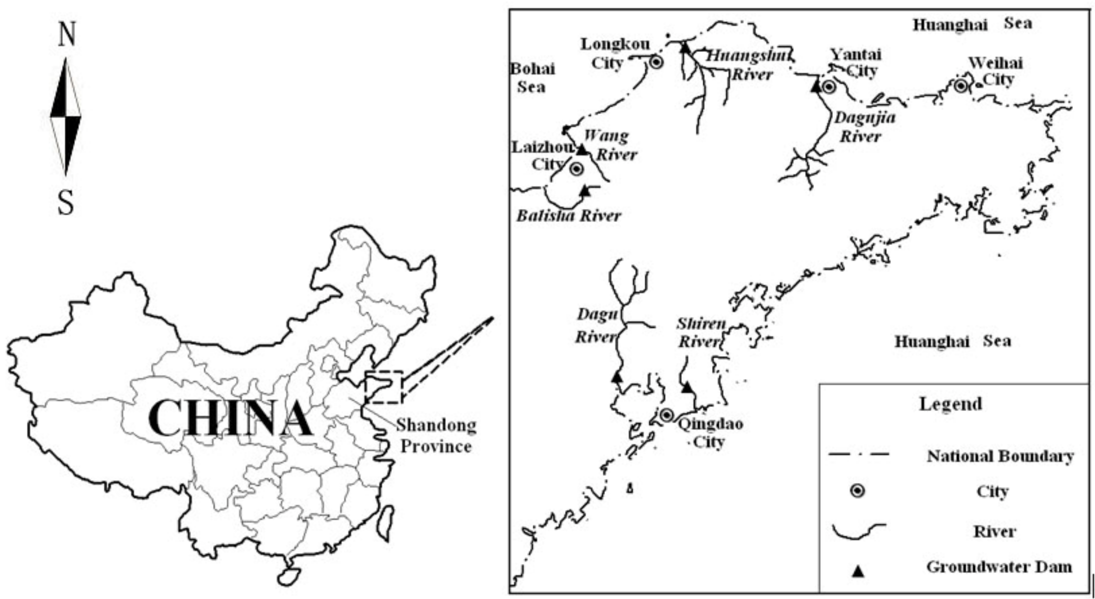

Figure 1 Distribution map of underground reservoirs in Shandong Peninsula

STRUCTURE OF UNDERGROUND RESERVOIR SYSTEM

According to geographical, geological and hydrogeological characteristics as well as groundwater exploitation conditions in Shandong Peninsula, construction of ground water reservoir in coastal areas is a systematic project, involving the construction of underground cut-off wall, water pollution control project, surface impoundment project, aquifer recharge project, water supply project, groundwater monitoring, etc.

The principle of site selection

Site selection for groundwater reservoir with cut-off dam should meet four basic conditions: available aquifer; abundant surface water and recharge catchment areas; no leakage in the dam site and in the groundwater reservoir area; groundwater pumping project which can meet the demand of intended water use. As for underground reservoirs supply for drinking water supply, surface water quality should meet certain quality requirements.

Construction techniques for cut-off wall

There are many technologies for construction of underground dam, such as the casing pile-forming method, vertical spread membrane method, concrete cut-off wall and high pressure jet grouting. In Shandong Peninsula with a deep overburden layer, low head high pressure jet grouting is adopted, and the reservoirs are fully enclosed two-way intercepted underground dams, with the role of preventing seepage from two-ways and avoiding seawater intrusion. When the permeability coefficient of low head cut-off wall is less than 10^{-5}cm/s, the cut-off wall is very effective in preventing seepage and seawater intrusion (LI Daozhen, *et al.,*1997; LIU zhenfan, *et al.*, 2003; HAO Zhuqing, *et al.*, 2005). Tests on Balisha River underground reservoir showed that the average thickness of underground walls built by jet grouting is 40cm, and the average permeability is up to 6.9×10^{-6}cm/s, the average head difference between the upstream and downstream of 2 m, but ranging from 1.8 to 5m.

Surface impoundment and aquifer recharge projects

The average annual rainfall for the Shandong Peninsula is 720mm, concentrated from July to September, and rivers are seasonal. Taking into account the amount of water impounded by river runoff project, 45% of the mean annual surface runoff flows into the sea, which provides water for underground reservoirs. Take Huangshui River as an example, the annual average surface runoff is 226 million m^3, of which 128 million m^3 is impounded by an upstream project, and runoff flowing into the sea is 0.98 million m^3. Natural flood time is 3-5 days, 8-10 days, 12-15 days, 15-20 days at frequencies of 95%, 75%, 50% and 20%, respectively. Aquifer recharge by river run-off is limited, so a sluice gate is needed to extend infiltration time. Two sluice gates (flap gate) were built in Huangshui River underground reservoir area, and a rubber dam was built on underground dam at estuary, all of which can hold water and block tide. In addition, aquifers in parts of the river channels located in Shandong Peninsula are two-layer structure, with a low permeability layer leading to poor recharge through leakage of the riverbed. Therefore surface water storage (sluice) and aquifer recharge project (seepage wells, seepage basins, seepage drainage) are both needed to make use of excess flood effectively, and improve recharge efficiency. By building sluice to store water, the duration of recharge can be extended by as much as five times (Liu Zhenfan, *et al.*, 2003), so that recharge project plays its role more effectively. A skylight by the percolation well which penetrates the upper sub-claypan into the lower sub-sand aquifer, has increased the amount and rate of recharge and conversion of excess flood water to groundwater. To combat the 3-17m sub-clay layer with low permeability distributed on about 5.8km of river surface in Huangshui River groundwater reservoir, 4400 dug seepage wells and 600 machine seepage wells were built in the riverways in this reservoir area (Daozhen Li, *et al.,*1997).

Underground reservoir indices

Net drainage area, storage capacity, recharge volume, as well as the layout of the exploitation well controlled by underground reservoir is decided by the extraction of the groundwater reservoirs,. Main technical specifications (Wang Weiping, *et al.*, 2010) concerning 6 underground reservoirs are shown in Table 1.

Table 1 Main technical indices concerning underground reservoirs in Shandong Peninsula

Reservoir	TSC	DMAC	CA (km^2)	NCA (km^2)	RA (km^2)	LUD (m)	ADD(m)	WS (10^4 m^3)	CT
Balisha River	42.9	35	14.7	8.8	14	756	8.5	1.699	1988
Huangshui River	5359	3852	1015.7	102.9	51	5842	10	4.000	1992
Shiren River	130	120	20.85	20.85	21	620	17	0.100	1994
Dagujia River	20520	6500	2296	1456	65	3564	14.9	15.000	1998
Wanghe River	5693	2080	326.8	173.4	68	14500	10	5.416	2005
Dagu River	9830	3100	4161	2239	421	4248	7	10.000	1998

TSC: Total storage capacity(10^4 m^3); DMAC: Designed Maximum Active Capacity(10^4 m^3); CA: Catchment Area; NCA: Net Catchment area; RA: Reservoir Area; LUD: Length of Underground Dam; ADD: Average Depth of Dam; WS: Water Supply; CT: Completion Time.

Water pollution control projects and water conservation

Aside from the impacts of seawater intrusion, heavy population density, relatively developed industry and agriculture in the plain area all have resulted in partial groundwater pollution. After the completion of underground reservoirs, groundwater quality has become a very sensitive issue. Therefore, it is necessary to prevent pollution of the rivers and also to prevent groundwater pollution caused by wastewaters and effective measures must be taken to protect groundwater from pollution. As a results the 8187m of sewage pipes were completed in the Huangshui River underground reservoir to discharge treated effluent directly into the sea.

Management organization and groundwater monitoring project

Monitoring system should be established and improved, monitoring dynamic conditions including groundwater table and quality, etc. inside and outside the reservoir area. A detailed groundwater monitoring system has been established in Huangshui River underground reservoir and Wanghe River underground reservoir, in addition to management division.

CALCULATION AND ANALYSIS OF WATER RESOURCE

For example, the numerical model simulation of the Wanghe River underground reservoir was built to analyze the water balances under three alternatives after completion of groundwater dam over a timescale of 31 years. Scheme III add extraction by 15 thousand m^3/d of Yinjia water source on basis of scheme II. The reservoir aquifer is mainly caused by Quaternary alluvial flood accumulation, and the main lithology is gravelly coarse sand, distributed in 3 and 4 layers in space with total thickness of 5.0 ~ 15.0m, from east to west, thickness gradually becomes bigger, up to the maximum embedded depth of 23.7m. The aquifer permeability is 16.9 ~ 89.3 m/d and water permeability and water content are fair. Each water layer is regarded as homogeneous and identical and the hydraulic connection is tight, which is a benefit for mutual recharge between sand layers and conversion from surface water to groundwater. According to analysis of the boundary conditions, water storage structures and groundwater flow field for many years in reservoir area, this underground reservoir can be generalized as 3-dimensional phreatic unsteady flow.

Scheme III is the best engineering solutions from calculation and analysis because water balance when maximum groundwater excavation is achieved. Design of underground reservoir should be based on this method.

CHANGE OF WATER QUANTITY AND QUALITY PRE AND POST CONSTRUCTION OF GROUNDWATER RESERVOIR

(1) Construction of underground dams has changed surface runoff and discharge of groundwater in reservoir areas. Before construction of Balisha River underground reservoir, there was little runoff flow out of the reservoir in normal or wet years and no runoff flow out of the reservoir in dry years because surface water retained upstream and groundwater exploitation downstream. After construction of the dam, upstream water turns from free flow into subsurface flow and downward discharge in base flow occurs after entering the reservoir, regulated by the groundwater dam. So the phenomena named stored-full runoff happened. The water table was raised because of impoundment and regulation of the underground dam and it increased rainfall infiltration coefficient, extended time of base flow, and the phenomena that total annual runoff of base flow is more than total annual surface runoff appeared. This fully shows that discharge of reservoir area is a result of regulation of underground dam.

(2) Underground dams have blocked lateral recharge, raised water table, expanded water supply and reduced discharge. Under the conditions of mining - supply, re-mining - re-supply in Balisha River underground reservoir, a re-storage index between 1.6-2 resulted from impoundment and regulation and the total amount of storage and impoundment caused by dam is greater than twice that of the recharge in the reservoir area before building the dam.

(3) After construction of the dam, conditions of runoff yield and concentration in reservoir area have changed. According groundwater level observations, water recharge at upstream and downstream of reservoir area was concentrated in winter after the flood season when exploitation is less. After construction of the dam, due to the water table elevation and increased rainfall infiltration recharge, groundwater recharge in the reservoir area has focused on flood season or ahead. From dynamic curve for downstream of Huangshui River from 1976 to 1987, its peak value is 48 days later than the peak of precipitation before installation of the dam. After the construction of the dam, due to the benefit of the aquifer recharge project, infiltration has increased, which raise groundwater from low level to peak in advance and then have eased tension of water use, its meaning is more than increase of water resources. Also, owing to water table elevation in reservoir area, runoff coefficient and surface runoff in flood season both have increased. This made annual groundwater lateral recharge in the region after the dam decreased in large extent, while surface

runoff of river increased in flood season and winter.

(4) Field data from Wanghe River underground reservoir which started in 1999 shows that after construction of the underground dam, average annual groundwater level increased by 4.44 m with an average elevation of 3.31 m, shown in Table 2; water level difference was measured by piezometric tube wells, the results show that water table inside dam is higher than outside dam with maximum of 5.05 m and average of 4.39 m. Monitoring of groundwater table in Huangshui River underground reservoir indicates that, comparing with the same time, water table in centre of overexploitation funnel has raised by 2.6 m under the artificial recharge project.

Table 2 Groundwater level in Wanghe River underground reservoir before and after the reservoir building unit: m

Location	1997	1998	1999	2000	2001	2002	2003	2004
Zhufeng	9.26	9.39	9.07	9.26	12.2	10.99	11.4	12.4
Guoxi	-1.56	-1.55	-1.61	-1.56	1.03	1.56	1.87	2.87
Shabu Village	-5.11	-5.20	-5.41	-5.11	-1.41	-5.41	-4.90	-2.9
Average	0.86	0.88	0.65	0.86	3.95	2.38	2.79	4.12

There are surface impoundment and artificial recharge projects in both Wanghe River and Huangshui River underground reservoirs, and the net catchment area of groundwater is large. While in Balisha River underground reservoir, compared within the same period, water table inside dam is higher than outside dam, but in dry season or irrigational season of wet years, water level outside dam is higher, each situation explains that underground dams have good water holding effects.

(5) After completion of underground dam, significant changes occurred in water quality inside and outside underground reservoir area. After Wanghe River underground reservoir has been completed, sea water intrusion area in Wanghe River basin reduced from 78.69 km^2 to 25.36 km^2, a decrease of 68%. Water supply capacity has been increased and water quality has been improved. Through samples collected from water table observation wells in reservoir area, it is found that after the dam, the concentration of chloride ion in groundwater decreased 70.1% at most, with average of 50.6% (HAO Zhuqing, *et al.*, 2005) (Table 3). Backward the dam, seawater intrusion is more serious, chloride is more than before the dam was built.

Table3 Average concentration of Cl^- before and after construction of Wanghe River underground reservoir Unit: mg / L

Location	1999	2003	2004	Relative reduced rate %
Shabu Village	468	290	253	45.9
Guoxi	159.2	145	95	40.3
Yinjia	185	93.4	53.4	71.1
Average	270	176.1	133.8	50.6

After construction of Huangshui River underground reservoir, interface of seawater intrusion backed off by about 30m.

EFFECTNESS AND ISSUES

Except for expanding direct economic benefits of water supply, important social and environmental benefits are also included in the effects of underground reservoir in coastal area, especially the last two benefits play important roles. Downstream of Wanghe River, Huangshui River, Balisha River, Dagujia River and Dagu River, due to the blind overexploitation of groundwater, sea water intrusion occurred leading to significant losses; resulting in retirement of original industrial and agricultural water supply facilities, barren fertile land, an increase in the number of people escaping water shortages in arid years, shutdown and even relocation of companies, and domestic water problems caused by lack of water. In order to alleviate the disaster, water diversion projects needing great investment have been built to cope with life and partial industrial water supply, but the problem of sufficient water for agricultural irrigation is still not solved. In addition to solving problems of water supply for industry, living and agriculture in seawater intrusion area, the more important benefits of construction of underground reservoirs is to effectively restrain seawater intrusion, improve water quality and the surrounding environment and to protect large blocks of farmland from contamination, which are all very significant.

After the completion of underground reservoirs, environmental geological conditions have changed a lot. Firstly, surface runoff is stored or used after turning into groundwater, secondly, most of the subsurface flow discharging to downstream is intercepted by underground dam, partial original salt water is impounded too, groundwater pre and post the dam changes from natural two-way exchange into one-way discharge in small quantity. As a result, the original groundwater flow field and chemical field are changed, and the spatial and temporal distribution of groundwater resources is changed too. Updating the water body in the reservoir area relies on extracting groundwater in reservoir and supply for outside. In addition, due to artificial recharge project, an impermeable stratum of unsaturated zone is penetrated, which may reduce the self-purification capacity of surface water in the process of infiltration. The remaining problems are salt water retention, groundwater protection in the reservoir area, water quality monitoring, operational management of artificial recharge project and water quality deterioration at upstream and downstream of partial underground dam, which need to be solved in the future.

CONCLUSION

Survey of construction, long-term operation and effectiveness for six underground reservoirs located in the coastal region of Shandong Peninsula in China shows that, underground reservoirs constructed in riverway of piedmont alluvial plain in hard rock areas can impound and regulate groundwater, increase water supply and prevent seawater intrusion, have made enormous social, environmental and economic benefits. Compared to surface reservoirs with the same scale built in mountainous area in this region, these underground reservoirs have characteristics of no immigration, no land occupation, few evaporation leakage, low investment and water supply cost as well as good water quality. Underground reservoir is an integrated engineering, there are both advantages and disadvantages in building underground reservoir in riverway of coastal area in

Shandong Peninsula, but the disadvantage can be offset by engineering measures.

To ensure the effectiveness concerning underground reservoir, pollution control measures for surface water and protection of groundwater in reservoir area should be enhanced to prevent water quality deterioration. Monitoring systems for surface water and groundwater should be established and improved to monitor dynamic of water quantity and quality (including river flux, water quality; groundwater table and quality within reservoir as well as inside and outside of the dam) in long-term. Management of underground reservoir should be enhanced, and maintenance of recharge projects such as seepage wells and percolation trenches must be done correctly. In the basis of monitoring, it is important to establish a management model of underground reservoir, optimize exploitation layout in reservoir area and improve utilization efficiency of the underground reservoir. Effective measures should be taken to replace salt water inside of the dam.

In short, through assessment on effectiveness of six underground reservoirs located in Shandong Peninsula, it is a wise choice to build underground reservoirs for areas with the shortage of water or coastal seawater intrusion where general conditions of building underground reservoir is ready.

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Reclaimed ASR – Assessment of Potential Storage Zones in the Middle East

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ABSTRACT

Reclaimed water use is becoming a critical element of water resources management in the Middle East. As the use of reclaimed water grows in this water-poor region, storage will become a critical need to ensure consistent delivery to meet demands of paying customers. Aquifer storage and recovery (ASR) can potentially be implemented now to stop the current loss of this precious resource by storing treated effluent for recovery in the future, even when no current demand has been established.

A detailed assessment was performed of potential subsurface storage zones within the Gulf Cooperation Council (GCC) countries. A screening process for potential reclaimed ASR systems within the Middle East was developed that includes considerations of aquifer hydraulic properties and hydrochemistry, existing aquifer water use, quality and consistency of treated sewage effluent supply, and distribution to demand centers. The hydrostratigraphy underlying several major municipal areas in the GCC countries was evaluated with respect to potential subsurface storage zones. The seasonality of supply versus demands in this region were also explored.

An ideal reclaimed water subsurface storage zone would be located at the treatment facility, result in excellent recovery efficiencies, and have no potential for drinking water aquifers to become impacted. However, such an ideal storage zone may be difficult to find. Therefore, potential scenarios utilizing non-ideal storage zones are explored.

KEY WORDS

Middle East, GCC, reclaimed water, treated sewage effluent, wastewater effluent, wastewater reuse, ASR, MAR, storage zone, recovery efficiency.

INTRODUCTION

Reclaimed Water Use in the Middle East

The Middle East, in particular the Gulf Cooperation Council (GCC) nations on the Arabian Peninsula, is a water-poor region that has experienced rapid growth of population and, therefore, also a rapid rise in water demands in recent years. For oil-rich nations, water demands, particularly for potable use, are being addressed to a large degree by seawater desalination. However, the water supply in the GCC nations also has significant components from other sources, including pumping of fossil groundwater from deep aquifers in the Kingdom of Saudi Arabia. Recently, reclaimed water or treated sewage effluent (TSE) has come under increasing scrutiny as a critical component for future water supply in this region.

Reclaimed water has been used for landscape irrigation in cities such as Abu Dhabi, Dubai, Doha, and Riyadh. Some other uses, such as date farm irrigation in Saudi Arabia, have been successfully implemented. The use of reclaimed water has been found to be consistent with Sharia law, and a

Fatwa (religious proclamation) was issued in 1978 stating that reclaimed water can be used for ablution and drinking provided that it is sufficiently and appropriately treated to ensure good health. One area that has currently seeing much attention in the region is construction of new tertiary wastewater treatment facilities, since there are still many cities that do not have adequate wastewater treatment facilities. An example of the severity of this concern is found in the City of Jeddah, where over a million people use septic tanks, from which wastewater has been carried away daily via truck east of the City to Musk Lake. Flooding in Jeddah in 2009 raised the level of concern for Musk Lake, and continued discharging of wastewater to this lake was banned in 2010. Many billions of dollars in infrastructure improvements have been allocated, and construction of new treatment plants is underway. However, getting wastewater service in place to all within this municipality will take time.

In cities where wastewater treatment is in operation, a great amount of the treated effluent has been allowed to discharge to wadis or to form surface water bodies. In some cases, the discharge effluent or a portion of the effluent is untreated, due to influent flows exceeding treatment capacity. The discharged water therefore causes significant concerns with regard to impacts to groundwater and public health concerns ranging from mosquitoes to infectious diseases. An increased incidence of West Nile virus in Jeddah contributed to the decision to ban further discharges to Musk Lake.

Water resources experts in the region agree that the reclaimed water will be a significant part of the long-term water supply solution for the area. In particular, there is a great societal benefit by the replacement of the current use of potable water (especially fossil groundwater) for irrigation with reclaimed water. However, there are obstacles to getting 100% of wastewater being reused, including but not limited to the following:

--Major demand centers (e.g. farms) may be at some distance from areas of wastewater production (cities). Installation of transmission systems is costly and takes time.

--Current water law allows farms to pump non-renewable groundwater with little restriction, although permitting of new wells is being addressed in several countries as a means to limit withdrawals in critical areas. In addition, subsidies for wheat production in the Kingdom of Saudi Arabia were ended, which has resulted in some reduction in farm withdrawals. However, a large part of the non-renewable groundwater use in the Kingdom is still for farms in this large desert nation.

-- As water is currently free, there is little incentive to replace such use with reclaimed water. However, regulatory mechanisms are being developed to address this concern, such as not permitting the construction of new wells in critically impacted areas.

--Reclaimed water users commonly have a reasonable concern regarding consistency and quality of water supply. Potential reclaimed water users are typically unwilling to commit to this water source unless they are confident that will receive the quantity and quality of water needed with the proper timing of delivery., Reliability of supply is critical for current and increased demand,

Reclaimed Aquifer Storage and Recovery (ASR) in the Middle East

Several ASR projects are being implemented in the GCC region; however, these ASR projects pertain primarily to operational and strategic storage of potable water. ASR has been discussed within the region as an important component of reclaimed water use, however, the primary objective of such ASR needs to be clearly defined. Potential reclaimed water ASR system objectives can include the following:

(1) Storage to meet current demands (i.e, operational storage). Excess reclaimed water could be stored during times of excess supply for later use during times when current demands exceeds the available supply. Operational storage of reclaimed water is the goal of several reclaimed water ASR projects in Florida, USA, and Australia under various stages of development. However, no situations have been identified in the GCC region in which a seasonality of reclaimed water supply and demand currently warrants the installation of an ASR system.

(2) Long-term resource conservation. An ASR or Managed Aquifer Recovery (MAR) approach of injecting or otherwise recharging TSE into a suitable aquifer storage zone could be implemented as a long-term resource conservation measure, pending later development of demand by reclaimed water users. Considering the vast amount of treated wastewater that is currently discharged into surface-water bodies or at land surface and lost through evaporation or seepage in unrecoverable shallow recharge, this objective would seem to be a reasonable for potential development.

(3) Water treatment. There has been mention of using ASR for "treatment" of wastewater in the region. Studies have shown that treated wastewater may commonly undergo a further "polishing" effect in the subsurface with reductions of pathogens (e.g. John et al. [1], John and Rose [2] and Toze [3]), disinfection byproducts, (e.g. [4]-[9]), and compounds of emerging concern (e.g. [10]-[22]). These attenuation studies are well summarized in Maliva, et al. [23]. This author, however, would urge extreme caution in looking at the subsurface as a "treatment" facility. Besides uncertainties regarding removal of contaminants, wastewater with an insufficient level of treatment that is injected in the subsurface would mean a greater chance of clogging of the aquifer. Occasionally, injection disposal of wastewater may be needed to protected public health and the environment, but generally, this should be considered primarily a disposal mechanism, and ideally would not be using an aquifer with potential to be used in the future for water supply.

Based on the objectives discussed above, only objective 2, long-term water conservation would appear to have a current basis to proceed with development of reclaimed water ASR in the region. Should a specific periodic deficit of supply to meet demand be identified, then development of ASR to meet objective 1 would be warranted.

ASR to Meet Seasonal Deficit of Supply to Meet Demand

Seasonality of wastewater production generally pertains to variation in populations, commonly associated with migrations (i. e., holiday travel). For example, in the State of Florida, USA, there is a pronounced difference in population between winter and summer due to the presence of numerous "snow birds" (i.e., seasonal senior-citizen residents) who seasonally migrate south in the fall and return north in the spring. In the GCC states, some migration trends include the following:

--Numerous people coming from outside nations to Makkah and Al-Madinah during the Hadj;

--Migration of people to the southwest coastal range of Saudi Arabia (e.g. Taif and Abha) during the hot summer months;

--Locals and expatriates vacationing abroad during school holidays including Eid al-Fitr and Eid al-Adha; and

-European tourisms in all shoreline areas of the UEA, Bahrain, Qatar, and other GCC states.

Seasonality in demand primarily pertains to evapotranspiration and irrigation rates for plants, both landscaping and agricultural uses. For example, dates grown in Tunisia show the minimum monthly water demand in the winter (January) to be 44% less than the maximum summer (August) demand (FAO 2010 [24]). This demand fluctuation could make an opportunity for reclaimed ASR to be used, if summer time supply is insufficient to meet demand. As the migration trends indicate, people leaving major cities in the summer time could further exacerbate the difference between supply and demand. However, there do not appear to be any cases in the region where seasonal reclaimed water use currently exceeds supply. Given the infrastructure expansion needed to get reclaimed water use closer to 100% of supply, this occurrence is not anticipated to be a common occurrence in the near future. However, once reclaimed water distribution systems are in place and annual demand approaches the reclaimed water supply, the need for seasonal storage-based ASR systems will become more evident.

ASR or MAR to Conserve Long-Term Resources

The current volume of treated wastewater in the region that is lost to evaporation or to unrecoverable aquifer zones is tremendous. For example, in Riyadh, where roughly 6 million people use approximately 1,338 trillion cubic meters per day of water in 2010 (Khatib & Alami 2009 [25]), the vast majority of the wastewater generated is currently not reused and discharges to Wadi Hunifa. Given the water-poor nature of this arid region, this loss can potentially be transformed into a potential future resource. Some of these discharges are not entirely wasted per se, such as where farms that are located within wadis (alluvial channels) downstream of discharges capture some of the water. However, in these cases, many of the farm operators are likely under the perception that their water supply is solely from their well water, and not necessarily related to the wastewater treatment plant discharge upstream.

Flow in Wadi Hunifa south of Riyadh
(photo source: Google Earth)

This indirect use of TSE is not particularly efficient. ASR may offer an opportunity to save water for paying customers in the future, provided an appropriate storage zone can be found. It is possible that current discharges and natural recharge could result in a recoverable resource; however unless detailed hydrogeological testing is performed, potential for such recovery cannot be ascertained. It should be noted that efforts are currently ongoing to significantly increase reclaimed water use in the Riyadh area.

POTENTIAL RECLAIMED WATER STORAGE ZONES IN THE MIDDLE EAST

ASR storage zones include physically bounded, and chemically-bounded systems (Maliva, et. al. [23]). The success of potable-water ASR systems generally pertains to the recovery efficiency of the system being sufficient to meet the system specific objectives. For a reclaimed ASR system, this criterion is the same, though recovery efficiencies may be significantly lower than with a potable ASR system and still meet project objectives. However, a second critical success factor is introduced with a reclaimed ASR system – separation of the stored water from aquifers used for potable supply. This second factor can impose a greater limit on where such a system can be installed than the first factor.

Hydrogeological Setting within the GCC Region

The aquifers of the GCC states located on the Arabian peninsula are primarily divided into two physiographic provinces; the Precambrian shield in the western part of Saudi Arabia, and the sedimentary platform of eastern Saudi Arabia, Kuwait, Bahrain, Qatar, and UAE. There are also different geological provinces in Oman and Yemen, however these areas are not discussed in this paper. The Precambrian shield province includes outcrops of crystalline rock, and alluvium in wadis. Along the coast of the Red Sea, the alluvium thickens along the coastal plain. Wells within in this region are generally less than 200 meters deep (MOAW, et al. [26]). The main cities of the Precambrian shield province are Jeddah, Makkah, and Madinah, Saudi Arabia.

The sedimentary platform province includes thick sequences of sandstone, shale, and limestone formations dipping to the east. These formations include primary and secondary aquifer systems which are used to supply agricultural operations and water supply for cities. Water wells range from shallow (less than 100 meters) to very deep (greater than 2,500 meters) (MOAW, et al [26]). Major cities in this physiographic province include Riyadh, Saudi Arabia; Kuwait City, Kuwait; Dammam, Saudi Arabia; Manamah, Bahrain; Doha, Qatar; Abu Dhabi, UAE; and Dubai, UAE.

Water quality is variable in both the Precambrian Shield and sedimentary platform provinces. Groundwater is typically saline to hypersaline in proximity to both coasts (Red Sea and Arabian Gulf). Salinity generally increases down dip in the sedimentary platform aquifers, generally ranging from fresh near outcrop areas, and gradually becoming brackish, then saline, and hypersaline (MOAW, et. al. [26]). For example, the Um er Radhuma aquifer, which underlies a large portion of the platform area, contains relatively fresh water near the western part of the Rub Al Khali desert, but is hypersaline underneath the UAE (GTZ et al. 2008 [27]). This same aquifer contains freshwater east of Riyadh, but contains saline water underneath Bahrain, and Qatar (MOAW, et. al. [26]). One area in this region that is fairly unique is the Tihama, the western coastal plain of southern Saudi Arabia and Yemen. A substantially higher amount of precipitation falls along the coastal escarpment in this region, hence the freshwater/saltwater interface is closer to the coast than in other parts of the Arabian peninsula (MOAW, et. al. [26]).

Factors Affecting the Suitability of Potential Reclaimed Water ASR Storage Zones

Factors that need to be considered in the evaluation of potential reclaimed water storage zones include the water quality of the storage zone, in particular salinity, the permeability and transmissivity of the storage zone , and the permeability of confining and semiconfining units that bound the storage zone.

Salinity – Salinity is a key variable for evaluating potential reclaimed water ASR storage zones because it affects both recovery efficiency and the potential use of the aquifer for potable water supply. It is well-understood that high salinities are generally unfavorable for ASR system performance because the stored water is susceptible to unacceptable salinity increases from mixing with native groundwater and buoyancy-induced flow (density stratification). Aquifers that contain fresh or lightly brackish water could potentially be used directly as water-supply sources in the future. Therefore, using a fresh or lightly brackish aquifer as a reclaimed water storage zone may not be advisable, unless the benefits of such an ASR system far outweigh the potential benefits from using the aquifer for drinking water supply. If such an aquifer is to be used, either the reclaimed water may have to be treated to a very high quality or geographic separation must be maintained between the stored water and any potable water supply well.

Aquifer permeability - A second factor for storage-zone efficiency is the nature of the aquifer permeability. Generally, a primary porosity-dominated aquifer system will have a greater chance of successful recovery than a system that is dominated by secondary porosity features such as fractures or karstic voids. In a chemically bounded ASR system, a secondary porosity dominated flow system can substantially impact recovery efficiencies of the ASR system (Maliva et al. [28]).

Confinement - A third factor affecting storage-zone efficiency is the nature of the physical aquifer boundaries; both laterally and vertically. A chemically bounded storage system usually relies on having upper and lower confining beds, or semi-confining beds that are significantly lower permeability than the storage-zone beds. Similarly, a physically bounded system relies on having both lateral and vertical boundaries to prevent loss of the injected water. Confinement is particularly important for reclaimed ASR systems, in order to assure protection of potable-water supply aquifers.

Proposed Screening Process for Identification of Potential Reclaimed Water Storage Zones

A proposed screening process for identification of potential storage zones is as follows:

Step 1 – Identify the complete hydrostratigraphy in the region of the wastewater treatment production.

Step 2 – Eliminate aquifers that are important current or potentially important for future potable water supply.

Step 3 – Eliminate aquifers that have weak or non-existent separation from important water-supply aquifers identified in Step 2.

Step 4 – Eliminate aquifers that are too deep to be economically viable (e.g. greater than 2,500 meters).

Step 5 – Identify potentially suitable aquifers with permeability and water-quality characteristics amenable to ASR within aquifers that were not eliminated in Steps 1 through 4. Propose locations for exploratory drilling and testing to assess salinity, permeability, and confinement in detail.

The screening process largely involves the elimination of unsuitable aquifers and locations from further consideration. The remaining aquifers are candidates for further investigation. The overriding consideration is ensuring the protection of water resources. Although an aquifer may have hydrogeological conditions that are not conducive to high-recovery systems, the ASR system may still provide some supplemental reclaimed water when needed. Any recovery would be better than nothing, since the water would otherwise be put to waste due to a lack of demand.

EXAMPLE CITIES IN THE GCC REGION

Riyadh, Saudi Arabia

Riyadh is a city of approximately 6 million people in the heart of the Arabian Peninsula. Besides desalted seawater from the Arabian Gulf, potable water is provided from wells tapping the Wasia-Biyadh aquifer to the east (approximately 400 meters deep), the Minjur aquifer west and northwest of Riyadh wells (over 2,000 meters deep), the Um Er Radhuma (UER) aquifer far to the east; and some wells tapping deep alluvium and sands within the Wadi Nissah graben, located south of the city. New potable supply wellfields are being developed in the Wasia-Biyadh and the UER aquifers further east of the City (Khatib & Alami 2009 [25]).

The city of Riyadh is underlain by a thick sequence of Jurassic-aged limestone formations which dip to the northeast (see **Figure 1**) (SEURECA 2005 [29]). Water within these limestone formations is reportedly highly variable both in terms of yield and water quality and occurs primarily in secondary rather than primary porosity (Naeem et al. [30]). Aquifers within these limestone beds are reportedly used only locally for irrigation supply (Khatib & Alami 2009 [25]).

The majority of the city is serviced with a wastewater collection and treatment system. Treated wastewater from the Manfouha treatment plant located south of the city discharges to Wadi Hunifa. Numerous farms are located within the wadi downstream of the discharge area. The farms end approximately where the surface water disappears within surface sands over 40 km downstream of the discharge. The area where this surface water flow ends corresponds to the vicinity of where the Jubaila limestone is overlain by the brecciated limestones of the Arab formation. The Arab formation and its overlying Hith anhydrite are known in the area for the occurrence of caverns and other karstic features. Hence, the surface-water flow apparently ends where the strata underlying surface sands are more karstic in characteristic.

It is uncertain at this time whether or not treated wastewater that seeps into the limestone formations is potentially recoverable in the future. An extensive drilling and testing program would be needed to establish water migration. In addition, an extensive drilling and testing program would also be needed to identify whether or not any of the limestone formations underlying Riyadh may have characteristics that would be suitable for an ASR system in terms of transmissivity, salinity, and confinement. The regional stratigraphy indicates the presence of shale beds within the Marrat formation which overlie the Minjur sandstone aquifer. Therefore, underlying confinement would appear to be potentially suitable to protect this important water-supply aquifer, though this should be confirmed with confining zone testing. The Wasia-Biyadh and UER aquifers are not present in the stratigraphic sequence beneath Riyadh – the outcrop of the base of the Biyadh aquifer occurs approximately 40 km east-northeast of the wastewater treatment plant. Therefore, protection of these overlying aquifers does not appear to be a concern with regards to reclaimed water storage in the Jurassic limestones.

Since karstic limestone has evident disadvantages in terms of understanding ASR well storage efficiency; it is recommended that the search for potential storage zones focus on strata in which flow is more dominated by primary porosity. The calcarenitic Jubail, Hunifa, and Tuwayq Mountain formations could be explored for such a use. The potential occurrence of localized evaporite beds within these formations should be considered with respect to potential impacts to the quality of stored water.

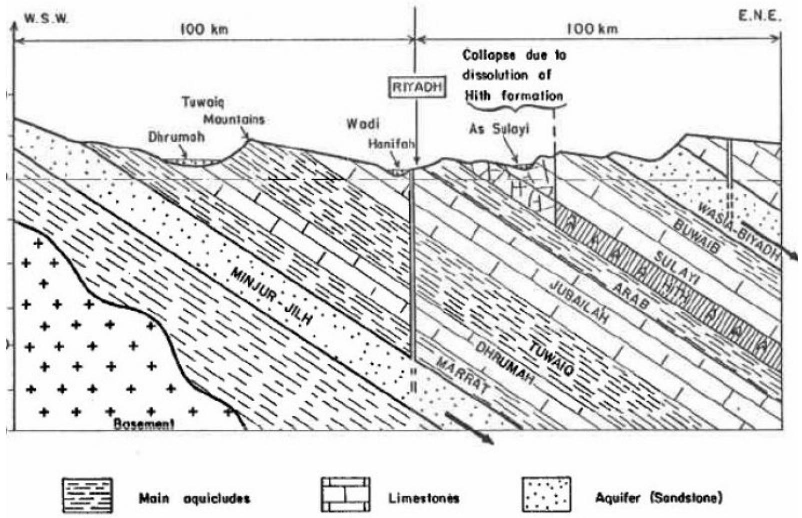

Figure 1: Generalized Hydrostratigraphy Beneath Riyadh (from GTZ 2005)

Jeddah, Saudi Arabia

Jeddah is located within the western coastal plain of Saudi Arabia, along the Red Sea. The coastal plain rises to an escarpment, which features outcrops of the Precambrian shield and Quaternary volcanic harrats (MOAW, et. al. [26]). Jeddah is located at the very north end of the region in southwest Saudi Arabia in which significant precipitation occurs, evidently because of orographic effects along the escarpment. Hence, the recharge to the coastal alluvium from wadis can be more significant than in other parts of the Kingdom.

The primary source of potable water supply for Jeddah is from desalted seawater. The wastewater treatment infrastructure is in early development for the city, with most wastewater going to septic tanks and hauled offsite. With the closing of Musk Lake for discharge of untreated wastewater, wastewater treatment is under increasing scrutiny, and extensive investment to rapidly improve the collection and treatment infrastructure is expected.

There is a major wadi system located approximately 30 km northeast of the Jeddah that includes some farms. Although the development of wastewater treatment infrastructure has been the primary emphasis of current improvements, it would be ideal if treated wastewater could be conserved for future beneficial reuse. Managed aquifer recharge using rapid infiltration basins (RIBs) or another injection approach could be utilized to store treated wastewater for agricultural uses. In addition, such storage could have some potential to “polish” quality of treated water prior to irrigation application. In order to develop such an approach, a drilling and testing program to characterize the hydrogeology and hydrochemistry of this wadis system would be needed.

Another approach to be considered in the coastal plain region is the salinity-barrier approach that has been developed in Orange County, California and other regions. This system involves the injection of highly-treated wastewater between the coast and inland water-supply wells to alleviate landward migration of saline water. In the Jeddah region, in particular moving south in the Tihama, there is a considerable amount of agricultural groundwater withdrawals from the coastal plain aquifers. A salinity-barrier system could protect the agricultural wells from saline-water intrusion. If pumped water is used solely for agricultural purposes, then there would not be the potential for indirect potable use of such water. The challenge for the Tihama region, such as in Jizan, is that the population largely is found in a higher-density rural distribution, and centralized wastewater treatment is not anticipated to be available for all in the near future.

Abu Dhabi, UAE

Abu Dhabi is the largest emirate in the United Arab Emirates (UAE) and includes the cities of Abu Dhabi on the coast, and Al Ain inland on the border with Oman. Abu Dhabi also gets its potable supply from desalted seawater, and has an extensive wastewater treatment infrastructure. Abu Dhabi also currently has extensive landscape irrigation using reclaimed water.

The coastal areas of the UAE, including Abu Dhabi are underlain by saline to hypersaline groundwater. Freshwater is found in wadi alluvium in Al Ain. Potable water ASR systems are currently in development, in shallow alluvial deposits in the Liwa area (over 150 km south of Abu Dhabi city, near the southern boundary with Saudi Arabia) and at Schwaib near Al Ain (approximately 140 km east of Abu Dhabi). Deeper formations throughout the emirate are expected to contain saline to hypersaline water, for example, the hypersaline water of the UER. Therefore, an appropriate storage zone would appear to be within shallow alluvial deposits.

Development of a shallow alluvial ASR system requires exceptionally good understanding of the fine-scale hydrostratigraphy, as understanding the flow and transport of injected fluid in a heterogeneous aquifer system is particularly important to allow recovery. Use of tools such as Time Domain Electromagnetic (TDEM) or resistivity surveying to map the shallow hydrostratigraphy horizontally (Dawoud et al. [31]), and use of advanced borehole geophysical logging in the vertical (Maliva et al. [32]) are methods used to collect such fine-scale data. In addition, use of groundwater modeling that can support such fine-scale data is important for designing a shallow alluvial ASR system.

The greatest likelihood of success of an alluvial ASR system would be expected to be in the eastern part of the emirate, at a distance from the Arabian Gulf. In an area such as the vicinity of Al Ain, storage of reclaimed water could be used to support agricultural operations, and such storage could result in some “polishing” of water quality. A pipeline of 140 km would be a major investment to develop such a system. However, there is a considerable amount of farming in the area west and south of Al Ain, and as fresh groundwater reserves in the area are depleted, a reclaimed pipeline system could become soon warranted as a means of continuing agricultural operations. Coupling of such a pipeline with ASR or MAR allows for this resource to be a longer-term irrigation solution for the emirate.

One approach that has been suggested (Missimer [33]), is to develop a shallow ASR system using slurry walls for lateral confinement. This approach would require a native lower permeability unit to serve as the base of the storage zone. Yet another approach could entail excavation of soils, placement of an impermeable liner, and then backfilling above the liner with native sandy soils around horizontal drain pipes. Since hypersaline groundwater underlies coastal areas in the UAE where population centers are found, this approach could provide for ground storage, as long as no salt residuals in the soils result in the stored water becoming saline. If the hypersaline unconfined aquifer is shallow, dewatering would be needed for construction, and hydraulic pressures from below the liner would be an important design consideration. Although excavation of soils to create a storage zone would generally be expected to be cost prohibitive, there is still potential for cost savings when compared to above ground tanks. A man-made physical-boundary ASR system, such as this, may be warranted for storing costly desalinated seawater to meet peak demands. Installation of such a system is less likely feasible for a reclaimed ASR system, unless perhaps, a system of paying users has been established, and some storage is needed to ensure continuous delivery to customers. Development of this type of ASR system for reclaimed water would therefore be expected to be at some time in the future, after the true economic value of such a water supply has been realized.

CONCLUSIONS

Reclaimed water ASR is expected to become part of the portfolio of water-resources management in the future in the Middle East. As reclaimed water is relied upon more, it will become more of a commodity and storage will be needed to provide a consistent supply. The added benefit of ASR will be some water quality “polishing”, although ASR should not be considered a substitute for wastewater treatment.

A second objective of ASR or MAR in the Middle East is to store water now that would otherwise be lost, for a yet unidentified future use. This rather imaginative approach is contrary to a traditional approach of reactive water management. However, there appears to be some momentum building for such an approach to take place. Since wastewater production is proportional to population, the largest urban centers have the greatest amount of water produced that can either be lost or saved for future generations. Hence, GCC cities such as Riyadh, Jeddah, Dammam, Makkah, Madinah, Manamah, Doha, Abu Dhabi, Dubai, and Muscat all would be good candidates to work for saving such water. Other Middle East cities outside of the GCC would also have potential to consider this approach. However, the hydrogeological setting and existing water supply play a critical role in determining what approaches for storage may be suitable in each location.

There may be some trade-offs. For example, if limited domestic supply is currently using an aquifer that is appropriate for a reclaimed water storage zone, then it may be appropriate to replace the water supply for such users for the greater benefit of all. One approach that should not be adopted, is to compromise on acceptable confinement separating potable supply aquifers from a potential reclaimed water storage zone. Although there is a strong body of evidence for "polishing" of water quality during underground storage, there are emerging contaminants of concern that appear to not be significantly attenuated underground. Strong public opposition may also be expected if reclaimed water is injected into what is considered a "pristine" potable water source.

Treatment of wastewater is a tremendous growth sector in the GCC. Construction of tertiary treatment facilities has been adopted as an approach to ensure safe reuse of treated wastewater. However, operationally, transient events, in which untreated wastewater is discharged, is an unpleasant fact, and such discharges should always be accounted for. In designing potential ASR or MAR systems for reclaimed water, it is important to understand whether or not untreated wastewater could get into the system. Ideally, storage can be constructed to prevent offsite migration of untreated wastewater, but this is not always achievable. In addition, there may be aquifer zones that have already been impacted by untreated wastewater discharges, and this should also be considered.

On the whole, wastewater is currently a net liability and environmental problem in the Middle East. However, as infrastructure is expanded, including appropriate use of ASR and MAR, then this liability is anticipated to soon become a major asset in this water-poor region.

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Technical criteria to improve artificial recharge in infiltration ponds and channels after eight years of management in Los Arenales aquifer facilities (Spain)

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Abstract
After eight years of management of the aquifers' artificial recharge facilities or Managed Aquifer Recharge (hereinafter MAR), which was constructed by the Ministry of the Environment and Rural and Marine Affairs (MARM) and the Castile and Leon Regional Government (JCL) in Los Arenales (a mainly eolic sand aquifer), more specifically in the Cubeta de Santiuste reservoir and the county of Carracillo (Segovia), simultaneous monitoring has been carried out on the artificial recharge, studying the strong and weak points in the facilities (channels, infiltration ponds and large diameter wells).

This article describes the experience of eight years of "experimental laboratories" and how this monitoring has led to the design and execution of improvements that have come from this experience and which are aimed at increasing the infiltration rate in a pre-operational situation, operational and post-operational stages.

Thus, headway is being made towards highly efficient designs in terms of water management for irrigation, with a view to its future use for urban storage in this aquifer and other analogous ones.

Keywords
Arenales aquifer, artificial recharge (AR), channels, clogging, DINA-MAR, infiltration ponds, MAR.

INTRODUCTION

The main aim of this article is to summarize the experience of eight years of monitoring in two "experimental laboratories" located at Santiuste basin and Carracillo district, Segovia, Spain, and how these tests have led to the design and execution of improvements in the design and operative aspects of infiltration ponds and channels that have come from this experience and which are aimed at increasing the infiltration rate, such as:

- In a pre-operational situation: Categorising detailed hydrogeological studies for specific objectives, selecting optimum prospecting techniques, etc.

- Operational stage: Improvements applicable to civil building works (construction of facilities and alteration of existing ones), establishing operating criteria such as "manual" control over inflow volumes according to the climate, pre-treatment, correcting the quality of water *in itinere*, and adopting techniques for the Soil and Aquifer Treatment (hereinafter SATs) to reduce the flow of air into the aquifer and minimise the clogging.

- Post-operational: The pouring of treated water into the channels during the semester in which there is no artificial recharge (hereafter AR) from rivers, cleaning and maintenance criteria, reduction of clogging, etc.

Framework

Los Arenales aquifer or Hydrogeologic Unit 02-17 covers an area of 1,504 km^2 located in Castille and Leon. Its origin is polygenic with a predominance of Arevalo facies, i.e. sands from a quaternary dune system with variable thickness (up to 50 m) filling a complex substrate from the Miocene epoch, notably argillaceous (Cuestas facies) or arenaceous and argillaceous (Puente Runnel).

The superficial quaternary aquifer exploitation has been intensified in the past few decades, causing the phreatic level to recede by 10 m, also bringing about salinisation and contamination processes. Therefore, three managed aquifer recharge devices for irrigation are being tested. These are shown in Figure 1.

AR first experiences in Los Arenales took place in La Moraña, where a thorough study was carried out to determine AR possibilities, both in the superficial aquifer and in its deeper levels. In the end, this option was discarded on fluvial waters, due to the difficulty in diverting water from the main rivers. This has led to the use of purified water for recharging purposes, which is now in progress.

Thanks to the actions carried out by MAPA and the i+R&D activities, the Cubeta de Santiuste is now a pretty well known aquifer, located in the west of the province of Segovia and south west of Valladolid. Lying on the left shore of Voltoya and Eresma rivers, it covers a surface of 48 km^2 and 600 ha of irrigation area. Artificial recharging (hereinafter AR) began in 2002/03 on a single channel with alternate basins, which has been successively expanded until today. The volume infiltrated into the aquifer has ranged between 0.97 (cycle 2004/05) and 12.19 hm^3 (2006/07). The device has been operating for 8 years and currently consists of 27 km of MAR channel, 5 infiltration ponds, 3 MAR wells, a River Bank Filtration (RBF) system and three artificial wetlands.

The county of Carracillo is located about 40 km east from the aforementioned region. It covers an area of about 150 km^2 and it is located at the interfluve of rivers Cega and Pirón. Irrigation has been practised extensively in this area with at least 2,700 ha using underground water. Artificial recharge by means of unlined irrigation ditches has also been practised for quite some time now, even though the largest devices started working in the winter of 2006/07, introducing 8 hm^3 into the aquifer, subsequently increased to 12 hm^3 in the 2009/10 cycle. It is composed by 40.7 km of MAR channel, 3 infiltration ponds, an RBF system and two artificial wetlands.

A thoroughly detailed description of these devices can be found in Fernández & López, 2002 and Fernández *et al*, 2009. These devices have remained the main “experimental laboratories” of the i+R&D DINA-MAR project and Tragsa Group since 2002.

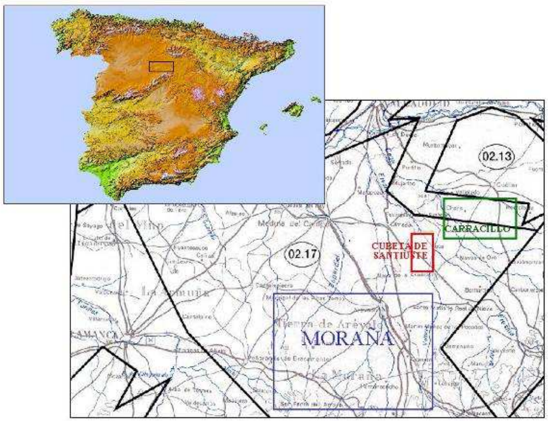


Figure 1. Geographic position of Los Arenales aquifer, also called UHG 02-17, and its position inside Spain (scale of the map 1:200.000).

OBJECTIVES

The main objectives include the study of the evolution of artificial recharge in the last eight years, particularly concerning the evolution of the infiltration rate and volume of infiltrated water, so as to analyse the performance of the channels and basins and make them more effective by means of structural improvements and/or by Soil and Aquifer Treatment techniques.

METHODS

Several materials have been used for data collection purposes. Those used to determine onsite permeability and infiltration rates are worth mentioning, such as double ring infiltrometers, pilot basins and Lugeon and Lambe tests), samplers, etc. Storage variation in the aquifer has been examined through the Water Table Fluctuation method (WTF) (Healy & Cook, 2002).

The study of the unsaturated zone (UZ) is being accomplished by means of humidimeters, thermometers and tensiometers connected to data-loggers, built in both telemonitoring stations (DINA-MAR ZNS). They simultaneously register 5 patterns within the surroundings of an AR channel. Sensors are placed crosswise to the channel, so as to identify and quantify the humidification bulb and determine its morphology, while estimating the amount of air held in the ground pores (in Fernández, 2009).

Additionally, another technique has also been implemented since 2010, consisting of the use of thermographic camera model Therma-Cam E2 developed by Flyr systems, in order to study the distribution of clogging due to temperature differences between these and the zones not affected by clogging, which is evidenced in the different colourings.

RESULTS AND DISCUSSION

Studies aim at determining the effects of artificial recharge frequency and flow on the infiltration rate and its control so as to increase the effectiveness of channels and basins

Period

The evolution of the infiltration rate at a given point of the East channel of Santiuste along the artificial recharge cycle (2004/05) was monitored by running eight infiltration tests on a channel section which was specially confined and conditioned (Table 1 & Figure 2). The morphology shown by this table is aligned with Blaxejewski's "average" curve (Blaxejewski, 1979).

DAY Nº AR CYCLE (2004/05)	INFILTRATION RATE(mm/year)
1 (01/10/2004)	35
7	159
14	140
19	315
39	301
108	300
126	291
149 (26/02/2005)	289

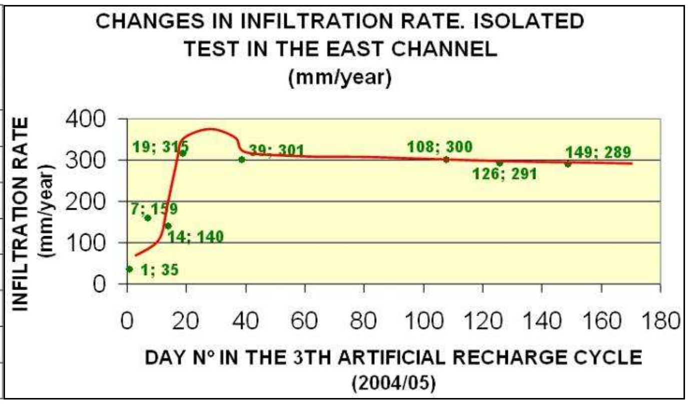


Table 1 & Figure 2. Infiltration rate evolution along the MAR cycle 2004/05 in an isolated channel of Santiuste basin inside the East channel.

The graph shows that the infiltration rate has been increasing continuously since November 1st when the concession started, reaching its peak around the month when the artificial recharge began and until the temperature drop occurred in early December. Saturation, air trapped in the aquifer pores and low temperatures trigger its decrease for approximately fifteen days.

As regards the rest of the artificial recharge cycle, it shows a slightly decreasing tendency during the winter (frost cycles seem to cause a delay in the infiltration rate increase). At some point slightly after half of the period (February-March), the aquifer has already trapped significant amounts of air (up to 35% according to references, in Stuyfzand, 2002) which may also be accompanied by *Lisse effect* (Krul & Liefrinck, 1946). This trapped air has not been quantified in the experimental laboratory, even though its presence has been noticed, as well as its negative effect on artificial recharge, as shown by the variations registered by the tensiometers at the DINA-MAR ZNS stations. The last tests allow us to estimate that air reaches 25% by the end of the second month of AR. As spring comes closer, bringing deaeration, the curve shows a more or less constant morphology. The final result is a decrease in the infiltration rate, accumulative in successive cycles.

Operating aspects derived from this study are aimed at practising manual control of the inlet valve according to the circulating flow, which should be shut off during frost cycles. This control is carried out by irrigators with the advice of DINA-MAR.

MAR Rate

Flows derived from Voltoya river for MAR, upon variation of volume stored in the aquifer under the same conditions and for all cycles between 2002 and 2005, when the device was expanded by building a new channel, are summarised in Graphic 3.

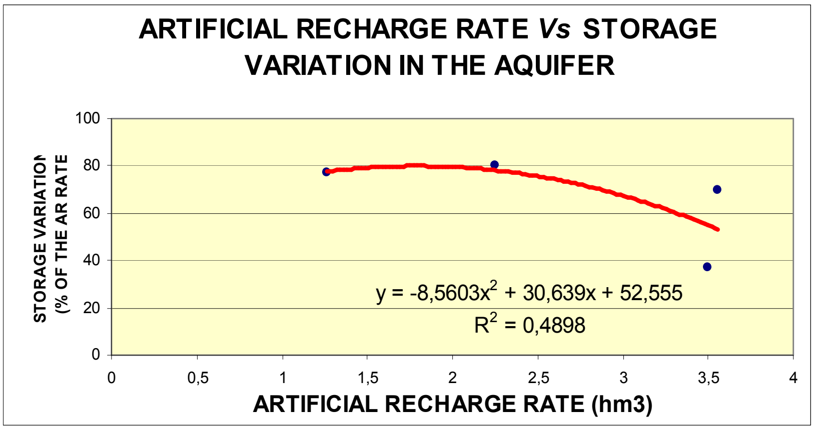


Figure 3. Relation between artificial recharge volumes taken from Voltoya river and aquifer storage increments measured by means of Water Table Fluctuation (WTF) method for initial four years' period in the East channel (Old).

The graph attached is based on calculations corresponding to the flow infiltrated during the first four-year-period, against the average recharge flows. Percentage wise, the infiltrated volume decreased along the four-year-period of “isolated” operations of the East channel, mainly as a result of clogging. Results corresponding to the first cycle are anomalous due to the adjustments made in the channel until achieving “optimal” performance. There are little data, as the device was substantially changed in 2005 after building and setting the "West" channel, which is longer and more effective. These data have been used to deduce the equation of the interpolating polynomial curve.

$$y = -8.5603x2 + 30.639x + 52.555$$

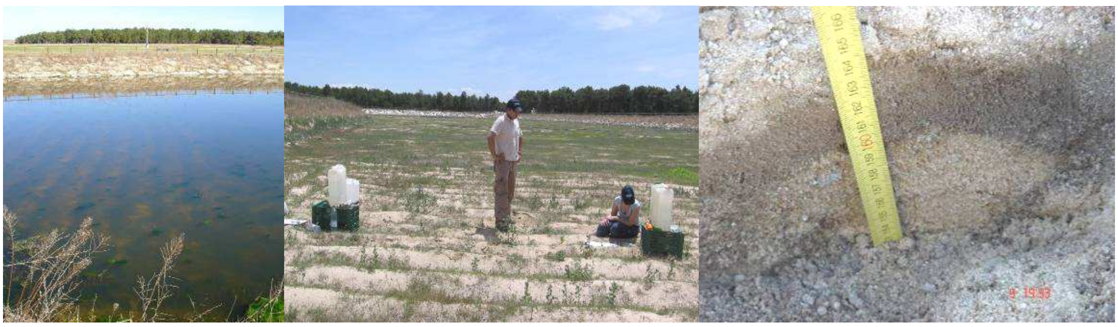
The maximum value obtained when deriving the equation and matching it to zero is 1.765, which means that the highest infiltration percentage is near the average value of the tested AR volumes, corresponding to flows of about 150-200 l/s. Therefore, more circulating flow implies more infiltrated volume. However, at low flows, there is a maximum value which is included in this interval.

Actions carried out on the morphology of infiltration ponds and channels

Infiltration ponds

In order to improve their effectiveness, the water-environment contact surface has been widened by the ploughing of furrows, which also allow the silts to be deposited in bottom furrows due to gravity, the ridges remaining higher up and relatively cleaned.

In order to quantify the differences between a flat bottom or a ploughed one, approximately 14 infiltration tests in Santiuste basin’s decantation and MAR pond were carried out. The first ones with flat bottom (tested in 2007 September). Shortly after and in order to find out the most suitable spacing between furrows to obtain the highest infiltration values, furrows were ploughed in the initial decantation basin at a “wavelength” between 60 and 100 cm with a Roman plough. These tests were repeated in June 2008 and 2009, at the end of each AR cycle and once the basin had dried up (Table 2 and Figures 4 and 5).



Figures 4 a-c). Furrows plugged with different width at the bottom of a decantation and infiltration pond, infiltration test by double ring infiltrometer in the convex and concave surface of them and the clogging profile. Headwater of the Santiuste´ AR device.

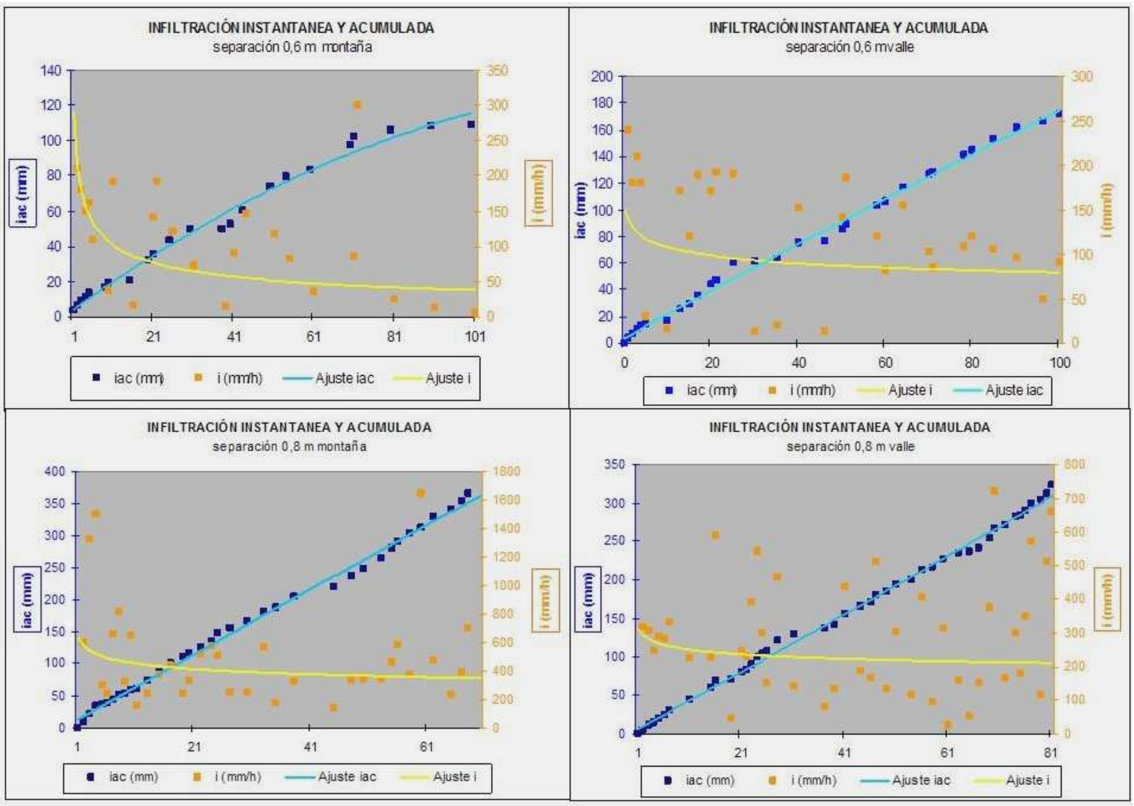
	Coordinates UTM		Campaigns: t & inf. rate Sept 2007/Jun 08/Jun 09		Characteristics	
STATION	**X**	**Y**	**Test (min)**	**Infiltration rate (mm/h)**	**Site**	**Soil type**
POND 1	369832	4557443	100/255/101	2500/95/38	ridge 0.6m	sand
POND 2	369839	4557436	100/248/100	100/90/65	valley 0.6m	silty sand
POND 3	369821	4557448	148/120/68	90/**420**/100	ridge 0.8m	sand
POND 4	369803	4557426	180/120/81	220/**232**/108	valley 0.8m	silty sand
POND 5			150/150/nd	200/350/nd	ridge 1.0 m	sand
POND 6			nd	250/220/44	valley 1.0 m	silty sand

Table 2. Results of infiltration tests from the headwaters´ infiltration pond. Values collected in 2007 September (pre-operational), 2008 June and 2009 June (post operational) respectively.

Comparing results of 2008 and 2009 (June in both cases), with smaller spacing between the ridges (60 cm), infiltration rates fell over fifty percent in the ridges and less in the furrows.

With 80 cm after making the furrows, the rates increased over four times in the ridge and such increase was slightly higher in the valley, with larger drops in the ridges than in the furrows. One year later, they turned 90 and 220 mm/h (ridge/furrow, respectively) with values of 420/232 mm/h.

With 1 m spacing, results were similar to those of 2007 in the ridges, being rather lower in the furrows.



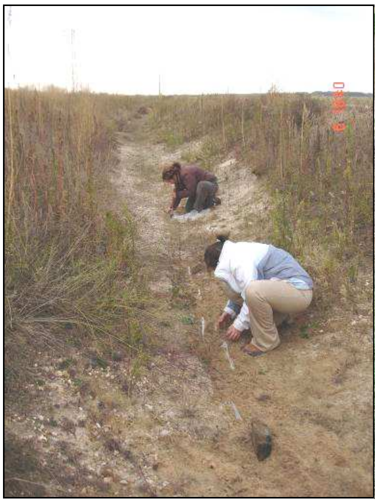
Figures 5 a-d). Test interpretation graphs with double ring infiltrometer in basin 3, with 0.6 and 0.8 m spacing. Data collected in from June 2007 to June 2010. Graphs of these results & interpretation of 2008.

All the results confirm that, according to the test place and conditions, furrows increase the infiltration rate when compared with flat-bottom basins, with higher values in the ridge of the mounds than at the bottom of them. Although it is not possible to set a defined trend, given the few number of tests, furrows with 80 cm spacing perform better in general terms. However, these results are not final.

It is a good practice to open furrows with disc ploughs, which proves to be less harmful than the mouldboard plough.

AR Channels
The main lines of action to increase the infiltration rate and the total infiltrated volume in the channels bottom and walls have focused on the channel morphology itself. They also focus on flow regulation and filtering of silt in the AR water.

From 2008 two sections of the channel have been tested with a longitudinal ridge fitted with a 5-metre long geotextile (Figs. 6). The section with a higher number of tests is located as from point UTM 30-369417/4559040. The tests are repeated on an annual basis in the section centre and ends (Table 3).



Figures 6 a-b). Geotextiles installation in the bottom of the AR channel at Santiuste basin on 2008 November and infiltration tests seven months later.

In the 2007 and 2008 campaigns the infiltration rate plunged, which could be consequence of several factors, such as the excess of sediments in the recharge water. However, in 2009, rates went up again. In the geotextile areas there was an actual retention of silt, with an associated infiltration fall. In the areas without geotextile rates were again similar to 2007.

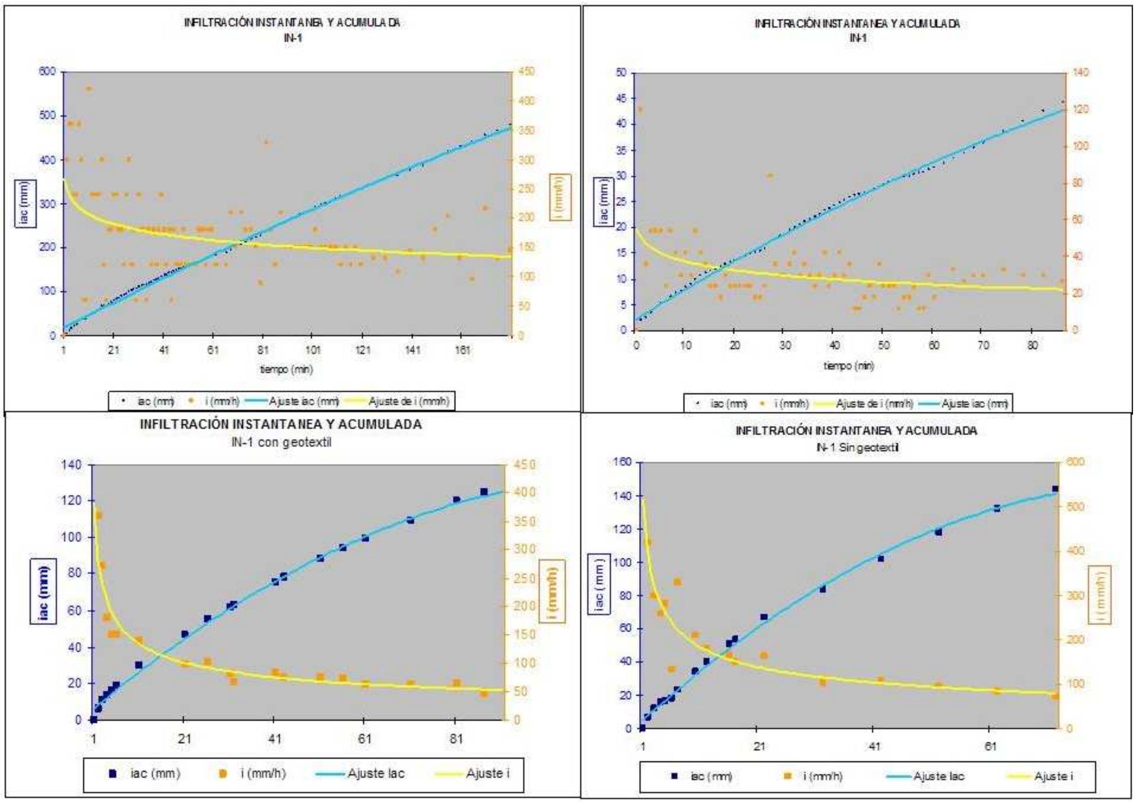
	Coordinates UTM		Campaigns: t & inf. rate Sept 2007/Jun 08/Jun 09		Characteristics	
CHANNEL STATION	**X**	**Y**	**Test (min)**	**Infiltration rate (mm/h)**	**Site**	**Soil type**
IV-1 GT-1i	369417	4559044	180/90/80	130/13/44	No geotextil	sand
IV-1 GT-1f	369417	4559045	180/103/90	210/22/38	Above geotextil	silt
IV-1 GT-2i	369714	4557572	120/86/na	90/108/na	Above geotextil	silt
IV-1 GT-2f	369713	4557576	90/70/60	150/160/150	No geotextil	sand

Table 3. Infiltration tests above and just near those channel fragments which count on installed synthetic geotextiles. Values for 2007 September (pre-operational), 2008 June and 2009 June (post-operational) respectively. IV-1 GT 2 was destroyed due to a flood.

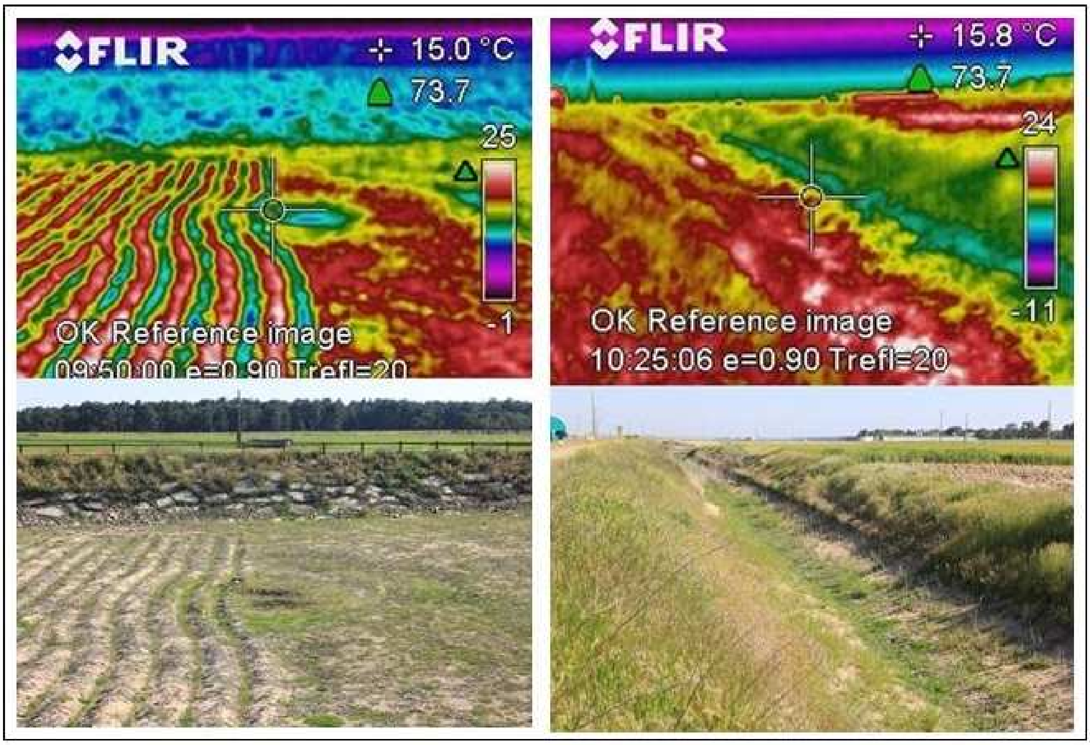
According to the analysis of results, the rate is usually higher in short-term tests, though it is lower in longer-term tests, so, it is believed that there is a higher silt concentration at a certain depth beneath the channel. Besides, although it is convenient to increase the number of tests to draw reliable conclusions, the infiltration trend showed a higher slope in the last tests, where the infiltration curve (marked in a lighter colour) allows reducing the increase of soil sediments, which accumulate in areas of lower hydraulic conductivity (first in the surface and then in the 40 to 60 cm depth range). These results must be considered for maintenance planning.

Studies with thermographic camera
This line of action first implemented in May 2010 enables the detection of areas where clogging processes may occur by analysing the distribution of temperature at the bottom of ponds, channels and profiles. In general terms, eolic sands tend to be clearer than clogging processes. They have a heavy slit load and are typically darker. These contrasts corroborate furrows' effectiveness, with more clogging present in furrows than in valleys and with quantitative differences throughout furrows. Figure 8 a) shows a basin along with its thermography. Thermal variations in the furrows result from differences in the height, vegetation or appearance of organic and physical processes on the surface. They are characterised by a darker colour and a higher heat absorption capacity.

In the walls of the channels and up to the elevation of the flowing water, a darker colour can be noticed. Besides, at the bottom, the areas with buried geotextiles, which have colder colours, can be differentiated from those without equipment, which are surrounded by clogging processes (Figure 8 b).



Figures 7 a-d). Infiltration tests in IN-1 station since September 2007, June 2008 and 2009. Double ring infiltrometer tests graphics and interpretation.



Figures 8 a-b). Comparison between normal and thermographic aspects, in order to allocate clogging processes. Left: MAR infiltration pond; right:: MAR channel.

Even though this technique is not sufficiently developed to perform quantitative measurements, the quality of the measurements is of great significance and allows for continuous improvement actions. thermographic photos are taken at different hours of the day, in order to find the best time for their detection.

CONCLUSIONS

The downward infiltration rate in the channels and basins of Los Arenales aquifer is being reduced through flow regulation and pre-treatment techniques (slit filtering and air reduction) in the AR water. Flow regulation allows reducing the amount of silt and air entering the aquifer. It can be noticed that the most effective recharge occurs at high flows or flows around 150 to 200 l/s. Higher flows reduce the infiltration rate due to water oxygenation and the increase in the suspended particulate matter. Positive results are coming from communicating vessels systems in shut-off devices and buried channel sections fitted with filter pipes.

As for the performance in the morphology of basins and channels, furrows have ultimately increased the infiltration rate in all the tests with respect to those carried out in “flat-bottom” basins. 80 cm spacing between ridges delivered the highest values, with rates that nearly double those obtained at 60 and 100 cm. However, data obtained so far do not render results final. Apparently, disc ploughs present better results than mouldboard Roman ploughs.

In maintenance operations, it is of paramount importance to keep furrows clean, both in basins' furrows and in the longitudinal ridge at the centre of the channel. The installation of low permeability and easy-to-replace geotextiles has been proposed for silt removal. Infiltration rate results were 160 mm/h without geotextile and 108 mm/h with it. The question is whether such minor difference in the infiltration rates makes up for installation costs and removal of “dirty” geotextile.

Test curve infiltration analyses indicate that there is a strip of land, between 40-50 and 60 cm deep, where drops in the vertical permeability ratio (Kv) have been detected, broadly attributable to clogging processes derived from a decrease in temperatures and calcareous precipitations. Therefore, mechanical treatment in conservation tasks should go far deeper in (increasing the strip removed to 40 and up to 60 cm deep).

With certain limitations, all these operational aspects could be applied to scenarios analogous to the Arenales aquifer.

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Implementation of Techniques for Soil and Aquifer Treatment (SAT) in Spain. Contributions to the state of the art

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Abstract
Among the lines of action carried out within the framework of the DINA-MAR project or the management of aquifer recharge taking into account sustainable development, one of the most important ones was the adoption of specific techniques of Soil and Aquifer Treatment (SAT) in different Managed Aquifer Recharge (MAR) experience areas with the aim of increasing the rate of infiltration, the effectiveness of the existing facilities, and creating design criteria for future ones.

The line of work has shed quite a lot of light on technical criteria for increasing the effectiveness of action taken in detritic aquifers and superficial facilities, proposing various "problem-solution" binomials based on engineering criteria, risk assessment and environmental impact.

Keywords
Artificial recharge, clogging, DINA-MAR, infiltration, Managed Aquifer Recharge (MAR), Soil and Aquifer Treatment (SAT).

INTRODUCTION

Within the framework of the DINA-MAR project and after eight years of carrying out trials and analyses for the design of artificial recharge facilities, this line has brought about the design and implementation of Soil and Aquifer Treatment Techniques (from now on, SATs) applied to water from its original source (in both quantity and quality), to the receiving medium and to the combination of all of them.

The SAT techniques most used in the study areas and which underwent permanent research has been:

- Study on the biggest impact that affects the AR facilities: Clogging.
- Influence of the period and flow volume of artificial recharge on the infiltration rate and effectiveness of the facilities (The studies are being carried out in channels and infiltration ponds).
- Action taken on the morphology of the receiving medium (recharge wells, channels and infiltration ponds).
- Reduction of air inflow into the aquifer around the AR facilities.
- Cleaning and maintenance operations.

It should be remarked that in this article is used the classic Dutch connotation for SATs (Krul & Liefrinck, 1946), as techniques to be applied to increase infiltration rate by means of operations on soils, aquifers and artificial recharge water; despite in the last years there is a tendency to apply this term to managed aquifer recharge operations from waste water treatment plants.

In spite of the fact that this article might result a bit generic, it compiles results from eight of research very summarized, and some pieces of information may be extended with the references.

OBJECTIVES

The aims of this article are three:

- Study the problems that affect the three aquifer recharge management devices used as pilot areas for the DINA MAR R&D+i project, which has been operational for up to eight years, with the aim to question its suitability.

- To propose various "problem-solution" binomials, to improve effectiveness of implemented facilities based on SATs, engineering criteria, risk assessment and environmental impact. It is so generated a continuous loop: new detected deficiencies-new improvements, increasing the state of the art.

- To propose a SAT techniques corollary, to be applied at analogous sceneries.

METHODS

The methodological approach consists of monitoring and tracking three MAR facilities operative in Spain: Santiuste basin and Carracillo district, in Segovia province, and Guadiana canal at Ciudad Real (Spain). The first couple consists of infiltration ponds, channels and recharge wells; the last one is composed by a battery of 25 boreholes.

The main data collected have been those related to permeability and infiltration rate *in situ,* by means of double ring infiltometers, Lugeon and Lambe tests, and also triaxial permeameter in the laboratory. Likewise sequential water current gaugings along the channels have been measured.

The storage variation in each aquifer is performed with the Water Table Fluctuation, WTF method (Healy & Cook, 2002), from the measurement of control networks made up of more than 50 water points in each case, and this information is treated with algebraic mapping operations using a Geographic Information Systems (GIS).

The dissociation of the volume coming from the management of aquifer recharge and the rainfall seepage has been carried out by means of Hydrological Evaluation Landfill Performance (HELP) model (Gogolev & Ostrander, 2000) and by water balances in isolated zones

The study of the air inflow into the aquifer around the MAR devices and the monitoring of unsaturated zone's parameters is accomplished with humidimeters, termometers y tensiometers connected to data-loggers (DINA-MAR ZNS stations).

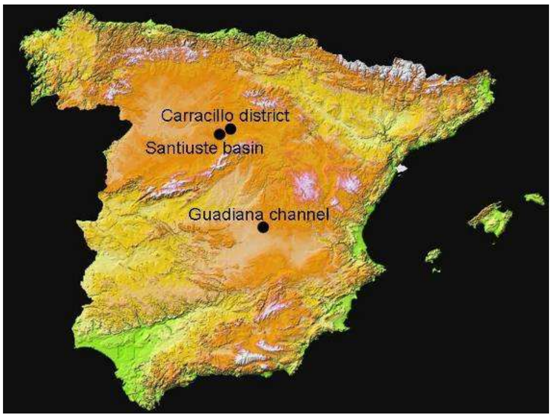

Figure 1. MAR facilities where pilot tests are accomplished. 1- Santiuste basin, 2- Carracillo district (Segovia province) and 3- Guadiana channel (Ciudad Real province).

RESULTS AND DISCUSSION

Study on the biggest impacts and inadequate designs that affect the MAR facilities

The main problems located and the action lines undertaken to resolve them in the three managed aquifer recharge mechanisms mentioned have been:

1-2. Being a superficial type device, alternatives have been analysed for the recharge in areas with a large occupation of land and pine groves, where it is not possible to implement superficial type devices. For this reason, studies and testing have been undertaken on methods for inserting sub-superficial devices inserted into the pipes themselves that direct the water coming from the intake rivers towards the MAR systems.

3. The original dams have been modified to facilitate water purification through the river alluvial and later captured in nearby wells, thus facilitating River Bank Filtration (RBF) facilities.

4-5. In all the cases have been detected problems caused by air inflow into the aquifer together with physical clogging (in high diameters infiltration wells and at the bottom of infiltration ponds and channels). The stopping devices emplaced at the channels headings and in the connections with the supply pipes, produced a rise in the dissolved oxygen concentration of MAR water and a reduction in infiltration rate and total amount due to Lisse effect (Krul & Liefrinck, 1946). According to isolated calculations in Santiuste basin and using the techniques of Blaxejewski, 1979, the infiltration volume decreased until a 25 % due to air inflow into the detritic aquifer in the second MAR cycle (2003-2004).

6-7. Variations were experienced in some aspects related to the hydrogeological operation of the system, such as changes to the water quality in adjacent wetlands, changes in the flows of springs, etc. These problems have required specific studies, usually based on special induced artificial recharge designs.

8. Clogging problems in infiltration ponds at different depths, as well as the generation of carbonated crusts in sectors of the aquifer with a chemism reducer of subterranean waters or originating from recharge during frost cycles.

9. Inadequate well designs and recharge probes that enable fines to enter, abundant intake of air in the aquifer and limited infiltration, usually to take advantage of preexisting abandoned wells.

10. The unbalanced distribution of clogging processes was detected in the slopes, which made it necessary to modify the morphology of the canals and ponds and design specific cleaning techniques.

The figure 2 synthesizes all this information.

Proposal for the design and implementation of Soil and Aquifer Treatment Techniques and structural designs

The solutions proposed for the environmental impacts and dysfunctions mentioned have involved several years of research and progressive improvements. Generally speaking, the initiatives have been a reiterative process, up to the point that there are still several problems that are not adequately resolved and designs are pending construction. However, the current devices present notable quantitative and qualitative improvements over the initial design built eight years ago. The main activities undertaken, presented in the same order of the statement of reasons, include:

1-2. The recharge devices inserted into pipes involved the insertion of pipes with punched holes and filtration sections for *"in itinere"* recharge, as well as the insertion of filters for pre-treatment of the water in the same pipe. These stretches are surrounded with gravel in "drainage trench" type devices. There is also a maintenance programme for periodic replacement of the gravel.

3. The construction of dams for the intake of water from the river has been accompanied by a private initiative of well perforation in special purpose RBF systems next to the intake dams for artificial recharge or from the Guadiana canal.

4, 5-9. The newly constructed artificial recharge dams have a specific design based on the insertion of decanters and filters, elevated diameter and perforation of a probe inside it. Many of the designs applied are inspired from the works of Bouwer, 1999 and 2002 and Olsthoorn, 1982. Communicating vessel systems are also being tested in all detention devices and chutes to minimise aeration, as well as emergence under the water level, minimising the cascading effect in the constructions. In general, this has achieved reduced entry of fines and air into the aquifer and greater infiltration rates.

6-7. Devices have been designed for the induced modification of the quality of artificial recharge waters destined for environmental purposes. In the case of hydric regeneration of saline wetlands located at the edge of the aquifer (La Iglesia lagoon), a chute has been constructed, which forces the interaction of natural salts that lie in the area with the recharge management water, thus preserving its quality. In this manner, with high surface interaction and little time, much residence time is compensated and less contact surface with which the water from these wetlands are generated, resulting from increased deep flows of subterranean water.

8. The activities in the infiltration ponds have involved studying the formation and distribution of clogging processes on the horizontal and vertical. Ploughed furrows have been tested in pilot ponds with different wavelengths, and several infiltration trials undertaken with annual frequency. In general the 80cm equidistant ridges have provided higher infiltration values. In the case of canals, there are replaceable geotextils in the furrows, which facilitate the harvesting of fines and replacement with clean natural land. A recharge programme has also been designed with manual valve control depending on climatic circumstances. In general, artificial recharge is minimised on frosty days to prevent the formation of carbonated crusts.

10. After several trials, slopes with a 2:3 incline have been built in the channels, with a higher rate of infiltration in the walls observed. In fine sands 1:1 slopes have been opted for given the greater durability. In order to improve cleaning and maintenance operations, a specific Basin Cleaning Vehicle (BCV) has

been designed, which is adapted to the morphology of the canal and based on the use of traditional machinery with easy assembly and disassembly structural modifications.

The detailed description of the least modern devices is found in the bibliography (Fdez. Escalante, 2007). Specific activities for canals and ponds can be found in Fdez. Escalante *et al,* 2009. Some management parameters are found in Pérez-Paricio, 2007.

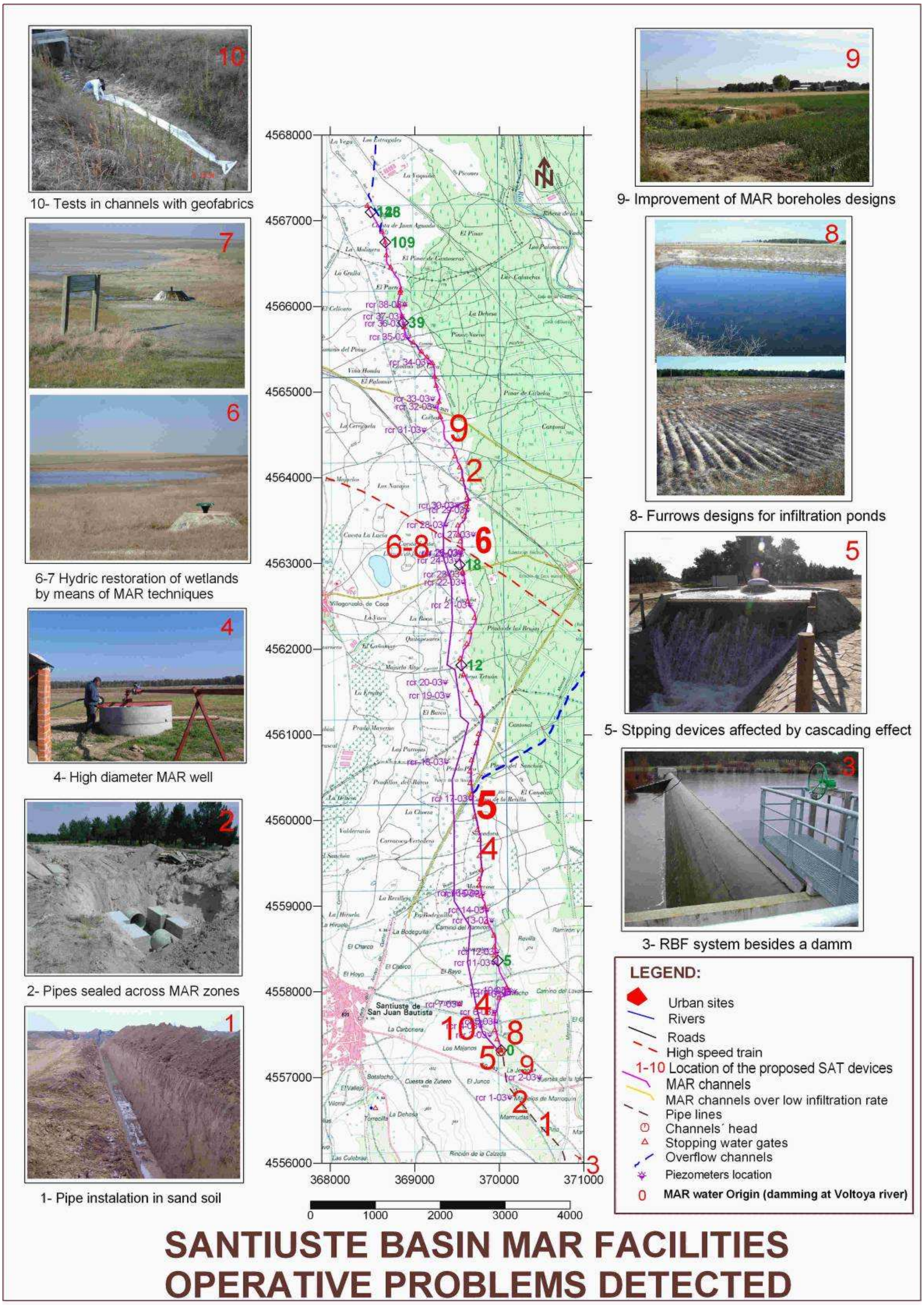

Figure 2. Examples and position of the biggest impacts and inadequate designs that affects the MAR facilities and also some action lines to solve them.

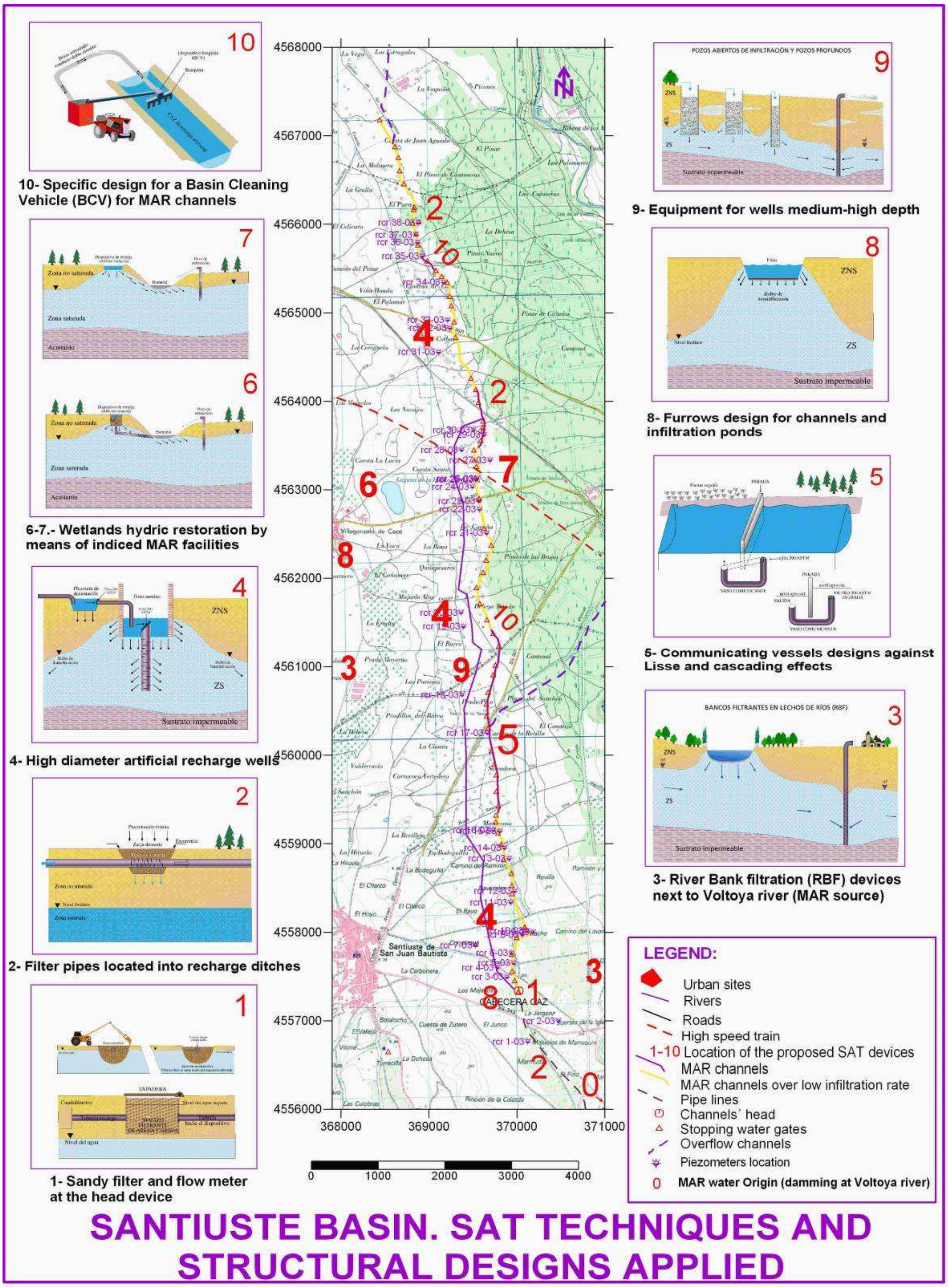

Figure 3. Proposal of solutions based on SATs techniques and structural designs to solve the biggest impacts and inadequate designs that were affecting MAR facilities. Example for Santiuste basin (the results from the rest of the pilot MAR sites are also enclosed in these techniques): 1-2. Filters along the pipe inside sandy aquifers without disposal of superficial terrain. 3- Proposals for RBF systems close to MAR dams. 4- Designs for high diameter artificial recharge wells. 5- Devices to avoid the increase of dissolved oxygen in recharge waters. 6-7- Hydric restoration of wetlands by means of induced recharge. 8-9. Designs to minimize the clogging and Lisse effects in wells, channels and infiltration ponds. 10. Specific BCV design for these test sites.

Corollary of SAT techniques to be applied in analogous sceneries

The results of all the trials and the study of art at a global level have enabled the following operational corollary to be proposed, which has been designed with the aim to present a catalogue of options that can be applied in similar scenarios to combat the problems and impacts that affect recharge management mechanisms. It is therefore about a system for decision making support.

There have been distinguished tour kinds of operations: applied to water from its original source (in both quantity and quality), to the receiving medium (in both soil and aquifer), to the combination of all of them, and also management parameters plus cleaning and maintenance operations

Recharge water (quantity)

- Temporary storage in surface reservoirs
- Control of the flow velocity of recharge waters
- Avoid operations in freezing weather /season /cycles
- Use of thermostatic cameras/Chambers
- Selective criteria at origin
- **Cleaning and maintenance**
- Use of BCVs (Basin Cleaning Vehicles)

Recharge water (quality)

- **Preselecting:** selective criteria for the origin of recharge waters: Filtering and decantation waters **AR**, etc. (membranes, mud lines, filters…)
- **Run-off** tramps and decantation / stagnation structures
- Anticorrosion **devices**
- **Design** and preservation of **slope** (rubble works, gabions…)
- **Design of channel bottoms** (furrows), use of geofrabrics
- **Limitation of the water layer height: Pretreating** type ***DBP*** *(Desinfection by Products)*:Cl, I, O3, H2O2, UV rays, etc.
- Cleaning **vegetation** during AR / Specific plantation during summer season
- **Avoid aireation** on AR waters: communicating vessels, open structures, velocity / reduce the speed of waters in channels
- **Deaireation** using piezometers, increase distance between injection-extraction points
- **Dual systems**: Algae drying, natural bed drying, cryotreating, cracking (cake), scarification of silting zones and cleaning /replacement
- **Isolation** from atmosphere/sunlight
- **Specific fishes** to decrease TOD concentration (e.g. medaka).
- **Filtering beds and chemical additives**, to eliminate clogging layers
- Avoid **recycling effect**
- **Denitrification** (e.g. anammox): irrigation / watering tuning the deep of pump placement
- **Avoid natural salinization**: Induced recharge. Barriers in salty areas

Receiving medium (soil and aquifer)

- **Pretreating of water for MAR**
- **Natural drying** of bed, and cracking
- **Cryotreating**
- Use of **dual systems** allowing cleaning of one of them whilst the other is operating
- **Inverse pumping** in wells close to canals/ponds
- **Alternate normal and inverse pumping**
- **Backwashing** in geo fabrics, membranes and filters
- **Use of jet type** cleaning techniques
- Mechanical (wall brushing and scratching) and chemical (use of chemical additives) techniques for the **regeneration** in recharge **wells**
- **Cleaning** techniques with the highest possible frequency
- **Alternate of injection / well daily pumping**
- **Use of specific Basin Cleaning Vehicles (BCVs)**

Management/good Practices/Use criteria and codes

Management parameters and ex situ techniques

- Choosing the most adequate **period** and place for MAR
- Initiate '**soft**' **MAR cycles**
- Input **flow** and **speed control**
- **Monitoring chemical properties** of MAR water during recharge cycles
- Use of **protection devices** for fauna and people
- Early adoption of the **best available techniques**
- Design and adoption of a proper Watch and **Control Program**
- Specific protocol for **clogging control**
- Protocol for proper **hydro-mechanical** aspects in space and time
- **Integral systems**: all elements are interconnected
- Limit fertilizers
- Promote participation of farmers in water management
- Installation of adapted waste water treatment plants and decrease untreated spilling
- Protected perimeter
- Public use regulation

Figures 4. Corollary. Lists of options to solve problems affecting to MAR facilities by means of SAT techniques, new designs and changes in the management practices. The operations can affect either recharge water´ quality and quantity or the receiving medium (soil and aquifer). Management parameters plus cleaning and maintenance operations are also considered.

CONCLUSIONS

After eight years of operation of three aquifer recharge management systems in Spain, negative environmental impacts and deficiencies have been detected that can be resolved. Many of these can be resolved by adopting SAT techniques, new structural designs and changes to management parameters.

The majority of impacts detected correspond to clogging processes (greater scale and intensity), excessive intake of air into the aquifer in the recharge water and limited pre-treatment of the recharge water. This is consolidated as the most effective alternative for the correct operation of the mechanisms to prolong their average service life.

The design changes and management parameters must be created “a la carte”, depending on the climate and characteristics of each system.

A corollary has been proposed with the aim to present a series of options to be considered when implementing a solution for a problem that affects the superficial type recharge management mechanisms, such as a graphical support instrument for decision making.

The initiated process is open. Each improvement applied becomes a new element to improve, which is why experiences must be complemented with technological monitoring of improved techniques available that may arise in similar scenarios.

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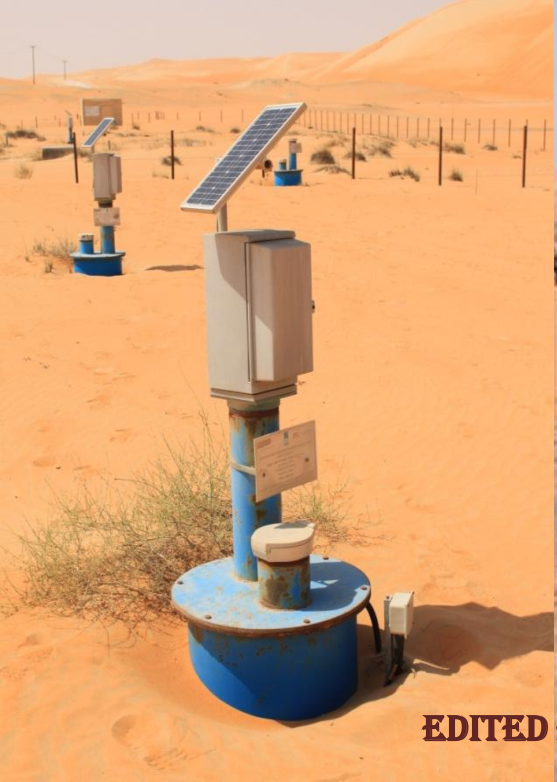

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